Shear Reinforcement of RC Beams Using Carbon Fiber Sheets

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Summary

An experimental study was conducted on shear reinforcement of reinforced concrete beam using carbon fiber sheets. The results indicated that ultimate shear strength of the strengthened beam is about 1.3 to 1.8 times higher than that of the virgin beam and similar shear-reinforcing effects of the sheets are obtained for the crack-damaged and afterwards repaired beam.

Introduction

External epoxy-bonding of thin carbon fiber reinforced plastics sheets (hereafter called the CF sheet) is a superior technique for strengthening of existing reinforced concrete (RC) structures or repair of deteriorated RC ones since the CF sheet is light in weight, high in stiffness and strength and superior in durability, and also the bonding work is easy and not skilled. Authors have already performed experimental studies on flexural and shear reinforcement of RC beams using the CF sheets and presented the results that ultimate flexural strength of the strengthened beam is increased by about two times that of the virgin one and any shear cracks can not be observed in the shear-strengthened area of the beam (1,2). This paper presents the results of a further experimental study on shear reinforcement of RC beams using the CF sheets.

Experimental

The configuration and bar arrangement of the RC beam specimen is shown in Fig.1. The specimens had a same cross section and two different lengths and shear spans. The stirrups were not arranged in the central shear span. Some specimens were initially crack-damaged by pre-loading and subsequently repaired by injecting epoxy resin into the cracked parts. The arrangement of the CF sheet is shown in Fig.2. Double sheets were bonded crosswise each other over both sides of the beam by epoxy resin adhesive. One more sheet was intentionally arranged on the soffit of the beam to reinforce the tension side of the beam. The bonding work was performed according to the same procedures as presented in Ref.1 and 2. The test was conducted under antisymmetric loading system as shown in Fig.2.

Results, Discussion and Conclusions

Photos 1 shows failure mode of the strengthened beam after the test. The bonded sheets were peeled off by force. All of the specimens failed due to diagonal tension cracking in the central shear area. Table 1 shows the measured values of cracking load P_{cr} and ultimate load P_u of the beam with the central shear span L=250mm. The cracking load P_{cr} was estimated from shear deformation behaviors measured in the central shear span. Specimen A was initially crack-damaged nearby ultimate stage by pre-loading, and subsequently repaired and strengthened. Specimen B was lightly crack-damaged, and strengthened. The results obtained are as follows. The reinforcing effect of the sheet was small for crack initiation. The ultimate shear strength of the strengthened beam increased by about 1.4 times that of the virgin one. The increasing rate of strength was about 1.3 to 1.8 for the beam with L=350mm. Similar reinforcing effects of the sheet were obtained for the crack-damaged beam and the crack-damaged and afterwards repaired beam.

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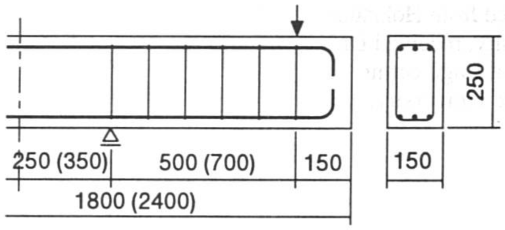
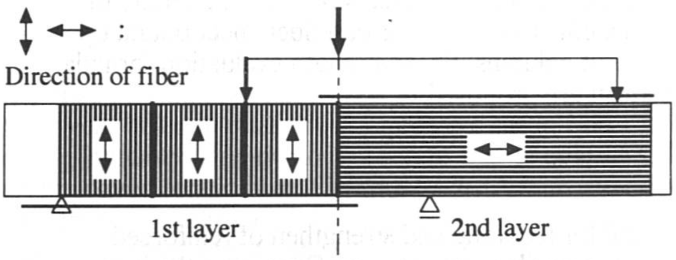


Main reinforcement : upper ; 3-D10 (SD345)
lower ; 3-D10 (SD345)
Stirrup : ϕ 5, @100mm
Concrete : nominal strength = 21MPa

Fig.1 Configuration and bar arrangement of specimen



Carbon fiber sheet (CF sheet) :
T. S. = 3400 MPa, T. M. = 2.3 x 10^5 MPa
Section area of CF = 167 mm^2/m
Adhesive agent : epoxy resin
Crack repairing material : epoxy resin

Fig.2 Arrangement of CF sheets and loading method (antisymmetric load)

