Seismic Behavior of RC Frame Structures with Beams Retrofitted by Externally Bonded FRP Sheets

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After the 1995 Hyogoken Nanbu earthquake, many RC frames in Japan were strengthened in the column portion to prevent undesirable shear failure. It was, however, found that such frames are still vulnerable due to the possibility of shear failure at beams. In this research an experimental investigation was conducted to investigate the seismic behavior of RC frames with beams strengthened by externally bonded FRP sheets. Results showed that, by increasing shear strength of beams, the seismic performance of the frame improves and the degree of enhancement depends on the wrap configuration of the FRP sheets. This paper also proposes an equation to evaluate the bond strength of the interface between FRP sheet and concrete depending on the bonded anchorage of the FRP sheets. The relation was then used in finite element analysis to numerically simulate the behavior of strengthened frames.

Key Words: FRP, RC frame, seismic performance, shear strengthening

1. Introduction

Many highway structures have been designed and constructed prior to the implementation of modern seismic design codes. Many bridges that collapsed in the 1995 Hyogoken Nanbu earthquake were designed before the introduction of 1980 seismic resistant design codes[1]. Based on investigations[2] different preventive actions were proposed[3,4] to identify the deficient bridges and to apply suitable strengthening techniques to overcome any unsatisfactory performance. Immediate action to retrofit over 6000 columns of highway bridges took place by incorporating steel or concrete jacketing[5,6]. It was, however, found that other critical members of some frames such as beams may be incapable of resisting similar seismic loads to those which the strengthened columns may sustain. The excessive damage of those members may limit the overall rotational and displacement ductility capabilities of the highway frames during future earthquakes[7]. This underscores the need of strengthening the beams of formerly strengthened frames. In the paper the effectiveness of strengthening the beams by externally bonded Fiber Reinforced Polymer (FRP) sheets are presented.

The past study conducted by several researchers[8-12)] shows that externally bonded FRP sheets can be used to enhance the shear performance of RC beams. It is, however, evident that the optimal utilization of FRP material strength could hardly be achieved due to debonding of sheets from concrete surface. The average effectiveness ratio of FRP sheet, which is ratio between FRP strain at failure to the ultimate tensile strain of FRP sheet, is limited to a value approximately equal to 0.15 to 0.25[13)]. It clearly indicates the importance of the bond between the FRP to concrete interface for the effectiveness of externally bonded FRP strengthening technique. Therefore, the main objectives of this study are to study the effectiveness of strengthening the beams of RC frames by externally bonded FRP sheets and to develop bond strength models to incorporate the effect of bonded anchorage length of FRP sheets on the shear behavior of RC frames.

2. Experimental Program

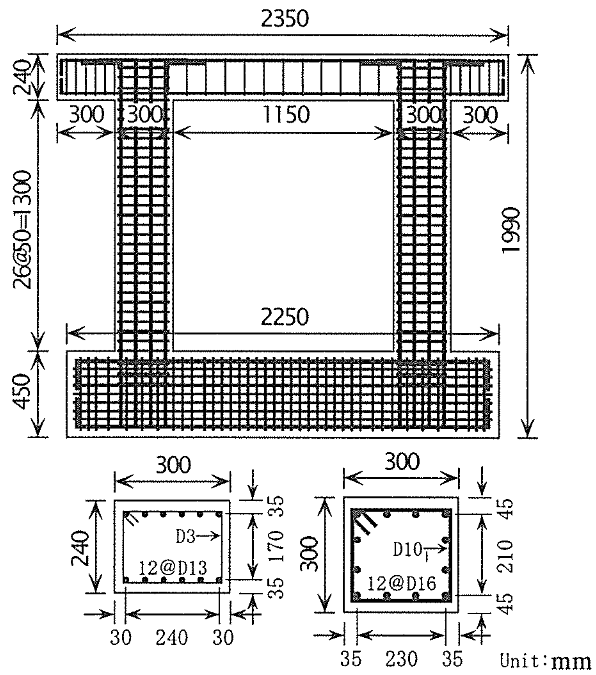
Experiments on a total number of five RC frame specimens are presented in this paper.

