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SEISMIC RETROFIT WITH CARBON FIBERS FOR REINFORCED CONCRETE COLUMNS

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ABSTRACT

Recent large earthquakes have revealed the vulnerability of existing reinforced concrete structures constructed according to old design codes. For these structures, seismic retrofit work is needed before the next earthquake. Carbon fiber winding method is a new seismic retrofit method for existing reinforced columns, and is superior to steel plate jacketing or reinforced concrete jacketing in cost or simplicity of construction. This paper describes structural performance of the columns retrofitted with this method, emphasizing evaluation of improvement in shear capacity and ductility.

KEYWORDS

Seismic Retrofit; Carbon Fiber; Reinforced Concrete Column; Ductility; Shear Reinforcement; Evaluation of Earthquake-Resistant Capacity

INTRODUCTION

The 1995 Hyogo-Ken-Nanbu Earthquake (or the Hansin Earthquake) struck and destroyed Kobe City and the neighbor area. This earthquake revealed the vulnerability of the old buildings designed and constructed according to the previous building code in Japan. One of important reasons for collapsing of old reinforced concrete buildings is considered to be shear failure of columns that have poor transverse reinforcement. Therefore, seismic retrofitting is needed for such brittle columns before the future large earthquakes.

Carbon fiber winding method (see Fig. 1), which provides additional transverse reinforcement, was already developed as a new seismic retrofitting technique for existing reinforced concrete columns (Katsumata, *et al.*, 1988). The carbon fiber winding is superior to steel plate jacketing or reinforced concrete jacketing in cost and simplicity of construction, which recently results in widely spreading this method in actual seismic retrofitting projects. Carbon fiber is not familiar to civil and building engineering fields. However, it has already been used for aircraft or sports goods because of many strong points, for example, high strength, high elastic modulus, light weight and high durability. On the other hand, there are some week points, that is, high brittleness and high price. However, transverse reinforcement of ordinary reinforced concrete members does not need to be ductile, so that even brittle carbon fiber can be applied for transverse reinforcement. In retrofitting operations, labor cost occupies a large part of total construction cost, so that high price of carbon fiber can be absorbed.

The carbon fiber winding improves both shear capacity and deformation capacity of existing reinforced concrete columns. The improvement of shear capacity is discussed in this paper, employing the shear capacity evaluation for ordinary reinforced concrete members by Architectural Institute of Japan (AIJ, 1990 or Aoyama, 1991). This paper also describes the improvement of deformation capacity, including an evaluation method for deformation capacity and a calculation technique for a ductility index F. Evaluation