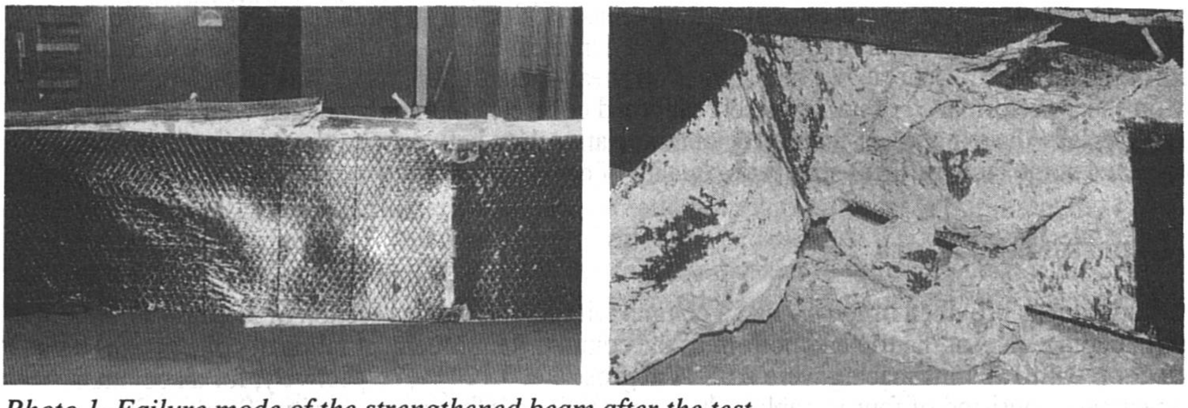


Photo 1 Failure mode of the strengthened beam after the test

Table 1 Test results (central shear span L = 250mm)

Specimen No.	State of Specimen	Pcr (kN)	Pu (kN)
A	virgin	125.5	188.3
	repaired and strengthened	58.8	268.7
B	virgin	103.0	-
	strengthened	73.5	274.6
C	strengthened	132.4	256.9

Pcr : cracking load
Pu : ultimate load

Shear Strengthening of RC Columns by Carbon Fiber Sheet

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Summary

This paper describes an experimental study on the use of carbon fiber sheet for seismic retrofit of non-ductile reinforced concrete columns. Fifteen column specimens designed originally to be failed in shear were tested under some variables which are shear span ratio, axial stress level, quantity of sheet reinforcement and etc. The test results show the effectiveness of carbon fiber sheet bound up column faces for improving the strength and ductility of columns. Shear strength evaluation formula using of an effective coefficient for sheet reinforcement are proposed.

1. Introduction

A carbon fiber sheet was developed as a new material for repairing and strengthen of reinforced concrete members. The sheet is arranged with long carbon fibers in one way. Consequently, it has high strength in one way, and has good workability because of light weight, flexibility and no use of concrete. The sheet is used in real structures for increase of flexural strength of beams, stiffening of slabs with bending cracks, repairing of columns failed in shear and so on. There are many studies of effects on flexural performance, but few studies on shear strength. The paper was an experimental study on shear performance of normal steel reinforced concrete columns bandaged with carbon fiber sheet. Especially the experiment focused to obtain relationships between the shear strength of such columns and some factors: shear span ratio, quality of carbon fiber sheet, axial stress ratio and so on.

2. Experimental Work

Twelve specimens provided originally had a column with a square cross section of 300 mm x 300 mm and two loading stabs at the top and bottom of the column. Reinforcement ratios of column axial bars and of hoop with 150 mm spacing were 2.65 % in gross and 0.124 %, respectively, for all columns. The specimens consisted of four variables: shear span ratio, reinforcement quality of carbon fiber (hereafter CF) sheet, column axial stress and repaired/strengthened. This variable of 'repaired' means a reloading test of the specimens which were repaired with CF sheet after failing in shear as original RC columns (CS2, CM2 and CL2, see Table). Then total number of test units was fifteen.

The arrangement of sheet reinforcement was the way which CF bandages cut out with 30 mm width from the sheet were bound up the column faces in every 50 mm pitch with one or three layers. This striping was to observe easily development of cracking on the columns. Each bandage had a lap joint of about 100 mm length at the ends, and was glued concrete with epoxy adhesive. The properties of CF sheet were thickness of 0.111 mm, tensile strength of 3.55 GPa, Young's modulus of 235 GPa, elongation of 1.5 %. The specimens were subjected to constant axial load in stress ratio of 0, 0.2 or 0.4 and lateral load reversals in drifts angles of 1/500, 1/200, 1/100x2, 1/50x2, 1/33x2, 1/25x2 and 1/20x2. Moment diagram of column was a point symmetry about the column midhight.

Table Summary of test results and analyses note: S=Shear, B=Bond unit: V (kN), R (10^{-3}rad.)