2.1 specimen details

Experiments were carried out on small-scaled one story RC frame specimens. The specimens were 1/7 scaled model of the prototype highway bridge frames. All specimens had same dimensions and reinforcement details. The dimension of footing was 450 x 600 x 2250 mm. Column dimensions were 300 x 300x 1300 mm, while that of the beam was 300 x 240 x 2350 mm. Fig. 1 shows the details of the frame specimen. The columns were designed to their full strength with high percentage of shear and longitudinal reinforcements in order to represent the formerly retrofitted columns.

Table 1 Details of test specimens

Sp. ID	Concrete Strength MPa	FRP Type	FRP Bonding Type	Anchorage Length mm
SP-1	35.18	-	0.05 % steel	Control frame
C-1	36.44	CFRP	U-warp	-
C-2	28.81	CFRP	bonded length	60
A-1	37.16	AFRP	bonded length	60
A-2	35.22	AFRP	bonded length	200, Full warp



Beam portion Column portion

Fig.1 Details of test specimens

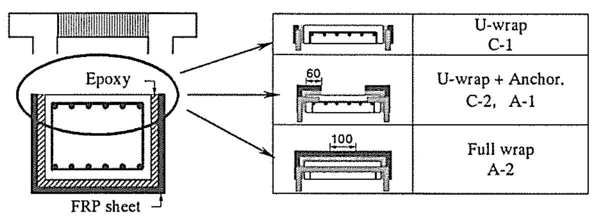


Fig.2 FRP wrap configuration

Specimen SP-1 was a control specimen in which the beam was unstrengthened. Beam portion of other specimens was strengthened by externally bonded FRP sheets with various bonded anchorage lengths of FRP sheets. Experimental variables and FRP anchorage schemes of strengthened frame specimens are presented in Table 1. The wrap configurations of FRP sheets in the strengthened specimens are illustrated in Fig. 2. The beams of the specimens C-1 and C-2 were strengthened by externally bonded Carbon Fiber Reinforced Polymer (CFRP) sheets with U-wrap and additional 60 mm bonded anchorage configurations respectively. While, the beams of the specimens A-1 and A-2 were strengthened by externally bonded Aramid Fiber Reinforced Polymer (AFRP) sheets with 60 mm bonded anchorage and full-warping configuration respectively.

2.2 materials

Ready-mixed normal weight rapid hardening concrete was used. The maximum size of the coarse aggregate was 20 mm and the average slump was 140 mm. Concrete strengths of all the specimens at their respective days of loading test are shown in Table 1. Mechanical properties of FRP sheets and steel reinforcements used in the experiments are presented in Table 2 and Table 3 respectively.

Table 2 Mechanical properties of FRP sheets

FRP Composites	Design Thickness mm	Tensile Strength MPa	Elastic Modulus GPa	Ultimate Elongation (%)
CFRP for C-1	0.167	4160	245	1.5
CFRP for C-2	0.167	3400	230	1.5
AFRP (AK60)	0.286	2000	120	1.8

Table 3 Mechanical properties of reinforcing bars

Bar type	Yield strength MPa	Elastic Modulus GPa
D3	292.3	191.1
D10	402.3	187.8
D13	375.4	167.6
D16	403.1	196.6

2.3 instrumentation and loading

Each specimen was tested under reversed cyclic loading with the experimental setup as shown in Fig. 3 by using an actuator of 500 kN capacity and a maximum stroke of 200 mm. Additionally, a vertical load of 106 kN was applied on the top of the beam at its mid span. Each specimen was subjected to pre-determined displacement excursions. The first displacement amplitude was 2.5 mm which was followed by an amplitude of 5 mm. The specimens were then subjected to sequentially increasing displacement amplitude of the integer multiple of yield displacement. Yield displacement is defined as the lateral displacement corresponding to the first yielding of longitudinal reinforcement of column. Three repetitions of each cycle were conducted during testing. The ultimate displacement was defined as displacement corresponding to the reduction of the maximum load to 80%.

