Load coefficient for steel re-bar's yield strength in the bending moment capacity calculation of CFS-reinforced RC Beams

Nihon University JSCE Member Ming-Chien HSU
Nihon University JSCE Member Tetsukazu KIDA
Nihon University JSCE Member Tadashi ABE
Nihon University JSCE Member Toshiaki SAWANO
Nihon University JSCE Member Yoshitaka OZAWA

1. INTRODUCTION

From the pervious studies [1], authors had found a load coefficient for the yield strength of steel re-bars and calculated the bending moment capacity of undamaged RC beams. Later, authors used this load coefficient to calculated CFS-reinforced RC beams with a coefficient of the reinforcing effect of CFS. In this study, authors used the CFS-reinforced with two different depth type of RC beams and calculated the maximum flexural load-carrying capacity of CFS-reinforced RC beams with two different bending moment capacity equations that are: ① modified bending moment from the test and ② bending moment from the CEB-FIP [2]. Moreover, authors had added load coefficient into the CEB-FIP calculation. At the end, the theoretical maximum flexural load-carrying capacities were compared with the test results.

2. PREPARATION OF TEST SPECIMENS

2.1 Materials used for Test Specimens: The test specimens were produced using ordinary Portland cement, coarse aggregates with a maximum size of 20mm (Compressive strength are 38.5N/mm^2 and 41.5N/mm^2 for Type Ⅰ and Type Ⅱ), and D16 steel re-bars of the SD 295A class (Yield and Tensile strength are 368N/mm^2 and 568N/mm^2). High-strength continuous CFS with a unit weight of 202g/m^2, a tensile strength of 4,420N/mm^2, a thickness of 0.111mm, and a width of 30cm were used as the reinforcing material to be placed on the bottom of each specimen. Epoxy resin was used to bond CFS to the specimens.

2.2 Specimen Size and Reinforcement Arrangement: Fig. 1 shows the detail of CFS-reinforced RC beams with two different depths that were 210mm for Type Ⅰ and 250mm for Type Ⅱ, respectively.

2.3 CFS Bonding Procedures: First, the bottom surface of RC beams was ground smoothly. Then, epoxy primer and connection epoxy were applied to the bottom surfaces of RC beam specimens. A single layer of CFS was then placed on the bottom of test specimen in the same direction as the primary steel re-bars.

2.4 Test Method (Bending Test): The bending test was used a static load that was performed by the wheels stopped in the center of the span, the point where the maximum bending stress occurs. The load was increased from 0.0kN with 5.0kN increments until the test specimen failed.

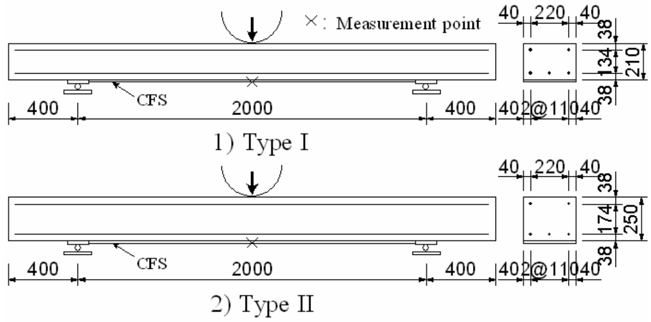

Fig. 1 Specimen size and steel re-bars arrangement (unit:mm)

3. MAXIMUM FLEXURAL LOAD-CARRYING CAPACITY

3.1 Test load-carrying capacity: Table 1 shows the test results of maximum flexural load-carry capacity. The average of maximum flexural load-carry capacity for Type Ⅰ and Ⅱ are 120.3kN and 137.5kN, respectively. The failure condition for both Types Ⅰ and Ⅱ were CFS peeled off after flexural failure.

3.2 Theoretical load-carrying capacity: The theoretical maximum flexural load-carrying capacity, P_u, is calculated from the following Eq.1.

$$P_u = 4 \cdot M_u / L \tag{1}$$

where, L is the span length and M_u is the bending moment capacity.