Name of Specimen	Shear span ratio	Axial stress ratio	CF reinf. ratio%	Conc. f_c (MPa)	Experimental results: Stiffness reduct. expVr	expRr	Max. load expVu	expRu	Fail. mode	Calculated results: Shear calVs	Bond calVb	exVu / calV
CS2	1.11	0.2	0.0	28.7	287	2.00	293	2.00	S	303	(245)	0.99
CS2-ARe	1.11	0.2	0.0444	27.2	-	-	297	20.8	S	335	(269)	0.87
CS2-A	1.11	0.2	0.0444	27.2	297	2.90	305	10.4	S	326	(261)	0.92
CS2-3A	1.11	0.2	0.1332	30.7	323	3.50	340	8.92	S	376	(290)	0.87
CS0-3A	1.11	0.0	0.1332	32.9	242	4.20	350	30.0	S	342	(263)	0.98
CS4-3A	1.11	0.4	0.1332	24.0	304	2.87	320	5.29	S	371	(283)	0.83
CM2	1.68	0.2	0.0	29.9	264	6.20	264	6.20	B	(234)	213	1.24
CM2-ARc	1.68	0.2	0.0444	29.9	-	-	170	20.	B	(267)	235	0.83
CM2-A	1.68	0.2	0.0444	29.4	271	5.08	271	5.08	B	(265)	233	1.16
CM2-3A	1.68	0.2	0.1332	26.5	283	6.23	302	20.1	S+B	291	(265)	1.04
CM0-3A	1.68	0.0	0.1332	27.9	208	6.27	292	20.3	S+B	258	(243)	1.13
CM3-3A	1.68	0.3	0.1332	28.3	323	6.80	336	20.1	S+B	320	(288)	1.05
CL2	2.24	0.2	0.0	30.0	239	10.1	239	10.1	B	(199)	194	1.24
CL2-ARo	2.24	0.2	0.0444	30.0	-	-	156	20.1	B	(231)	215	0.73
CL2-A	2.24	0.2	0.0444	23.0	215	9.12	215	9.12	B	(204)	186	1.15

$$calVs = [0.115\, k_p k_u (180+\sigma_B)/(M/Vd+0.12) + 2.7\sqrt{(p_w \cdot \sigma_y + \alpha \cdot {}_f p \cdot {}_f\sigma_u)} + 0.1\, \sigma_0]\, b \cdot j \quad \text{(Eq.1)}$$

$$calVb = \tau_b [0.95 + 0.0018\, \sigma_0 - 0.066\, M/VD]\, n \cdot \phi \cdot d \quad \text{(Eq.2)}$$

$$\text{where } \tau_b = [0.3 + 0.8C/\phi + 13\, \phi\, M/V]\sqrt{\sigma_B} + (a_w \cdot \sigma_y/\chi + \alpha \cdot {}_f a \cdot {}_f\sigma_u/{}_f\chi)/(11 n\, \phi)$$

3. Test Results and Discussion

Response performances of all test units is shown in the table. All columns of S-series (small shear span) were failed in diagonal compressive shear. Shear strength of column CS2-A with one-layer binding increased slightly comparing to that of column CS2 with no sheet, but ductility of the former was improved remarkably. Column CS2-3A with three-layer binding exhibited enhancements of shear strength and ductility. Columns of M-series (middle shear span) with no or one-layer binding were failed in bond split along column axial bars, and those with three-layer binding were failed in the mode mixed with shear and bond split. Columns CM2 and CM2-A failed in bond split had similar maximum strength and deformation at ultimate stage. High reinforcement with CF sheet can improve bond split strength more effectively than shear strength, judging from the fact as the failure mode was changed from the bond split failure to the mixed one. Columns CL2 and CL2-A of L-series (large shear span) showed similar characteristics to CM2 and CM2-A in the strength, ductility and failure mode.

To evaluate the shear strength of columns, Ohno-Arakawa Modified Equation (Eq.1) proposed for RC columns was used. In order to apply this equation to evaluate the shear strength of RC columns bound with CF sheet, the value of fiber reinforcement ratio ${}_f p$ multiplied by its tensile strength ${}_f\sigma_u$ was added in the second term of steel reinforcing effect in the equation. At this time, the use of a reduction factor α of about 2/3 needed to close the calculated values to the experiment values because CF fiber did not reach its tensile strength at the maximum strength of columns. The ratio of expVu/calVs was 0.99 for column CS2 with no sheet and 0.95 in average for columns failed in shear or mixed mode. This means it might be better to use a less value for the reduction factor. As for evaluating the bond split strength of columns, the same manner mentioned above was applied to Shibata-Sakurai Equation (Eq.2). The ratio of expVu/calVb was 1.24 and 1.25 for CM2 and CL2 with no sheet, respectively, and was 1.19 in average for columns failed in bond split or mixed mode. This means the reduction factor of 2/3 is almost an adequate value for evaluation of bond split strength.