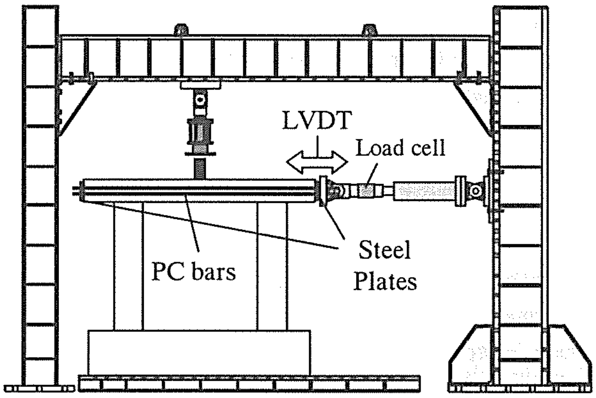


Fig.3 Test setup

The specimens were instrumented by an array of strain gages attached on longitudinal bars and shear reinforcements of both beam and columns, and on the surface of externally bonded FRP sheets. Fig. 4 shows the arrangement of strain gages on FRP sheets.

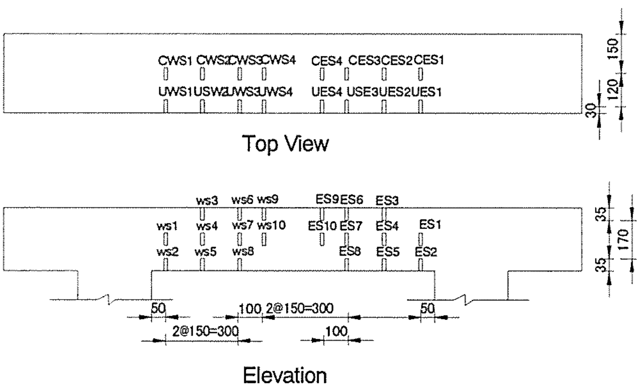


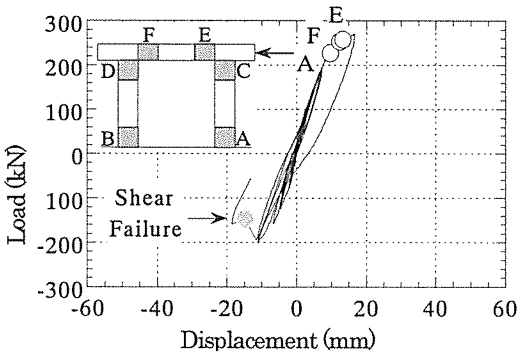
Fig. 4 Location of strain gage for FPR sheet

3. Experimental Results

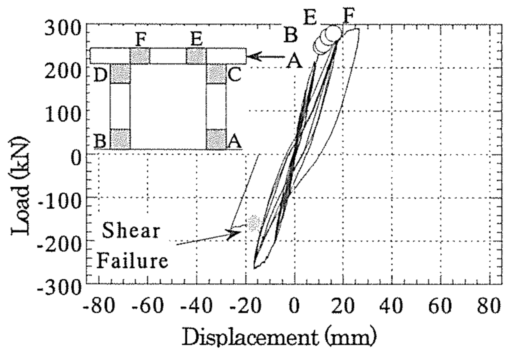
Experimental results of each specimen are presented in terms of hysteretic load-displacement relationship at the loading point as shown in Fig. 5. Fig. 6 illustrates the crack patterns of specimens SP-1 and A-2 at the ultimate state.

In the control specimen SP-1 yielding of longitudinal bar first occurred at the column near the column-footing joint. Yielding of the bars then occurred at either ends of the beam. Finally, diagonal shear crack appeared on the beam at the displacement of 11.5 mm. The load carrying capacity of the frame then suddenly dropped due to the shear failure. In the specimen C-1 in which the beam was strengthened by attaching CFRP sheets to three faces of the beam, showed a better load-displacement behavior compared to the control specimen. Debonding of the sheet started to occur at the displacement of 16mm. The final failure occurred due to occurrence of shear failure followed by the debonding of FRP sheet.

