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ORIGINAL ARTICLE

New High-Workability Seismic Retrofitting Techniques for Existing Reinforced Concrete Structures

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***Abstract*:** When Japan's seismic design standards are updated based on learning from observed earthquake damage, older reinforced concrete (RC) structures are often retrofitted to bring them up to standard. However, conventional retrofitting work methods are not always applicable because of workspace and time restrictions, so new methods are required for strengthening work in such cases. From such background, two newly developed methods are described in this paper. The first involves inserting plate-anchored reinforcing bars into pre-drilled holes for the shear strengthening of underground RC structures. The second is a method of installing carbon fiber composite panels for the shear strengthening and ductility improvement of RC structures such as viaduct columns. The conceptual background to these newly developed retrofitting techniques is explained, along with design methods, implementation processes and experimental verifications carried out by the authors.

Keywords: seismic retrofit, underground structure, column, shear, ductility

1. Introduction

During Japan's period of high economic growth in the 1960s and later, the country's infrastructure developed rapidly. However, infrastructure dating from that period suffered serious damage in the Hyogoken Nanbu Earthquake of 1995. Similarly, in major earthquakes since 1995, a large amount of damage has been experienced. In response, there have been revisions to the seismic design criteria, such as updating the 'seismic action in design'. This means that existing reinforced concrete (RC) structures need to be retrofitted if they are to have adequate aseismic performance into the future.

Various retrofitting techniques are available for improving flexural capacity, shear capacity or ductility according to performance requirements. The load-displacement relationship of an RC member before and after strengthening using a retrofit technique can be conceptualized as in Fig.1. An example of the retrofit process, from planning to implementation, is described in the JSCE Standard Specification for the Maintenance of Concrete

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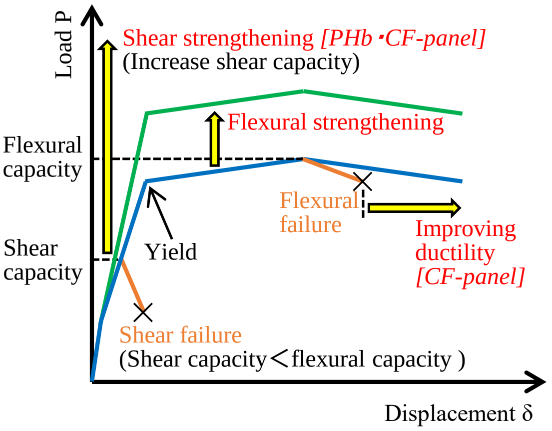


Fig.1 The load-displacement relationship of RC members

Structures (JSCE 2018c) in the chapter "Correspondence to change of required performance level". Specific seismic retrofit methods include increasing the thickness of members, concrete jacketing, fastening or jacketing with steel plates or a continuous fiber material, etc. The most suitable method must always be selected according to the members to be treated, design conditions and site conditions.

Site conditions often impose severe constraints on the application of conventional seismic retrofit methods, especially in urban areas. In response to such difficulties, the authors have developed two seismic retrofit techniques that offer excellent workability combined with good strengthening effects. The first is a method using retrofitted plate-anchored reinforcing bars (PHb) to improve the shear capacity of members, typically wall members, of underground structures. The second is a method using carbon fiber composite panels (CF-panel) to improve the shear capacity and ductility of columns in locations where workspace is limited.

This paper outlines the concept behind the seismic retrofitting of RC members using these methods and describes the development, design and implementation of the PHb and CF-panel techniques.

2. Development of shear strengthening method using retrofitted plate-anchored reinforcing bars (PHb method)

2.1 Shear strengthening concept and overview of PHb method

When RC members fail in shear, the failure tends to be brittle. Brittle failure generally leaves little redundancy and is not a desirable failure mode. For this reason, RC members should be designed to fail in a flexural mode that does not lead to immediate collapse once the ultimate displacement is reached. To ensure this, it is necessary for shear capacity to exceed flexural capacity by a suitable margin.

According to the pre-1980 design standard, the allowable shear stress of concrete was relatively higher than under the current standard and there was no allowance for the reduction in shear strength with larger cross sections. Further, the standard stipulated that all shear force was to be borne by the shear reinforcement alone once the induced stress exceeded the allowable shear stress. Therefore, in the design of RC members at that time, it was considered economical to reduce the shear reinforcement to a minimum by enlarging the cross section, thereby increasing the share of shear force borne by the concrete. As a result, there are cases where the shear capacity of members designed under the earlier standard is insufficient compared to the latest criteria, so appropriate shear strengthening is required.