4. Conclusion

Lateral load reversal tests of non-ductile R/C columns exhibited the following performances:

1) reinforcement by binding of one-layer carbon fiber sheet around column faces can improve the ductility only, 2) three-layer binding is required to increase not only the ductility but also the diagonal shear and bond split strengths, 3) effectiveness of carbon fiber sheet binding on shear strength decreases according to column axial stress level, and 4) the use of reduction factor for carbon fiber reinforcement is needed to evaluate the shear and bond split strengths.

Seismic Retrofit of Concrete Columns Using Advanced Composite Materials

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Summary

In order to develop design guidelines for seismic retrofit of existing bridge columns using prefabricated composite jackets, six large scale bridge columns (Dia.=610mm) have been tested. Three columns with lap-spliced longitudinal bars were subjected to cyclic lateral forces in a single bending mode, while the other three columns were tested to simulate shear dominated bridge columns under cyclic loading with double bending. Test results demonstrated the superior retrofit effectiveness of the composite jackets.

Test of Shear Dominated Columns

The reinforcement details of shear test model columns are shown in Fig.1, and the test setup is shown in Fig.2. One column was tested in the "as built" condition and the other two models were tested after retrofit for the full column height using prefabricated composite jackets. Fig.3 shows the installation of the individual cylindrical prefabricated composite jacket. The "as built" column suffered a sudden shear failure during the loading cycle corresponding to a limited ductility factor of 3.0. The retrofitted columns developed significantly improved seismic performance characterized by large energy absorption capacities and stable hysteretic responses up to displacement ductility factors of 12 to 14, as shown in Fig.4.

Flexural Column Testing Program

Three circular columns with lap-spliced longitudinal reinforcement were tested under constant axial load and cyclic lateral forces in a single-curvature mode. One column was tested in the "as built" condition and others were tested after retrofit for the potential hinge region using prefabricated composite jackets. The "as built" model column failed without developing its predicted flexural capacity due to severe deterioration in the lap-spliced longitudinal bars. The two retrofitted columns, one retrofitted with 5-layer individual prefabricated composite jackets and the other with 5-layer continuous jackets developed significantly improved seismic performance demonstrating the excellent effectiveness of prefabricated composite jackets for flexural retrofit. The retrofitted columns exhibited stable behaviors until a displacement ductility factor of 8.0. Although a gradual degradation of load carrying capacity was observed during loading cycles corresponding to large deformation, the two retrofitted columns were able to develop an ultimate displacement ductility factor of 10.0.

Concluding Remarks

Current study provides methods for seismic retrofit and repair of reinforced concrete columns. Prefabricated composite jacketing can also be considered for repair and protection of column supported structures from environmental damage and exposure. Studies will be initiated in near future to address the applications of prefabricated composite jacketing in: columns subjected to freeze/thaw and deicing salt corrosion in snow belt states and countries; structures deteriorated due to water absorption of porous concrete; harbor wharf and pilings; power/telephone poles; structures damaged by fires, etc.