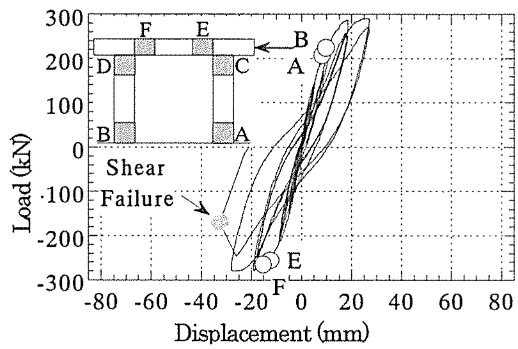
In specimen C-2, in which an extra bonded anchorage was provided, slight debonding occurred at the displacement of 18 mm. Since there was an extra bonded anchorage, the frame continued to carry further load. At the displacement of 26 mm the splitting of concrete occurred and the frame failed due to the shear failure at beam. Similar results were obtained for specimen A-1 strengthened with AFRP sheets of similar wrapping configuration. First sheet debonding occurred at the displacement of 18 mm while the concrete splitting occurred at the displacement of 23.9 mm. Final failure was due to the shear failure of beam.



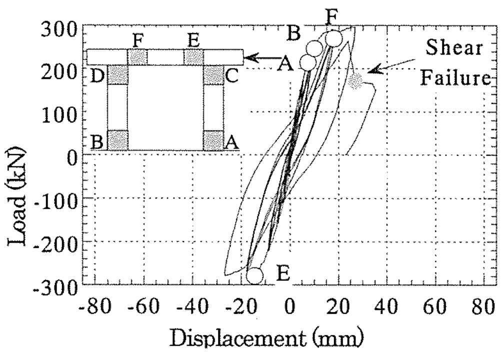
(a) Specimen SP-1



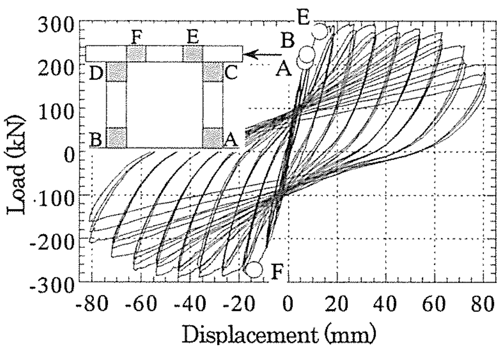
(b) Specimen C-1



(c) Specimen C-2

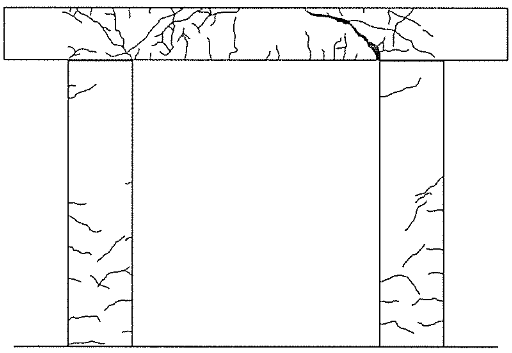


(d) Specimen A-1

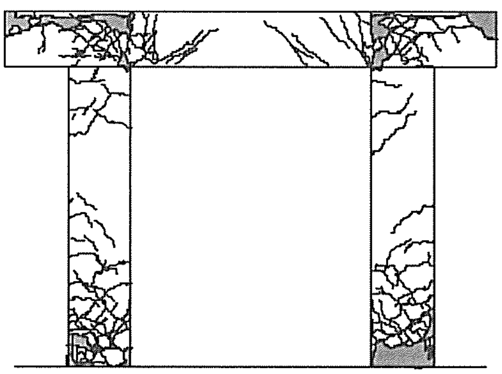


(e) Specimen A-2

Fig.5 Load-displacement curves of all the tested specimens



(a) SP-1



(b) A-2

Fig. 6 Crack pattern at failure

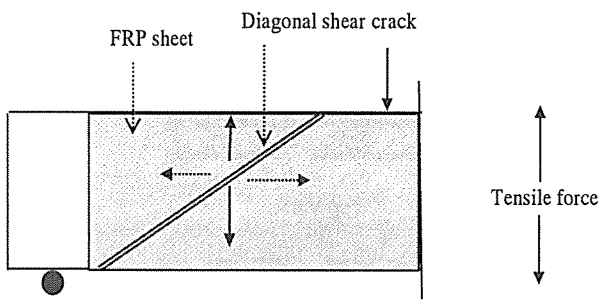
In Specimen A-2 in which full wrapping was provided, slight debonding of sheets started as early as the displacement of 9mm. Since there was a better anchorage as compared to previous specimens, sudden shear failure did not take place. The failure mode of this frame completely changed from shear to flexure.

4. Analysis of Shear Strengthened Frame

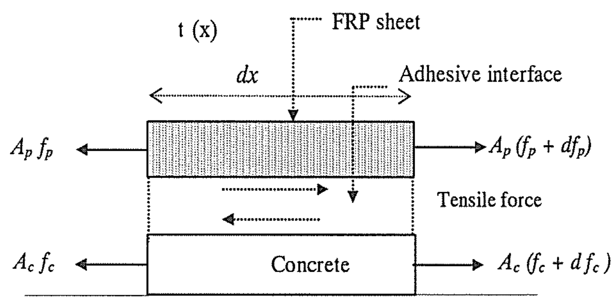
The influence of bonded anchorage on shear behavior of RC structures was duly studied based on the experimental results. The failure modes of shear strengthened RC frame was found to change from FRP sheet debonding to concrete splitting or flexure depending on the length of bonded anchorage.

4.1 shear stress at FRP to concrete bonded joints