Supposing a wall is reinforced against seismic forces in the out-of-plane direction, it is necessary only to increase shear capacity without increasing flexural capacity. This would be applied to box culverts and in-service storage tanks, etc. One seismic retrofit method available in such cases is to excavate the backfill and increase the concrete thickness, but cost and construction period are enormous. An alternative is desired in which shear capacity is

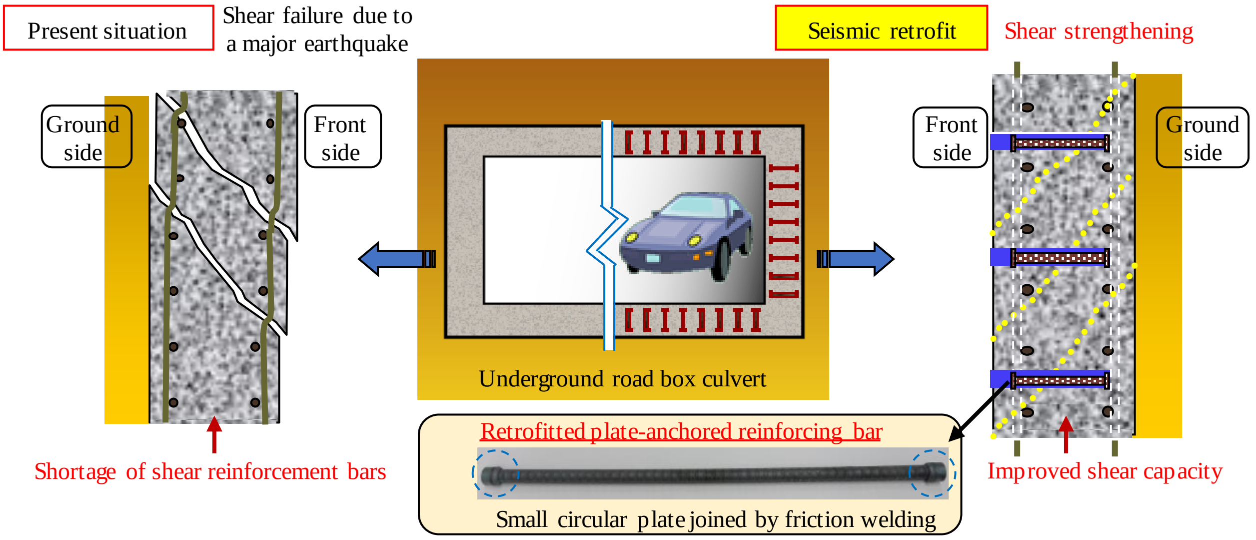


Fig.2 PHb shear strengthening method

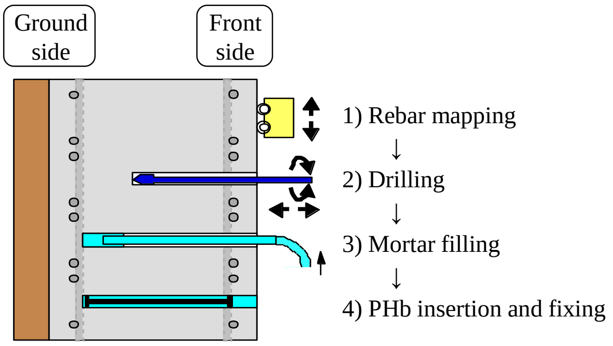


Fig.3 PHb implementation procedure


Drilling
PHb insertion and fixing

Photo1 PHb work in progress

improved by working only from inside the wall without increasing wall thickness.

The shear strengthening technique developed for such cases entails inserting PHb reinforcing bars into the existing RC member, as shown in Fig.2. A PHb is a reinforcing bar (rebar) with small circular steel plates attached to the ends by friction welding. These plates improve the anchorage performance.

Using this method, it is possible to carry out a seismic retrofit at lower cost and without excavating behind the structure. The method is also effective in some cases where there must be no increase in cross-sectional dimensions, such as in members for weirs and water gates where a change in dimensions could disturb the flow of water.