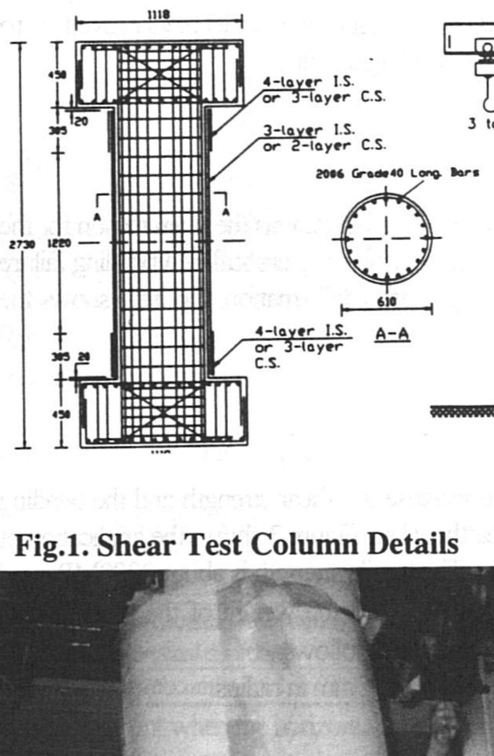

Fig.1. Shear Test Column Details

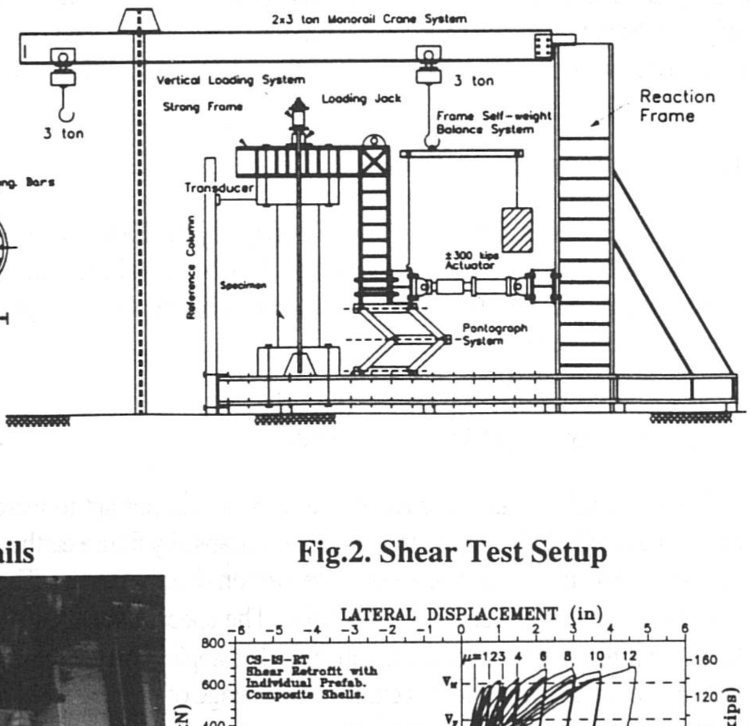

Fig.2. Shear Test Setup

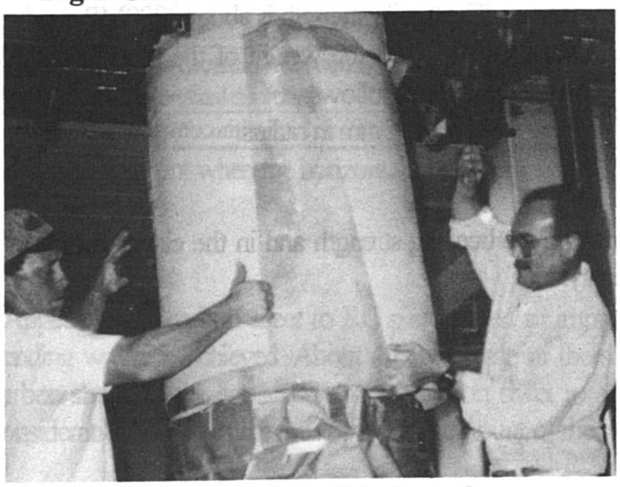

Fig.3. Installation of Prefabricated Composite Jackets

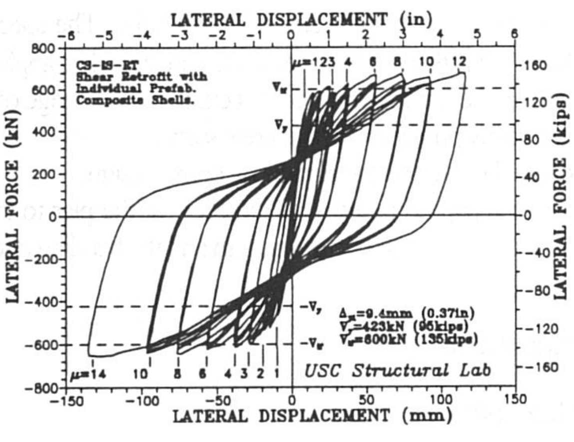

Fig.4. Hysteretic Response of Shear Column with Individual Shell Retrofit

Strengthening of Reinforced Concrete Bridge Piers by Carbon Fiber Sheet

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Summary

The 1995 Great Hanshin Earthquake destroyed many concrete bridge piers as well as the other structures. Lack of shear capacity is considered to be the main reason for the collapse. The piers with a proper capacity in bending ductility were escaped from collapse although they were severely damaged by bending. Strength improvement of the existing concrete bridge piers using carbon fiber sheet is proposed . This method is relatively easy to apply since it is only necessary to glue the sheet on the surface of a pier. The load bearing test of the model piers proved 20 to 40% improvement in the shear strength and 2 times improvement in the bending ductility.

1. Damage to RC piers

A lot of RC single piers suffered very severe damage by the Earthquake. Shear failure was the main reason for the collapse. Figure 1 shows the typical shear failure. On the other hand, relatively higher piers suffered bending failure but escaped from collapse although the damage was severe with a large plastic deformation. Figure 2 shows the example of bending failure.

2. Strength improvement by carbon sheet

According to the failure pattern of RC single piers, it is important to increase the shear strength and the bending ductility of the existing bridge piers to avoid their collapse by future earthquakes. Figure 3 shows the application of carbon sheet to RC piers. The thickness of the carbon sheet is 0.1mm. The tensile strength is about 2800MPa and the modulus of elasticity is about 2.5×10^5MPa. The specific gravity is 1.8, then the weight of the sheet is only $0.18kg/m^2$. It is therefore very easy for handling. The application procedure is as follows;

1. Clean up the concrete surface and cut the corner edge of piers by more than 30mm in radius.
2. Paint epoxy primer on the concrete surface.
3. Adhere the sheet on the surface by epoxy resign.