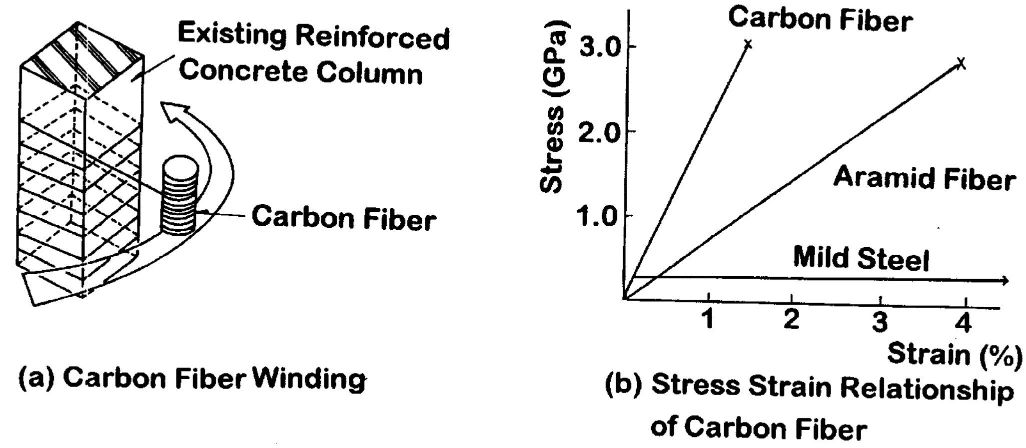


Fig. 1 Carbon Fiber winding

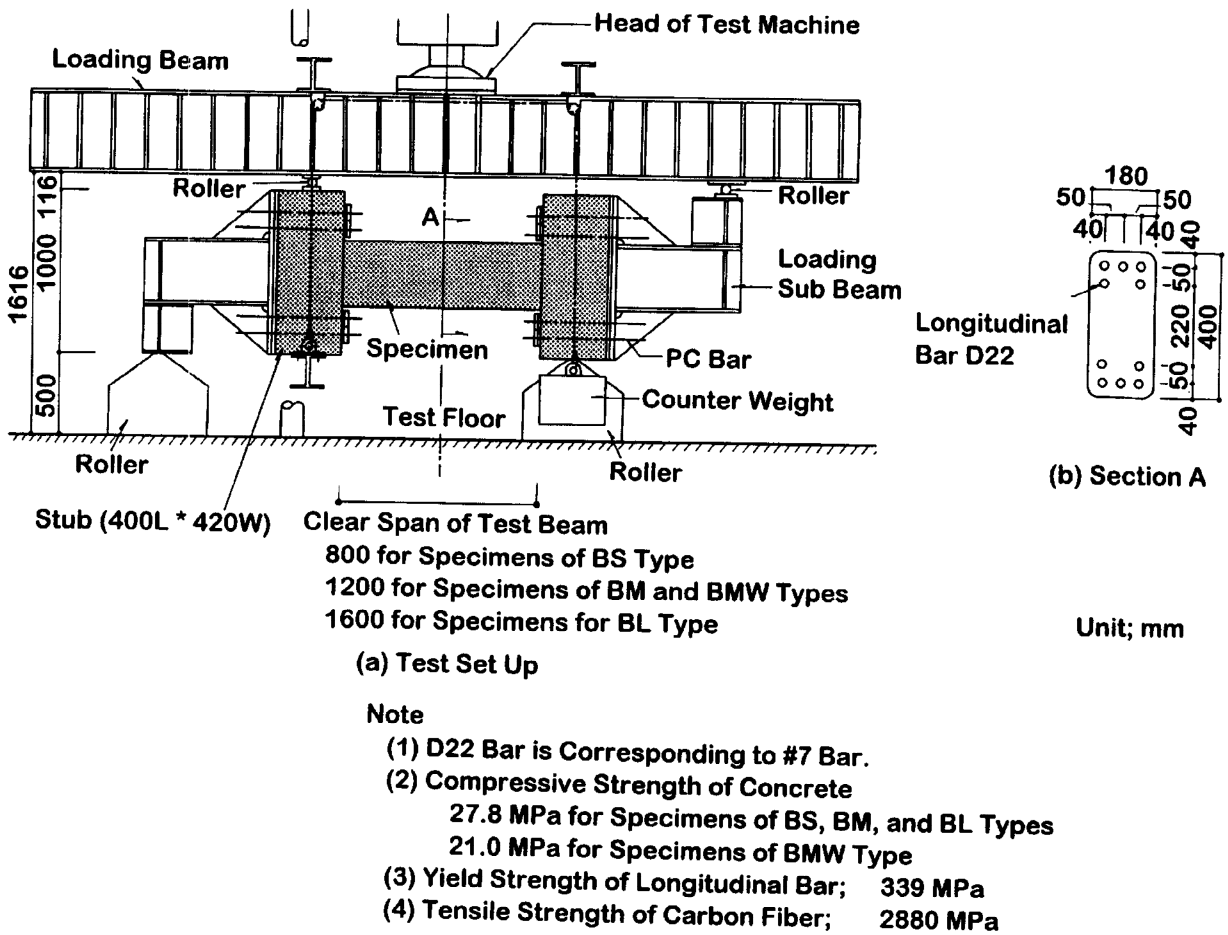


Fig. 2 Test Set Up for Shear Capacity Test

of the F index is strongly needed for seismic retrofitting design according to "Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings " (JBDPA, 1977; revised in 1990). This standard, which is the most popular guideline for retrofitting in Japan, employs the F index as one of essential indices of seismic resistant capacity.

SHEAR CAPACITY EVALUATION

Shear Loading Test

A static loading test was carried out to establish a shear capacity estimation method for carbon-fiber-retrofitted reinforced concrete columns and to define an effective tensile strength of carbon fiber for the shear capacity evaluation. Test specimens, shown in Fig. 2, were scaled reinforced concrete beams retrofitted with carbon fibers. Test variables were as follows.