2.2 PHb implementation procedure

The procedure used to implement the PHb method is shown in Fig.3. An example of actual work is shown in Photo1. Before carrying out the work, existing rebars near the surface are mapped using radar. Based on the results, holes are drilled until just short of the main rebar at the rear (ground) side of the wall using a specially developed drill. Two types of drill have been developed for this work. One is a percussion leg drill operated by hydraulic pressure and with low propulsion torque to avoid damage to the rebar. The other is a special core drill equipped with a device that automatically stops drilling at the time of rebar contact. They incorporate a roughening tool to prepare a fixing surface at the hole wall. The special core drill is used when workspace is limited or there is considerable depth to be drilled. This drill is also effective when it is desired to reduce noise during drilling. After drilling has been completed, the hole is filled with a non-shrinking mortar and the PHb is inserted and fixed. The mortar used has been specially developed to ensure the filling of narrow spaces and its filling performance has been verified in tests.

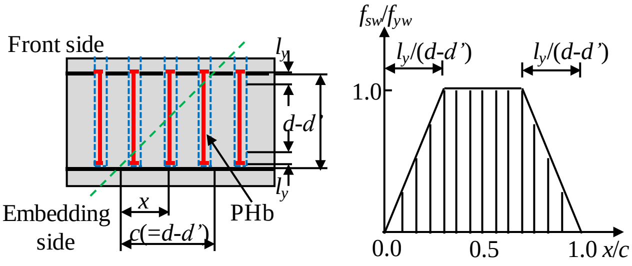


Fig.4 Stress distribution of PHb with diagonal crack

2.3 Design method for shear strengthening with PHb

The shear capacity of RC members strengthened with PHb, V_{yd}, can be evaluated using formula (1). Here, V_{yd} is expressed as the sum of the shear capacity V_{phbd} contributed by the PHb and the shear capacity calculated using modified truss theory as shown in the JSCE Standard Specification for Design of Concrete Structures (JSCE 2018b). It is assumed that an RC member strengthened with PHb carries the load imposed by a shear force based on a truss mechanism. However, the anchoring performance of PHb is different from that of ordinary shear reinforcement bars, where semicircular hooks are hung from the main rebars. Therefore, an appropriate value of V_{phbd} is obtained by multiplying the truss theory value, V_{awd}, by a factor β_{aw} representing the shear strength effectiveness of PHb, as shown in formulas (2) and (3).

$$V_{yd} = V_{cd}+V_{sd}+V_{phbd} \tag{1}$$

$$\begin{aligned} V_{phbd} &= \beta_{aw}\cdot V_{awd} \\ &= \beta_{aw}\cdot\{A_{aw}\ f_{awyd}(\sin\alpha_{aw}+\cos\alpha_{aw})/S_{aw}\}z/\gamma_b \end{aligned} \tag{2}$$

$$\beta_{aw} = 1\text{-}l_y/(d\text{-}d')\ \ (d\text{-}d'\geqq 2l_y) \tag{3}$$

where, V_{cd} is the design shear capacity of linear members without shear reinforcing steel (N) (JSCE 2018b); V_{sd} is the design shear capacity of the existing shear reinforcement (N) (JSCE 2018b); V_{awd} is the design shear capacity if PHb is regarded as normal shear reinforcing steel (N); β_{aw} is a factor indicating the shear strength effectiveness of the PHb (effective factor); A_{aw} is the total cross-sectional area of PHb placed in S_{aw} (mm^2); f_{awyd} is the design yield strength of the PHb (N/mm^2); α_{aw} is the angle between the PHb and the member axis; S_{aw} is the PHb spacing (mm); z is the distance from the location of the compressive stress resultant to the centroid of the tension steel (mm); γ_b is a member factor; l_y is the required development length of PHb end (mm); and d-d' is the distance from compression steel to tension steel (mm).

This design methodology is adapted from an earlier idea for evaluating the reinforcement effect of a stirrup where the bent part has failed due to rebar corrosion (Regan and Kennedy Reid 2004). The effectiveness factor β_{aw} calculated using formula (3) is a reduction coefficient related to the performance of the end anchorage. The basic concept of β_{aw} is shown in Fig.4. In the figure, f_{yw} indicates the yield stress of PHb and f_{sw} is the stress that PHb can bear where it intersects with the diagonal crack. In this distribution, there is no capacity to bear shear stress at the ends, but a shear stress equivalent to the yield stress can be borne at points deeper than the development length l_y from both the ends. In the section up to l_y from ends, the capacity to bear shear stress is assumed to have a linear distribution.