The sheet is applied in the axial direction of the pier to improve the bending strength and in the circumferential direction to improve the shear strength and the bending ductility.

3. Experiment

3.1 Shear test

A pier model with the column of 119cm high and 60cm square was employed for the shear test. Horizontal force was applied at the top of the column in the back and forth direction under the constant axial force of 539kN. The shear span ratio is 2.5. Five specimens were tested, S-1 being without carbon sheet, S-2 with 1 layer, S-3 with 2 layers, S-4 with 5 layers of carbon sheet and S-5 with steel plate of 3.2mm thick. Figure 4 shows the test results.

According to these results, application of 2 layers of carbon sheet improved the shear strength of the test pier by 40%, while steel plate improved the shear strength by 64%.

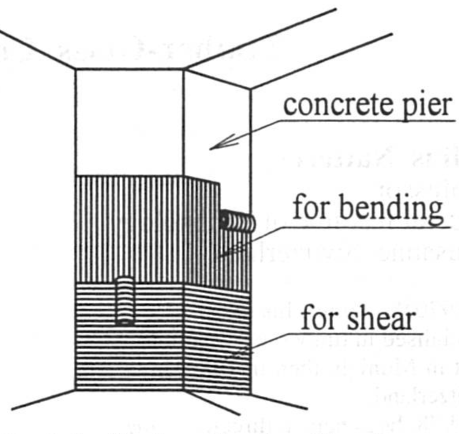

Fig. 3 Application of carbon fiber sheet

3.2 Bending test

A pier model with the column of 254cm high and 60cm square was employed for the bending test. The shear span ratio is 5.0. Five specimens were tested. Table 1 shows the summary of the test specimens and test results. These results indicate that reinforcement by the carbon fiber sheet improved the ductility satisfactorily.

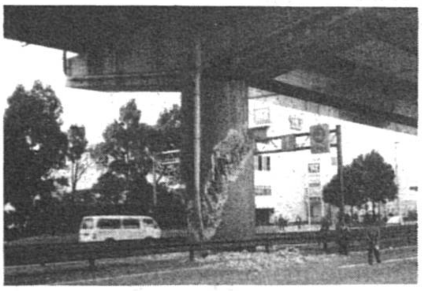

Fig. 1 Shear failure of a RC pier

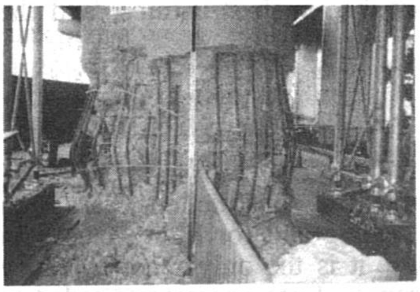

Fig. 2 Bending failure of a RC pier

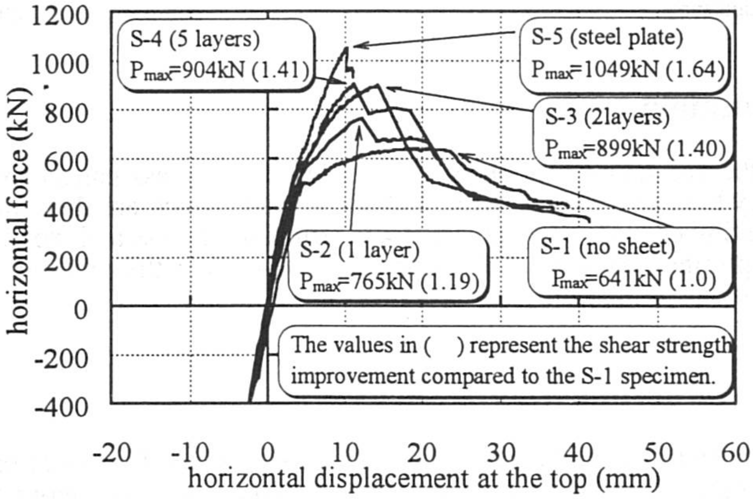

Fig. 4 Load-deflection curve of the shear test model

Table 1 Bending test results

Specimen	Reinforcement	Maximum horizontal force (kN)	Yield displacement δy (mm)	Ultimate displacement* δu(mm)	ductility δu/δy
M-1	without carbon sheet	206.8	15.6	87.9	5.6
M-2	2 layers(circumferential direction)	203.8	13.6	98.8	7.3
M-3	4 layers(circumferential direction)	211.7	13.9	107.7	7.7
M-4	4 layers(circumferential direction) +1 layer(axial direction)	228.3	12.2	96.8	7.9
M-5	8 layers(circumferential direction)	225.4	13.1	144.8	11.1

* displacement when the horizontal force dropped to 80% of the maximum horizontal force

Conclusion

Application of carbon sheet to RC pier proved to improve the shear strength. Improvement of the ductility in bending was also achieved. About 40% increase of the shear strength was obtained by applying 2 layers of the carbon sheet. In the application of the carbon sheet to an actual pier, the size of the pier should be taken into consideration to determine the appropriate amount of the sheet to obtain the required strength improvement.