(1) Quantity of carbon fibers (transverse reinforcement ratio p_w is 0.00 to 0.24%)
(2) Aspect ratio of the beams (2.0 to 4.0)
(3) Concrete strength (21.0 MPa and 27.8 MPa)

The above transverse reinforcement ratio p_w is defined in the same way for ordinary reinforced concrete beams using steel stirrups, employing the following equation.

$$p_w = a_{cf} / (bx) \tag{1}$$

where, a_{cf}; gross area of carbon fiber within spacing x
b; width of the column

These variables are considered to have a large effect on shear capacity of reinforced concrete members. Other factors, for example, axial force or stirrups, were eliminated to clarify the retrofitting effect by carbon fibers. Note that the longitudinal reinforcement of the specimens was quite large so that the specimens would fail in shear. The employed loading apparatus is shown in Fig. 2. This apparatus can produce the anti-symmetric stress state that may occur in actual buildings under an earthquake load.

Crack patterns of the representative specimens are illustrated in Fig. 3. The non-retrofitted specimen showed cracks along the diagonal direction while the retrofitted specimens showed more inclined cracks. The angle of inclined cracks in the heavily retrofitted specimen was almost 45 degrees from the member axis. If the crack direction indicates the compressive principal stress direction, then it is necessary for equilibrium against the concrete compressive force that carbon fibers produce tensile reactions. Such action of concrete stress is identical to "truss action" called by the AIJ's method (AIJ, 1990). The diagonal cracks, previously mentioned, indicate "arch action" called by the AIJ's method, which is the main shear resisting mechanism for beams with small transverse reinforcement. Moreover, horizontal cracks along longitudinal bars were also observed. Although these cracks were quite small during the testing, bond splitting might occur because of a large amount of longitudinal reinforcement.

The stress distributions of carbon fibers are shown in Fig. 4. From the carbon fiber strains measured when the specimen showed maximum shear force, these distributions were calculated using elastic characteristics of carbon fibers. Fig. 4 indicates almost constant stress distributions although carbon fiber did not have a