An additional premise of this design methodology is that the spacing of PHb reinforcing bars must not exceed half of the effective member depth so as to ensure that a PHb intersects with the diagonal crack. In a case where this requirement is not satisfied, it is important to keep in mind that the reinforcement

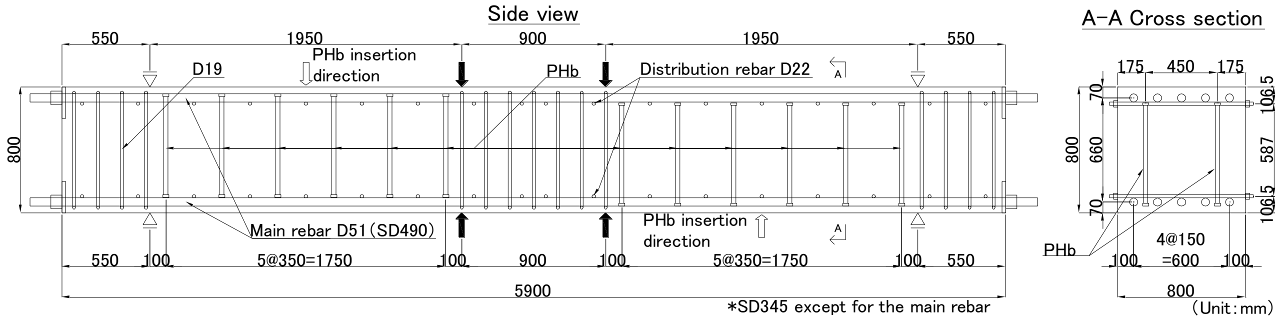


Fig.5 RC beam specimen

Table1 Specifications of the specimen and material test results

Hole diameter of PHb insert	46mm
Dimensions of end anchor plate of PHb	φ38mm t=16mm
PHb total length	655mm
Ratio of shear reinforcement	0.36%
Yield strength of PHb	396N/mm^2
Ratio of tension reinforcement	1.74%
Yield strength of tension reinforcement	525N/mm^2
Compressive strength of concrete	40.7N/mm^2
Compressive strength of mortar	63.5N/mm^2

effect assumed in design may not be achieved (Kumagai et al. 2017).

2.4 Experiment to verify shear strengthening effect of PHb method

In order to verify the shear strengthening effect of the PHb method, an RC beam specimen is subjected to cyclic loading tests. As shown in Fig.5, the specimen is 800 mm wide x 800 mm in height x 5,900 mm in total length, and the shear span ratio *a*/*d* is 2.67. Specifications of the specimen and material test results are shown in Table1. Shear strengthening is carried out by inserting PHb reinforcing bars of diameter D25 into the specimen, which has no shear reinforcement bars in the shear span. The insertion direction, as shown in Fig.5, is from the top in the left half of the shear span and from the bottom in the other half.

Photo2 experimental setup

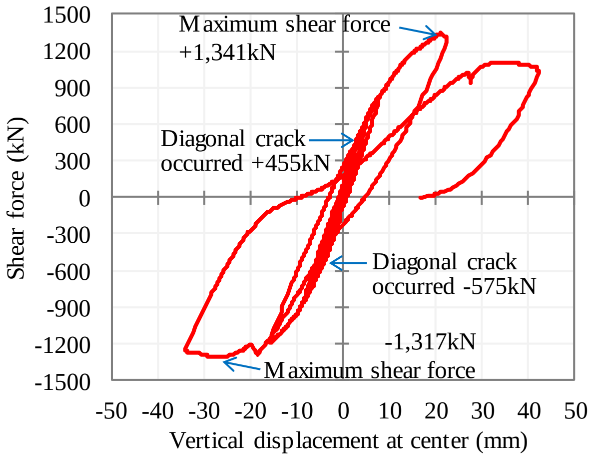


Fig.6 Relationship between shear force and vertical displacement

The two center points of the specimen are initially loaded with about 1/5 of the calculated maximum force and the load is then increased alternately from the top and from the bottom at the two points. The setup of the loading test is shown in Photo2.