Acknowledgment

The authors wish to acknowledge their indebtedness to Mr. S. Sakanishi of Sunkit Co.,Ltd., Mr. K. Miyata of Nippon Oil Co.,Ltd. and Mr. T. Matsuo of Konisi Co., Ltd. for their cooperation in the experimental work.

Timber-Glass Composite in Structural Glazing

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Summary

In the normal application of glass in buildings, glass panels are mounted in frames and designed to carry their self-weight, wind and snow load, which are then transmitted to the supporting structure. The timber-glass concept is a mixed structure of glass and wood, with both components acting as the supporting structure. This paper aims to present the problems related to such structures, and the solutions proposed by our research.

Introduction

More and more architects and engineers are looking for an ecological way of construction. Glass is an ecological material in the sense that we can economise energy for heating and for light, if used in construction. In the same way wood is an ecological material because it is the only renewable raw material that stores carbondioxide which is necessary for wood growth. Less energy is needed to fabricate a timber beam compared to metal, aluminium or concrete. Therefore, timber and glass are often used in combination for construction like for greenhouses or facades.
In the traditional (not composite) timber and glass construction, the beam-sections are often so important, that it causes problems with aesthetic and natural lighting. So a timber-glass composite element has been developed, where the wooden frame is directly glued to the glass-plate (fig.I).

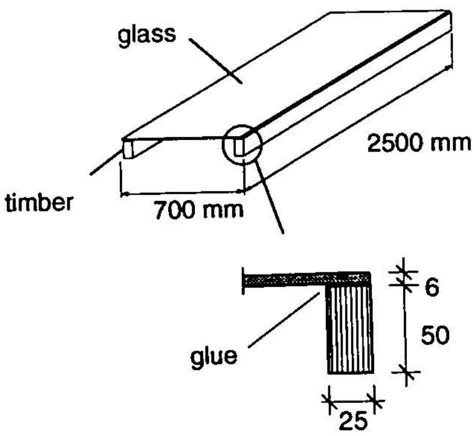

fig.I: composite element timber-glass

A comparison between the traditional and the composite structure shows that the wooden sections are much smaller with timber-glass composite structures (fig.II, next page).

This kind of composite structure has the following advantages:

- prefabricated sections
- minimal cold bridges
- optimised energetic profit
- light structure
- ecological
- economical

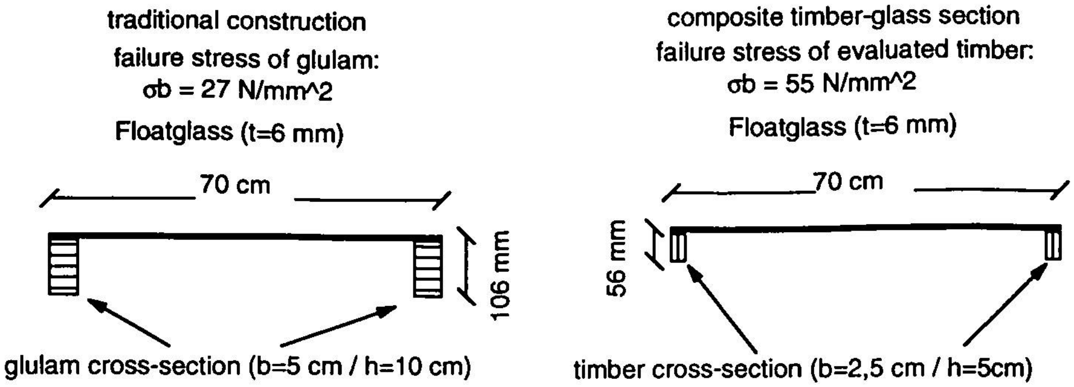

fig.II: comparison between the traditional and the composite structure

Tests

To find the right glue for the timber-glass composite structure, several small specimens have been tested under shear compression loading. Because the composite elements are under climatic variations the test specimens were submitted to several cycles. One cycle consisted of 4 hours at -30°C, 4 hours at +70°C, 16 hours at +30°C and 80% humidity. This cycle was repeated up to seven times and specimens were tested after 0-cycle, 2-cycles, 5-cycles and 7-cycles to evaluate the possible degradation. Of the four glues which were tested, only one resisted to the stress-test.
After the right glue was found, 4-point loading tests on timber-glass composite plates were executed. The results of these tests showed that the efficiency of the glueing was nearly 100%. The rupture of these elements was due to the excess of the tensile-bending stress in the wood (in average: 66,24 N/mm^2).