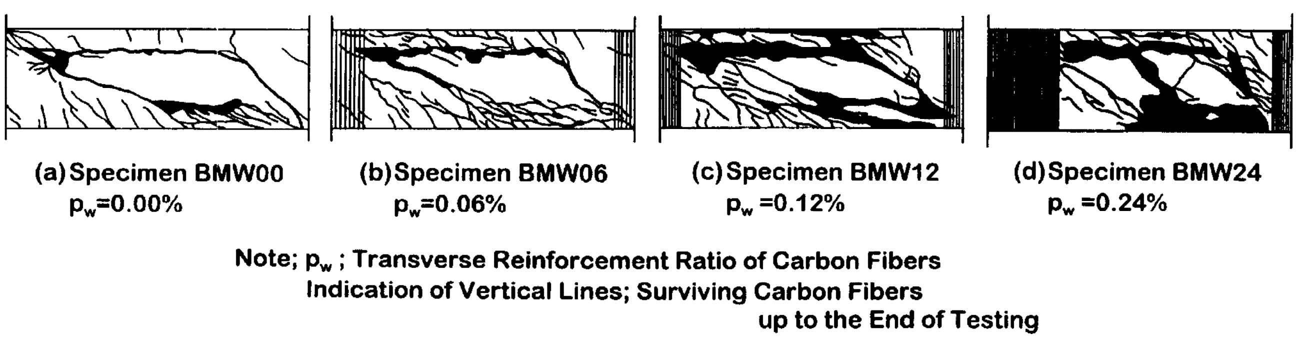


Fig. 3 Representative Crack Patterns of Carbon -Fiber-Retrofitted Specimens

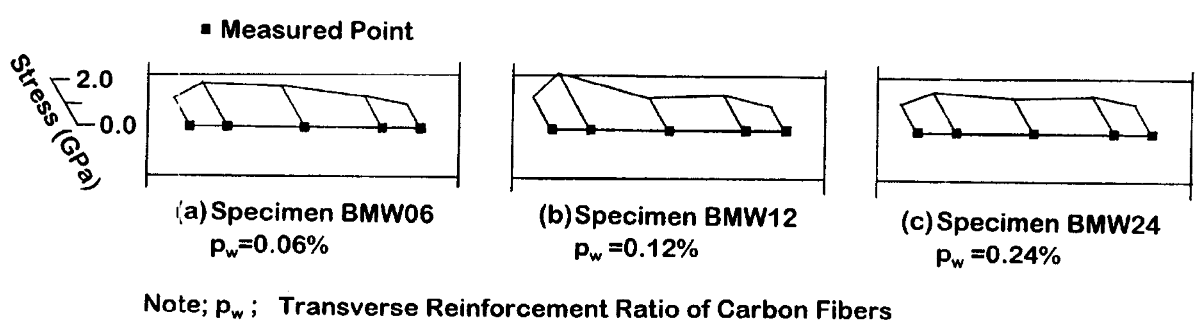
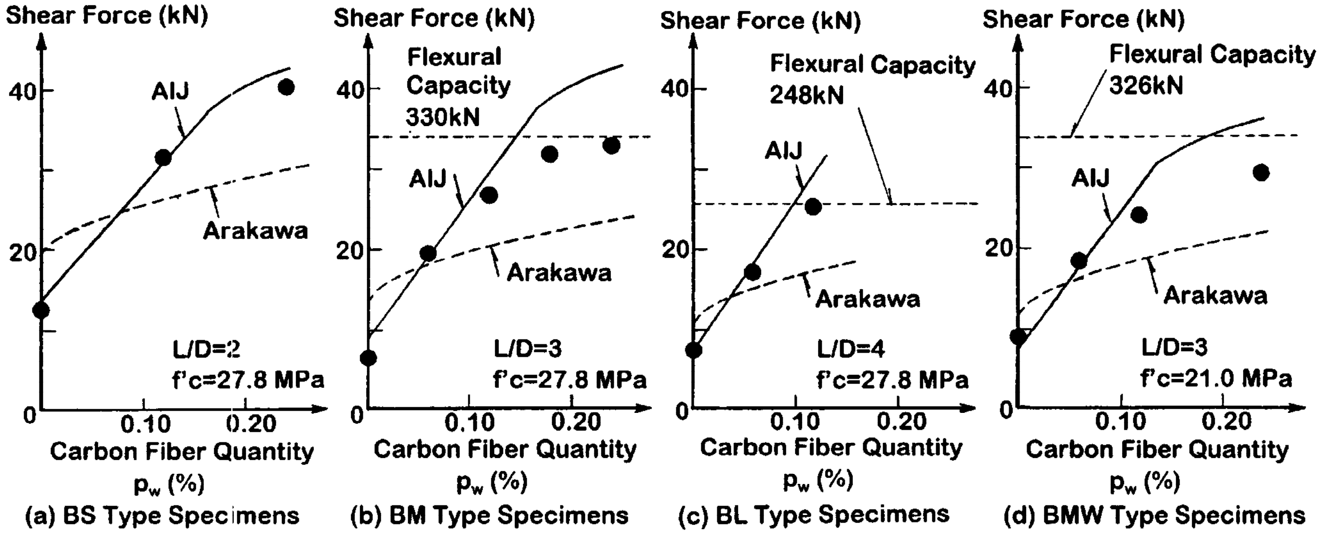


Fig. 4 Stress Distributions of Carbon Fibers



Note; p_w; Transverse Reinforcement Ratio of Carbon Fibers
L/D; Aspect Ratio of Test Beam
f'c; Compressive Strength of Concrete
●; Observed Maximum Shear Forece in the Testing
Arakawa; Predicted Shear Capacity by Arakawa's Estimation
AIJ; Predicted Shear Capacity by AIJ's Estimation

Fig. 5 Predicted and Observed Shear Capacity

plasticity to contribute to such constant distributions. Consequently, it is considered that the plasticity of concrete, which is not so large, contributed to these distributions. The stress of carbon fibers, moreover, was approximately 2/3 of full tensile strength of carbon fiber. As a result, an effective strength of carbon fibers for shear strengthening is defined to be equal to 2/3 of full tensile strength of carbon fiber.