The contribution of the FRP sheets on shear strength can be modeled by the bond strength approach in which the bond stress is determined at the interface between FRP sheet and concrete surface. When the diagonal shear cracks are formed in shear strengthened RC beam wrapped with FRP sheets, interface bond stress between FRP sheets and concrete develops as a result of vertical separation of rigid bodies of concrete on the either side of the diagonal cracks. Figure 7 (a) shows the FRP strengthened RC beam with tensile forces developed in the FRP sheet after the initiation of diagonal shear crack.



(a) Tensile force developed on FRP



(b) Bond stress on adhesive interface

Fig. 7 Stress transfer between concrete and FRP

Tensile stresses induced in FRP sheet are then transferred to the concrete on each side of the crack by interfacial bond stresses. If this interfacial bond stress is compromised before the rupture of the FRP sheets, debonding failure occurs. The interfacial shear stress can be analytically evaluated by considering the equilibrium condition of the bonded joints as shown in Figure 7(b) as follows.

$$\tau_b = E_p t_p \frac{\Delta \varepsilon_p}{dx} \quad (1)$$

Where,

τ_b = average bond stress
t_p = thickness of FRP sheet
E_p = modulus of elasticity of FRP sheet
ε_p = strain in FRP sheet
w_p = effective width of FRP sheet

Once the average interface bond strength is evaluated, the ultimate force carried by the FRP sheets can be calculated by using the following relation.

$$\tau_b = \frac{P_{\max}}{A_p} \quad (2)$$

Where, τ_b is the average interface bond strength, P_{max} is the ultimate load carried out by FRP sheet and Ap is the effective bonded area of FRP sheet. Therefore, from the measured strain profiles along the bonded joints, the shear stress distribution along with the bonded joints can be obtained. One of the examples of strain profiles for frame C1 are presented in Figure 8. The free end of the FRP sheet is considered as the reference.