The relationship between shear force and vertical displacement at the center of the specimen is shown in Fig.6. During both positive and negative loading, bending cracks begin to occur on the tension side between the loading points before diagonal cracks

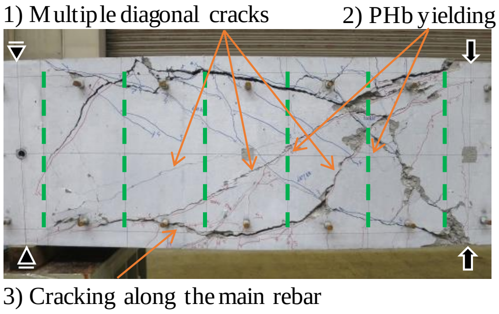


Photo3 State of cracking after experiment

are confirmed in each shear span. The shear force continues to increase even after occurrence of the diagonal cracks, suggesting that the PHb reinforcing bars intersecting with the diagonal crack bear the shear force. The shear force is first observed to decrease at +1,341kN during positive loading, and this is followed by a decrease at -1,317kN during the next negative loading.

The cracks in the span that led to shear failure are shown in Photo3. Lines on the specimen surface indicate the cracks, while PHb reinforcing bar positions are shown with green dashed lines. Numbered arrows indicate the order of occurrence of major deformations during loading on the positive side. First, dispersed diagonal cracks occur. It is thought that at this stage the integrity of bonding between PHb, filling mortar, and concrete is secured, and the tensile force on the PHb bars is transmitted to the concrete. Then PHb reinforcing bars yield around the mid-point of member thickness. This demonstrates that, although the PHb bars are not hung on the main rebars at both ends, the strengthened member has until this point a load resistant mechanism like a truss. This results from the anchoring performance of the circular end plates. At the maximum shear force, one of the diagonal cracks expands and the final fracture mode is diagonal tensile fracture. A crack along the main rebar also develops rapidly along with the diagonal crack. This may be because the influence of the PHb reinforcing bars does not extend to enclosing the main rebars as do ordinary shear reinforcing rebars.

The shear capacity value calculated by formula (1) using the actual strength of each materials is 1,099 kN. The maximum shear force in the experiment is 1,329 kN (the average of positive load and negative loads), giving a ratio to the calculated value of 1.21. With the experimental result exceeding the calculated value, it is confirmed that the designed shear strengthening effect has been obtained.

3. Development of seismic retrofit technique for columns using carbon fiber composite panels (CF-panel method)

3.1 Seismic retrofitting concept and Overview of CF-panel method

Some RC columns supporting railway and road viaducts also require seismic retrofitting. If retrofitting to increase flexural capacity using RC or steel plate jacketing, the additional longitudinal rebars or steel plates must be fixed to the footing. In other cases where the aim is to improve shear capacity and ductility without increasing flexural capacity, no such fixing is necessary and this section describes a new method for such cases with excellent workability.

In general, when RC columns are subjected to repeated horizontal forces, buckling of the longitudinal rebars can occur near the base and strength decreases as the cover concrete falls away. Improving the ductility of a column provides more stability even during earthquakes with large horizontal displacement. For example, in the JARA Design Specifications for Highway Bridges Part V “Seismic Design” (JARA 2017), the limit state is defined as the point where the horizontal force cannot be sustained. The specifications call for verification that the maximum horizontal displace- ment anticipated during an earthquake does not exceed the limit state. In the design of new structures, longitudinal reinforcement and core concrete are confined by a close arrangement of hoop reinforcement to secure sufficient deformation performance.

In order to secure sufficient ductility in structures designed based on old standards, it is necessary to jacket the columns for adding strength, because the amount of existing hoop reinforcement is inadequate according to the current design equations.

Many construction methods are available for such jacketing, but a suitable one must be chosen for each application in careful consideration of required performance, site conditions and future maintenance requirements. RC jacketing is relatively straightforward to implement and maintain, but column cross section and weight increase considerably. Steel plate jacketing is effective when there are limitations on weight or available space. But where workspace is restricted and manual methods are necessary, steel plate jacketing is difficult because it requires the use of heavy equipment. The application of continuous fiber sheet such as carbon fiber or aramid fiber is effective in such circumstances. However, careful layer-by-layer resin impregnation has to be carried out. The greater the required reinforcement effect, the more layers of sheet must be laminated, increasing the number of days required for resin impregnation work and curing. Further, careful surface treatment is needed to ensure a smooth concrete surface and the column faces have to be provided with protection to prevent future deterioration.