Evaluation method for shear capacity

The maximum shear forces obtained from this parametric test were compared to the prediction according to the AIJ's method (AIJ, 1990), as shown in Fig. 5. Note that Fig. 5 also indicates Arakawa's shear capacity estimation, which was developed in 1960s and still provides the basis of the shear design equation of AIJ's code (AIJ, 1988). The AIJ's method assumes two shear resisting mechanisms (arch action and constant truss action). The observed crack patterns and the measured constant stress distributions of carbon fibers approximately satisfy this assumption. From Fig. 5, it is understood that the AIJ's prediction agrees with test results, especially for the range that carbon fiber quantity is not so large. On the other hand, Arakawa's estimation is almost conservative and does not predict enhancement of shear capacity by carbon fibers. It is noted that the flexural capacity of the tested specimen was larger because of a large amount of longitudinal reinforcement. As a result, it is concluded that the AIJ's prediction method for shear capacity can be applied for carbon-fiber-retrofitted members with an ordinary amount of longitudinal reinforcement.

DEFORMATION CAPACITY EVALUATION

Ductility Test

Fig. 6 shows a ductility test result of carbon-fiber-retrofitted reinforced columns with 3.0 in aspect ratio and 1200 mm in clear height (Katsumata, et al., 1995). Under reversal horizontal loads and a constant axial force, anti-symmetric deformation was applied to the column specimens. Fig. 6 presents a typical example on the improvement in ductility by carbon fibers. In this study, ultimate displacement is defined as the displacement beyond which decrease in bearing capacity for horizontal loading is clearly recognized. Fig. 6 shows that the carbon fiber winding improves the ultimate displacement very much. For the heavily retrofitted specimen, the ultimate displacement may be larger than 100 mm (drift angle = 1/12), which was the limit of the loading apparatus of this testing. It is noted that the ultimate displacement of carbon-fiber-retrofitted columns is usually determined by fairly large fracturing of carbon fibers. The quick reduction of

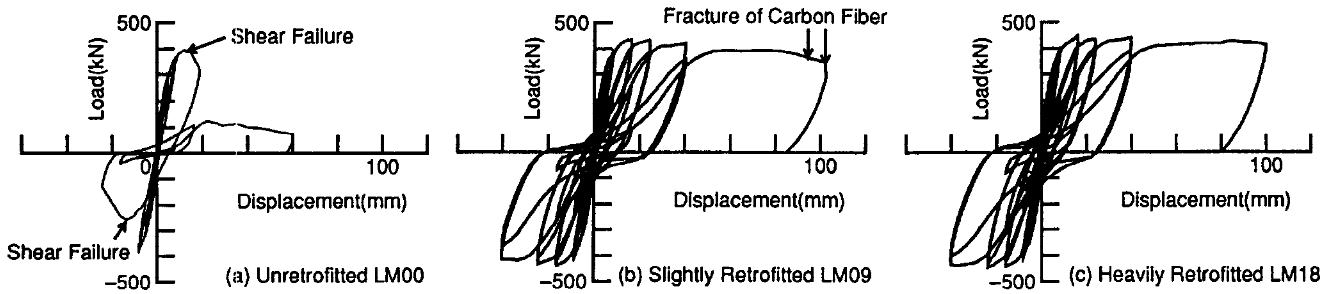


Fig. 6 Load-Displacement Relationship of a Ductility Test

bearing capacity is observed after such fracturing.

From the previous test results on the carbon fiber winding (Katsumata, *et al.*, 1986, 1988, and 1995), the data of ultimate displacement were obtained in the above mentioned way. In these tests, some variables were almost fixed. The aspect ratio of column specimens was 1.5, the level of axial stress was approximately f'c/6 (f'c; compressive strength of concrete), and transverse steel reinforcement ratio within the column concrete was 0.10% in average. However, other variables slightly varied. For example, longitudinal reinforcement ratio was 0.88 to 1.27%, dimensions of cross section were 200 to 400 mm square or 226 mm in diameter, and the concrete strength f'c was 17.6 to 27.4 MPa.

Evaluation of Ultimate Drift Angles

The standard for existing buildings (JBDPA, 1977) evaluates ultimate ductility factor of long columns by means of a function of a "shear safety margin." The shear safety margin is defined as ratio of shear capacity Qsu to flexural capacity Qmu. The basis of this is the well-observed fact that columns with the large shear safety margin have large deformation capacity. In this paper, the shear safety margin is also employed for evaluation of deformation capacity. Note that flexural capacity is calculated according to the JBDPA's standard, which is derived from AIJ's standard for more general reinforced concrete structures (AIJ, 1988) than the other AIJ's code (AIJ, 1990). On the other hand, shear capacity is evaluated according to the previously mentioned method, and not according to the JBDPA's standard. Consequently, the expression of the function for ductility evaluation in this paper is different from the original expression in the JBDPA's standard.