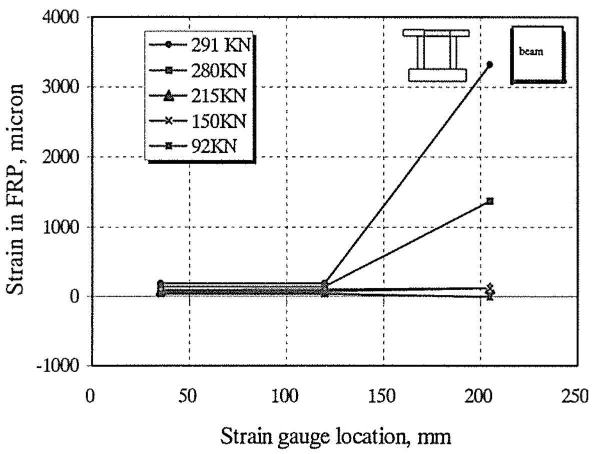


Fig. 8 Strain profile of specimen C-1

4.2 bond strength model for bonded joints

The experimental interface bond stress at the ultimate load of the strengthened RC frames was calculated. The strain profiles used for the calculation of the interface bond stress at the ultimate failure load of the specimens are shown in Figures 9. For the specimens, which failed in shear with sheet debonding, the interface shear stress at ultimate failure can be taken as bond strength. Interface bond stress however decreases with the increase of bonded anchorage length. Due to the affect of bonded anchorage, interface bond stress could not reach the critical bond strength (τ_b) at joints prior to concrete splitting and thereby debonding of FRP sheets is prevented.

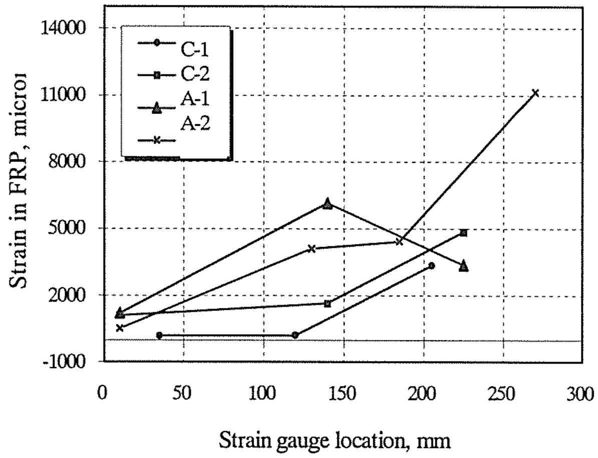


Fig. 9 Strain profile at ultimate condition

To establish the relation incorporating the effect of bonded anchorage length on the interface bond stress, a non-dimensional parameter (ß) is introduced, which is the ratio of bonded anchorage length to the width of beam section. Analysis revealed that, bond stress is related to the stiffness of the FRP sheets, the bonded anchorage length and concrete strength. Since the interface bond stresses calculated by using Equation 1 were with different concrete strengths of the test specimens, those bond stresses were normalized with the concrete strength of the specimens of C-1 and A-1 for the CFRP and AFRP series respectively.

The curves of the normalized bond stress and the dimensionless parameter (β) were plotted for the CFRP and AFRP series separately. The reduction in bond stress with the increase in the bonded anchorage can be represented by polynomial curve of second degree as shown in Figure 10. The relation obtained was then modified by taking into account the compressive strength of concrete.

For CFRP sheets

$$\tau_b = \left(0.103 E_p t_p - 0.287\beta + 0.013\beta^2\right)\left(\frac{f_c'}{37.2}\right)^{2/3} \quad (3)$$

For AFRP sheets

$$\tau_b = \left(0.114 E_p t_p - 0.383\beta + 0.017\beta^2\right)\left(\frac{f_c'}{39.6}\right)^{2/3} \quad (4)$$

Where,

dimension less parameter $\beta = 20l_a/b_w$
E_p: modulus of elasticity of FRP laminates, GPa
t_p : thickness of FRP, mm
l_a : bonded anchorage length, mm
b_w : width of concrete beam, mm
f_c' : compressive strength of concrete, MPa