The two bending moment capacity calculations show below. The First, the modified bending moment capacity equation is modified from the previous tests results with consideration of the load factor coefficient of steel re-bars' yield strength and the cross-section (β_{cf}) of the RC beams in the calculation. The second, the bending moment capacity equation is from CEB-FIP.

(1) The modified bending moment capacity from the test [1]: The maximum flexural load-carrying capacity of CFS-reinforced RC beams can be calculated by adding the maximum flexural loading capacity of CFS to the maximum flexural load-carrying capacity of a non-reinforced RC beam. Accordingly, the maximum

Key words: Carbon Fiber Sheet, load factor, maximum flexural load-carrying capacity, yield strength of steel re-bars, and CFS-reinforced
Address: Izumi-Chou, Narashino-Shi, Chiba, 275-8575, Japan Civil Engineering Dept., College of Industrial Tech., Nihon University

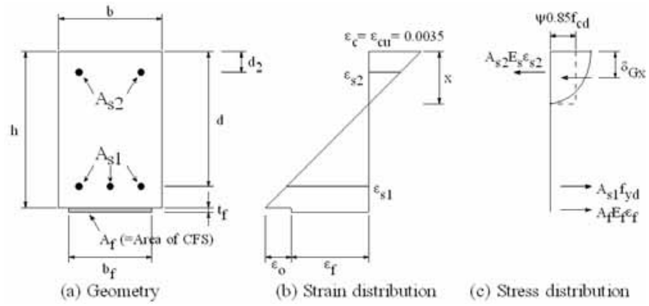

Fig. 2 Cross section of the maximum limit state in bending [2]

flexural load-carrying capacity of the RC beam reinforced with the CFS under the static load can be expressed by the Eq.2.

$$M_{uc} = \rho_k A_{s1} f_{yd} \left(d - a/2\right) + A_{s2} E_s \varepsilon_{s2} \left(d_2 - a/2\right) + 0.9 A_f f_{ycf} \beta_{cf} \left(h - a/2\right) \qquad (2)$$

where, ρ_k is the load coefficient (=1.13), $ß_{cf}$ from Eq.3, A_f is the cross-sectional area of CFS, f_{ycf} is the tensile strength of the CFS, x is the compression height, a is the height of equivalent stress block, and h is the depth of beam.

The coefficient for the reinforcing effect of CFS ($ß_{cf}$) [1] is the relationship with the ratio of beam width (b) to beam height (h) and its given by Eq.3.

$$\beta_{cf} = 0.57\left(b/h\right) - 0.15 \qquad (\beta_{cf} = 0.7 \text{ for } \beta_{cf} > 0.7) \qquad (3)$$

where, b is the width of beam.

In Eq.2 [1], the yield strength of the steel re-bars is set to $1.13 \cdot A_{s1} \cdot f_{yd}$ that the specimen is an undamaged RC beam, and its strength remains after the yielding of the steel re-bar due to the strain hardening.

(2) The bending moment capacity from CEB-FIP [2]:

$$M_{Rd} = \rho_k A_{s1} f_{yd} \left(d - \delta_G x\right) + A_f E_f \varepsilon_f \left(h - \delta_G x\right) + A_{s2} E_s \varepsilon_{s2} \left(\delta_G x - d_2\right) \qquad (4)$$

where, $x = \dfrac{A_{s1} f_{yd} + A_f E_{fu} \varepsilon_f - A_{s2} E_s \varepsilon_{s2}}{0.85 \psi f_{cd} b}$,

$\varepsilon_{s2} = \varepsilon_{cu} \dfrac{x - d_2}{x}$, $\varepsilon_f = \varepsilon_{cu} \dfrac{h - x}{x} - \varepsilon_o \le \varepsilon_{fud}$, $\varepsilon_{s1} = \varepsilon_{cu} \dfrac{d - x}{x} \ge \dfrac{f_{yd}}{E_s}$,

δ_G is 0.4, ψ is 0.8, and ε_o is initial strain of concrete.