The alternative solution developed to overcome these problems from a workability perspective uses carbon fiber composite panels for efficient and quicker implementation (CF-panel method). A CF-panel is a three-layer composite panel consisting of a carbon fiber sheet sandwiched between two flexible fiber-reinforced cement boards. This structure is shown in Photo4. This lightweight precast reinforcement material offers excellent workability for manual construction in a relatively short time. This is a particular advantage when retrofitting the columns of railway viaducts under which stores or warehouses are located.

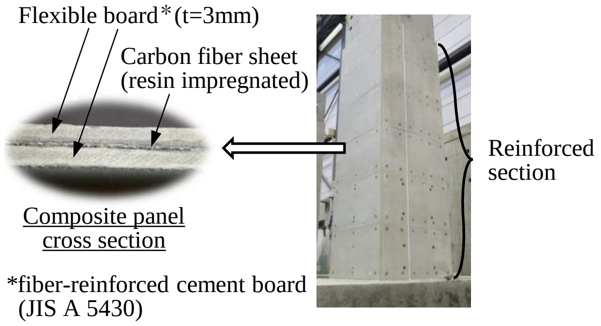


Photo4 Seismic retrofitting of column using CF-panel method

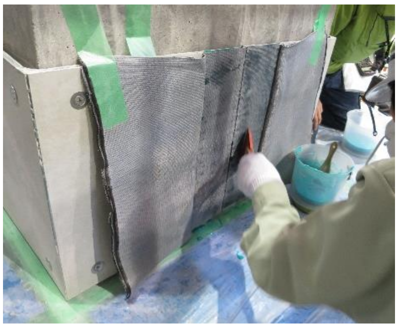
3.2 CF-panel implementation procedure

The procedure used to implement the CF-panel method is illustrated in Photo5. After surface treatment such as cleaning and application of a primer, the CF-panels are installed using temporary anchors. The panels are prefabricated into U-shaped or semi-circular shaped modules to suit the column shape.

Where the panels join, the carbon fiber sheet is left unimpregnated with epoxy resin in the factory. Then, as part of the on-site work, the fiber sheets on the left and right sides of the joint are laminated alternately and impregnated with resin. The cross section of a column strengthened using the CF-panel method is shown in Fig.7. It has been confirmed that this joint method achieves a tensile strength equal to or exceeding that of the panels themselves. Finally, after sealing the joints and the upper and lower edges, non-shrinking mortar is injected into the space between column and CF-panels via the pre-formed injection holes. Corners are formed with a curved section of carbon fiber sheet for the purpose of relieving the stress concentration.

1) Surface treatment / Primer application

2) Installation of CF panel



3) Joining (resin impregnation)

4) Seal joints and edges

5) Mortar injection

6) Construction completed

Photo5 CF-panel work in progress

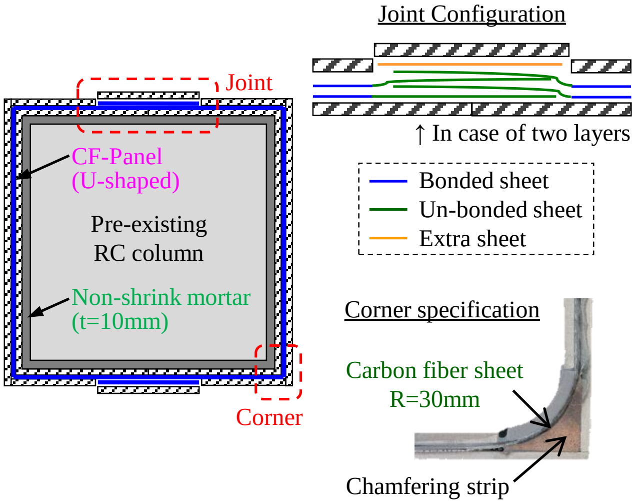


Fig.7 Cross section of column strengthened by the CF-panel method

3.3 Design method of shear strengthening and improvement ductility effect by CF-panel

The purpose of the flexible fiber-reinforced cement boards that sandwich the carbon fiber sheet is to improve workability. Reinforcement is provided only by the carbon fiber sheet. This means that the shear capacity and ductility of an RC member strengthened by this method can be evaluated using the formula for the continuous fiber sheet jacketing method proposed in the JSCE Guidelines for Repair and Reinforcement of Structures by FRP Bonding – draft – (JSCE 2018a).