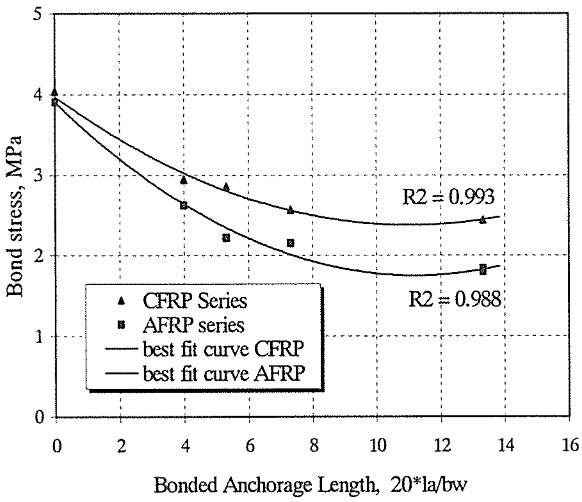


Fig. 10 Reduction of bond stress with bond anchorage length

5. Finite Element Analysis

A 2D finite element analysis[15] was carried out to study the behavior of RC frames with beam wrapped by FRP sheets. Since 2-D analysis is unable to model the three-dimensional affect of bonded anchorage length of FRP sheets, new debonding failure criteria for adhesive interface elements have to be incorporated. In the analysis, RC elements were modeled by eight-node plain stress element and FRP sheets were modeled by eight-node orthotropic elements. Adhesive interface between FRP sheets and concrete was modeled by using the sixteen-node adhesive interface elements. Fig. 11 shows the details of the finite element model.

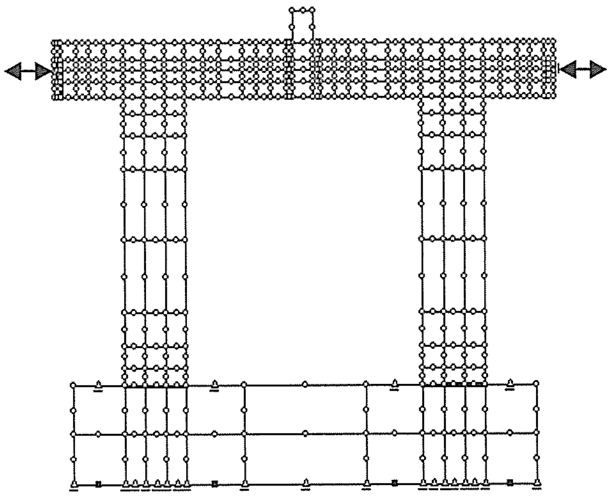


Fig. 11 Finite element model of frame

It was found that, bond stress at the FRP-concrete interface is reduced with increase in bonded anchorage length of FRP sheets. Since the specimens with bonded anchorage did not fail due to debonding, the bond strength could not be determined. Hence the idea of reverse computation was utilized. The rate of reduction of bond stress was considered as the rate of increase in bond strength. Therefore, bond strength model for the adhesive interface is obtained by adding the reduction trend of bond stress as the increase in bond strength due to bonded anchorage length of FRP sheets as given in Equation 5 and Equation 6 for the CFRP and AFRP materials respectively.

For CFRP sheets

$$\tau_b = \left(0.103E_p t_p + 0.287\beta - 0.013\beta^2\right)\left(\frac{f_c'}{37.2}\right)^{2/3} \quad (5)$$

For AFRP sheets

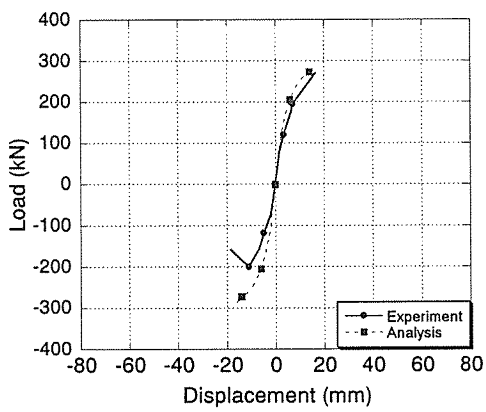
$$\tau_b = \left(0.114E_p t_p + 0.383\beta - 0.017\beta^2\right)\left(\frac{f_c'}{39.6}\right)^{2/3} \quad (6)$$