Fig. 7 shows the relationship between the shear safety margin Qsu/Qmu and an ultimate drift angle Ru, that is, normalized ultimate displacement (= ultimate displacement / clear height of the column). Although the plotted marks are scattered, this extent of scattering is often found for ordinary reinforced columns. In general, it is found that a large shear safety margin provides a large ultimate drift angle. Consequently, a line connecting lower limits of the plotted marks is established as follows.

$$Ru = (Qsu/Qmu - 1)/15 + 1/75 \qquad (2)$$

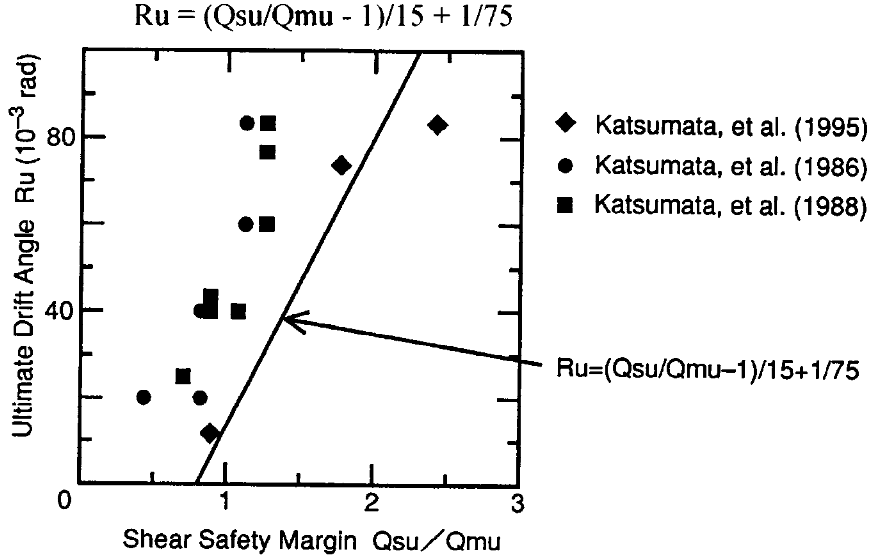


Fig. 7 Relationship between Ultimate Drift Angles and Shear Safety Margins

evaluation of the F index

In the JBDPA's standard for existing buildings, the F index is defined as a function of the ultimate ductility factor μ of a story in a building. In this study, the calculation from the ultimate ductility factor to the F index is carried out, employing the same function as the JBDPA's standard. The relationship between the F index and the ultimate ductility factor μ was established by computer analyses, which means that the restoring force characteristics of the analyses determined this F-μ relationship. On the other hand, carbon-fiber-retrofitted columns generally show the same restoring force characteristics as ordinary reinforced concrete columns that fail in flexure. Therefore, there is no need to change the F-μ relationship.

To establish the F index evaluation from the former discussion of the ultimate drift angle, conversion of drift angles is required from a member level to the story level. The conversion is carried out by multiplying the ratio of the column clear height to the story height. Note that in ordinary low rise buildings needing for seismic retrofitting in Japan, dimensions of typical columns are 600 to 700 mm square and the story height can be assumed to be 3000 to 4000 mm. As a result, the previously mentioned specimens of the ductility test with aspect ratio of 1.5 can be considered to have clear height of 1800 to 2000 mm. In this paper, the story drift angle is assumed, in the safety side, to be half of the member drift angle.

The JBDPA's standard for existing buildings also expects that the yield drift angle of a story is always equal to 1/150. This study also employs this value because of the same reason for the F-μ relationship mentioned before. Under these assumptions on the ultimate and yield drift angles, the ultimate ductility factor μ can be expressed as follows.

$$\mu = 5(Qsu/Qmu - 1) + 1 \qquad (3)$$

CONCLUSION

Estimation methods of shear capacity and deformation capacity were developed for the seismic retrofit method with carbon fibers for reinforced concrete columns. The shear capacity by this estimation agreed well with the results of the shear behavior test. The ductility estimation was conservative however within satisfactory precision. Employing these estimation methods, practical seismic retrofitting calculation is possible, so that many columns have already been retrofitted with carbon fibers in Japan.

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