Conclusion

The goal of this research is to develop composite timber-glass elements for structural glazing. A glue has been found and tested by shear compression loading after several climatic cycles. It was proven, that composite elements which are loaded perpendicular to its plane has a high rigidity and resistance. The research-team of timber-glass composite for structural glazing is now testing composite elements as shear-walls to stabilise greenhouses and facades. Also composite timber-glass beams will be tested.

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Advanced Composite Stay Cables

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Summary

The paper summarizes the ongoing research at the University of California in San Diego about the performance of composite stay cables available on the world market. Research emphasis is placed on investigating the long term behavior of such cable systems, in particular of their anchorages. Five cable systems offered on the world market were selected for the testing; three were composed of unidirectional carbon and two of aramid fiber reinforced polymer. The selected cable systems were subjected to short- and long-term testing.

Introduction

The infrastructure industry is in the search for more durable materials for key structural elements. Advanced composite materials like carbon or aramid fiber reinforced plastics stand out for their light weight, excellent performance, durability and chemical resistance and seem to be a good addition to conventional construction materials. Stay cables made of these advanced composites are promising, and an increasing number of products is appearing on the world market. However, a lot of investigation, in particular of the long term behavior, has to be done to gain the necessary confidence into the structural reliability of these new cable systems. In a research program at the University of California in San Diego, available cable systems from the world market were subject to long and short term tests to investigate the behavior of such systems. The results provide short term characteristics and lead to an estimate of their long term behavior. The program is not completed yet. Initial and preliminary results and trends are discussed in the following chapters.

Cable systems

Scanning the world market for commercially available cable systems, most of the systems are found in Japan and Europe. Only a few companies offer cables with appropriate anchorage systems which could fulfill the requirements for stay cables.

Two different types of anchorages can be identified. The first one has tendons potted in polymer matrix and the load is introduced by bond forces between the matrix and the composite tendons. The second type is similar to conventional high strength steel tendon anchorages and anchors the tendons mechanically by wedges and the force is introduced by friction. There are also systems using a combination of mechanical and bonded anchors.

Short Term Test

Three specimen of each of the cable systems were subject to short term load tests. The procedure of the test was similar to those recommended by the Post-Tensioning Institute (PTI) and the Federation Internationale de la Precontrainte (FIP) for steel tendons. However, the recommendations were adjusted for composite cables.

Even though only three specimen per tendon type were tested, the repeatability of the test results was very satisfactory. Cable systems which failed due to slippage in the anchorage had a deviation of 8 % of the average failure load and the other systems only 4 %. The deviation is hereby defined as the difference between the highest and smallest failure load. However, the tested failure load was sometimes significantly higher than the manufacturers specified guaranteed breaking load.

Performance of the bond anchorages

Strain gages at the outer shell of the bond anchors allowed the estimation of the tendon force transfer within the anchorages. The bond stresses between the strand and the matrix were roughly estimated by evaluating the strains at the surface of the sleeve. During cyclic loading, high bond stresses of up to 20 MPa were developed. With each cycle, the peak stress was moving slightly towards the back of the anchorage, and the bond stress concentration at the front of the anchorage was decreasing. During loading to failure, the peak of the bond stress distribution was moving to the back of the anchorage, developing up to 44 MPa of maximum average bond stress before tendon failure. This observation was independent of the failure mode in the cable systems.

Long Term Test

Two specimens of each of the selected cable systems were mounted in separate steel frames for long time monitoring. The tendons were tensioned and then anchored at their specified maximum service load of 65 % and 55 % of their nominal breaking load for carbon and aramid based systems, respectively. This was the same load level as of the upper load during cyclic loading in the previous tests. After about 1000 hours, they were restressed to the service load. The observed initial relaxation is generally high. The bond anchors reveal a similar behavior as during cyclic loading. The peak average bond stresses tend to move more to the back of the anchors.

Conclusions

All cable systems showed a very good performance during the short term tests with a high repeatability. The behavior of the cable systems is different for the two load cases in short term and long term tests. Therefore, the long term behavior cannot be predicted satisfactorily by short term testing. This is true in particular for the bond anchor systems, because of the viscous behavior of the matrix. As a third test for the estimation of the long term behavior of composite cables it is recommended to perform accelerated aging tests of the anchors under sustained load.