5.2 analytical results and comparison with experiment

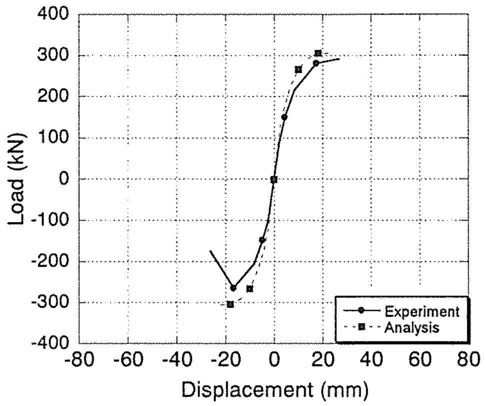
Analytical results of all the specimens are presented in Fig. 12 in terms of monotonic load-displacement behavior and are compared with the experimental load-displacement envelope curve. Analytical results for specimen SP-1 showed shear failure at the maximum load of 287 kN, which is close to that obtained in the experiment.

With the application of externally bonded FRP sheets in specimen C-1 with U-warp, load carrying capacity increased due to the delayed shear failure of beam. Deformability of frame further increased by providing extra bonded anchorage of 60 mm in specimens C-2 and A-1. Final failure was, however, of shear mode followed by the bond failure of the interface.

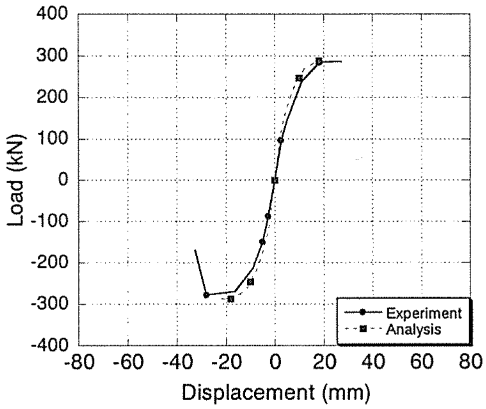
In the specimen A-2 with full wrap, the failure mode was changed from undesirable shear to a ductile flexural one.



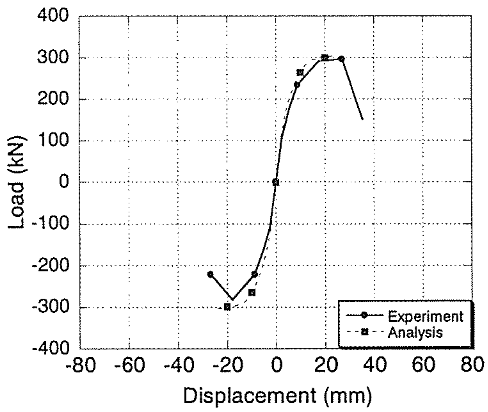
(a) Specimen SP-1



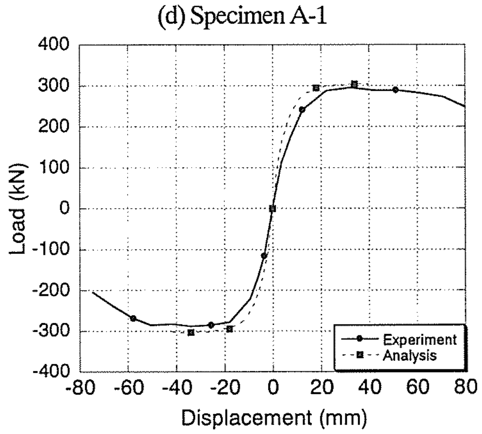
(b) Specimen C-1



(c) Specimen C-2



(d) Specimen A-1



(e) Specimen A-2

Fig.12 Comparison of analytical and experimental results

6. Conclusion

An investigation on shear strengthening of the beam portion of one story RC frame by externally bonded FRP sheets were carried out. Based on the study following conclusions are drawn:

1. The retrofitted columns have adequate rotational capabilities but the beam suffered the damage due to shear failure.
2. Strengthening with Both CFRP and AFRP are effective in the overall enhancement of the performance of RC frame while providing the bonded anchorage can further improve the performance of the frame by preventing the debonding failure of FRP sheets.
3. The proposed debonding model is effective in predicting the behavior of RC frames strengthened with FRP sheets of various configurations.

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