The shear capacity after CF-panel strengthening, V_{fyd}, is given by formulas (4)-(8), in which the shear capacity added by the CF-panel, V_{fd}, is added to the shear capacity of the column obtained by the modified truss theory. Here, V_{fd} is evaluated by multiplying the value calculated by truss theory at a compression angle of 45° by the reinforcement effectiveness, K.

$$V_{fyd} = V_{cd}+V_{sd}+V_{fd} \tag{4}$$

$$V_{fd} = K\cdot[A_f\ f_{fud}(\sin\alpha_f+\cos\alpha_f)/s_f]z/\gamma_b \tag{5}$$

$$K = 1.68\text{-}0.67R\ \ (0.4\leq K\leq 0.8) \tag{6}$$

$$R = (p_f\cdot E_f)^{1/4}(f_{fud}/E_f)^{2/3}(1/f'_{cd})^{1/3}\ \ (0.5\leq R\leq 2.0) \tag{7}$$

$$p_f = A_f/(b_w\cdot s_f) \tag{8}$$

where, V_{cd} is the design shear capacity without shear reinforcing steel and FRP reinforcement (N); V_{sd} is the design shear capacity of the shear reinforcement (N); V_{fd} is the design shear capacity of the FRP (N); K is a coefficient representing the shear reinforcing effectiveness of FRP; A_f is the total cross-sectional area of FRP placed in s_f (mm^2); s_f is the spacing of FRP (mm); f_{fud} is the design tensile strength of FRP (N/mm^2); E_f is the modulus of elasticity of FRP (kN/mm^2); α_f is the angle between the FRP and the member axis; z is distance from the location of the compressive stress resultant to the centroid of the tension steel (mm); γ_b is a member factor; f'_{cd} is the concrete design compressive strength (N/mm^2); and b_w is the web width (mm)

The ductility ratio μ_{fd} can be evaluated using formula (9), which is based on experimental results. Here, the ductility ratio is defined as the ratio of horizontal displacement when the horizontal load falls below the yield strength.

$$\mu_{fd} = \left[1.16 \cdot \frac{(0.5 \cdot V_c + V_s)}{V_{mu}} \cdot \left\{1 + \alpha_0 \cdot \frac{\varepsilon_{fu} \cdot \rho_f}{V_{mu}/(B \cdot z)}\right\} + 3.58\right] / \gamma_{bf} \qquad (9)$$

where, V_c is the shear contribution of the concrete (N); V_s is the shear contribution of the shear reinforcement (N); V_{mu} is the maximum shear force when a member reaches its existing flexural load-carrying capacity M_u (N); α_0 is a coefficient used to calculate member ductility ratio (the modulus of elasticity for the lateral ties may be used); ε_{fu} is the ultimate strain of the FRP reinforcement; ρ_f is the shear reinforcement ratio of the FRP reinforcement; B is the member width (mm); and γ_{bf} is a member factor.

3.4 Experiment to verify effectiveness of CF-panel method

In order to verify the shear strengthening and ductility-improving effect of the CF-panel method, cyclic loading tests are carried out on an RC column specimen. As shown in Fig.8, the cross section of the specimen is 600 mm x 600 mm, the loading height is 1,620 mm and the shear span ratio a/d is 3.00. Specifications of the specimen and material test results are shown in Table2. The column itself is designed to fail by shear failure after bending yield of the reinforcement. To add strength, two layers of CF-panels each with a height of 500mm are installed up to a height of 1,000mm from the footing.