The ultimate limit state design of CEB-FIP is based on the critical cross section that occurs by yielding of the tensile steel re-bars followed by crushing of concrete. Fig.2 shows the design bending moment of the strengthened cross section that based on principles of RC design. Eq.4 is the design bending moment capacity and the load coefficient has been added. When the load coefficient is 1, it is same as the original equation from CEB-FIP. When the load coefficient is 1.13, it was been modified from the pervious studied and experiment results [1]. The results show in Table 1.

4. COMPARING THE MAXIMUM LOAD-CARRYING CAPACITY FOR THE TEST AND THE THEROTICAL

The theoretical calculation for the modified bending moment is

Table 1 Maximum flexural load-carrying capacity

		Ultimate flexural load-carrying capacity (kN)			Test
		Test	Load coefficient (ρ_k)	Theoretical	Theoretical
Modified from test (Eq. 1 and Eq. 2)	Type Ⅰ	120.9	1.13	115.00	1.05
		119.7			1.04
	Type Ⅱ	139.8	1.13	135.30	1.03
		135.1			1.00
From CEB-FIP (Eq. 1 and Eq. 4)	Type Ⅰ	120.9	1	112.85	1.07
		119.7			1.06
	Type Ⅱ	139.8	1	165.85	0.84
		135.1			0.81
	Type Ⅰ	120.9	1.13	122.01	0.99
		119.7			0.98
	Type Ⅱ	139.8	1.13	177.35	0.79
		135.1			0.76

115.0kN and 135.50kN for the Type Ⅰ and Ⅱ, respectively. When the load coefficient is 1, the bending moment from CEB-FIP is 112.85kN and 165.85kN for the Type Ⅰ and Ⅱ, respectively. When the load coefficient is 1.13, the bending moment from CEB-FIP is 122.01kN and 177.35kN for the Type Ⅰ and Ⅱ, respectively. The average ratios between the test results and the theoretical results with the modified bending moment are 1.05 and 1.02 for the Type Ⅰ and Ⅱ, respectively. When the load coefficient equals 1, the average ratios with the bending moment from CEB-FIP are 1.07 and 0.83 for the Type Ⅰ and Ⅱ, respectively. When the load coefficient equals 1.13, the average ratios for the CEB-FIP are 0.99 and 0.78 for the Type Ⅰ and Ⅱ, respectively.

5. CONCLUSION

(1) From the pervious studies, the load coefficient (ρ_k) is related to the yield strength of the steel re-bars. Moreover, the test specimens failed around 13% higher of the steel re-bars' yield strength for RC beam specimens under the static load. In the word, adding this coefficient into the CFS-reinforced RC beam's calculation, the RC beams with or without CFS-reinforced were both failed around 13% higher of the steel re-bars' yield strength. Therefore, it is possible to add this load coefficient into the bending moment calculation of the CFS-reinforced RC beams.

(2) The bending moment from CEB-FIP is base on the stress distribution. Also, the CFS strain (ε_f) should be checked and not exceed the ultimate strength (ε_{fud}), and the tensile steel re-bar strain (ε_{s1}) should be greater than the design steel re-bar yield strain (f_{yd}/E_s). If calculation does not follow these limitations, the calculation of the bending moment should consider the steel yielding followed by the CFS fracture instead of the concrete crushing. In the other hand, the ultimate strength of CFS (ε_{fud}) will be used in the bending moment calculation that is what happens for calculating the TypeⅡ specimen.

References:

[1] Tadashi ABE, Tetsukazu KIDA, Toshiaki SAWANO, Masaaki SHIMIZU, Kiyoshi KATO: The 53rd Nat. Cong. Of Theoretical & Applied Mechanics, pp.215-216 (2004)

[2] CEB-FIP, Externally bonded FRP reinforcement for RC structures, pp.34-36 (July 2001)