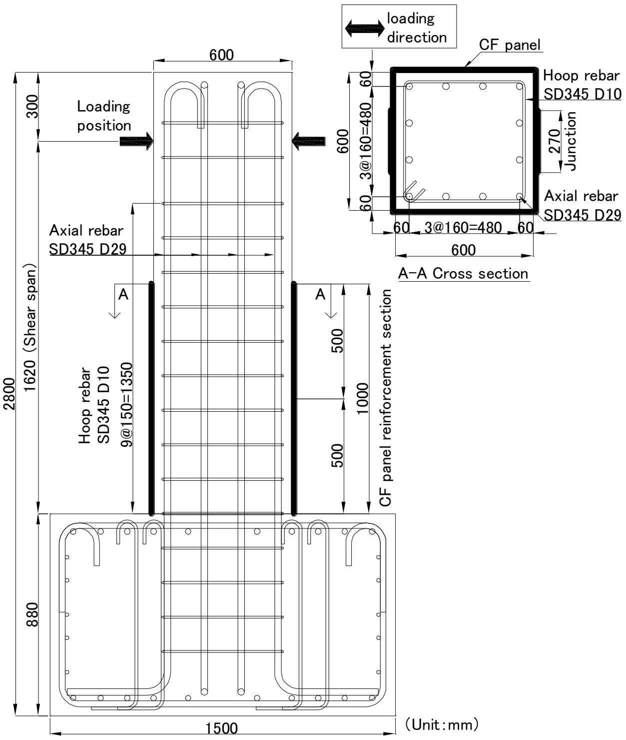


Fig.8 RC column specimen

Table2 Specifications and material test results

Ratio of axial reinforcement	2.14%
Yield strength of axial reinforcement	375N/mm^2
Ratio of hoop reinforcement	0.16%
Yield strength of hoop reinforcement	384N/mm^2
Amount of carbon fiber sheet	1,200g/m^2
Tensile strength of carbon fiber sheet	3,654N/mm^2
Compressive strength of concrete	35.3N/mm^2
Compressive strength of mortar	74.5N/mm^2

Initially, axial compressive force is applied with a vertical jack until the axial stress at the base reaches 3.0N/mm^2. Then a horizontal jack is used to apply cyclic loading with a pushing and pulling motion. The horizontal displacement at the point when the tensile strain in a longitudinal rebar placed on the

Photo6 Experimental setup

tension side at the base reaches the yield strain is defined as the yield displacement, δ_y, and the horizontal force at that moment as the yield load, P_y. Thereafter the maximum displacement at each step is an integer multiple of δ_y on both positive and negative sides and three cycles of loading are performed at each step under displacement control ($\pm 2\delta_y$, $\pm 3\delta_y$, …). Photo6 shows the loading test setup.

The relationship between horizontal load and horizontal displacement is shown in Fig.9. After yielding of the longitudinal rebar placed in the tension side, stable bending deformation behavior develops, with deformation progressing while the horizontal force remains above the yield load. In step $10\delta_y$, swelling is confirmed at the bottom. In step $11\delta_y$, the horizontal load falls below P_y. The carbon fiber sheet suffers partial failure at the lower corner near the compression side in step $13\delta_y$ where the horizontal load finally falls to about 50% of P_y.

The condition of the longitudinal rebar is checked after completion of the experiment by removing the CF-panel near the bottom and the crushed concrete cover. The condition of the column is shown in Photo7. All longitudinal rebars are buckled in the range of about 350 to 400 mm from the footing and a partial fracture has occurred. Crushed concrete is present even deeper inside than the longitudinal rebar.

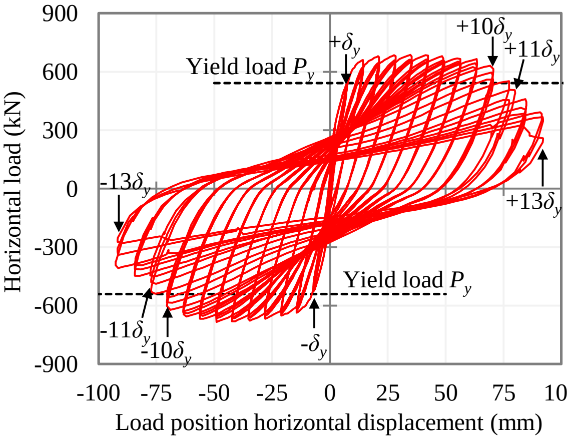
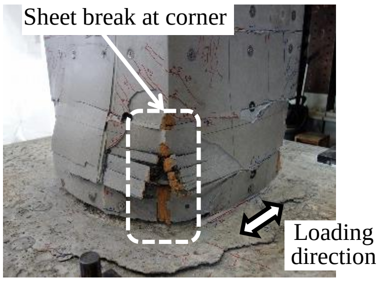


Fig.9 Relationship between horizontal load and horizontal displacement



(a) At the end of the experiment

(b) After removing CF-panel and crushed concrete

Photo7 Damage situation near the bottom

The shear strengthening effect of the CF-panel causes the column to fail in bending failure mode. The ductility ratio calculated by formula (9) using the actual strength of each material is 6.1. The experimental result is 10.2 (average value of positive and negative loads). The experimental results exceed the calculated values, confirming that the improvement in ductility is as predicted from the design calculations.

4. Conclusion

The authors have developed two seismic retrofitting techniques for RC structures that offer excellent workability. They are particularly suited to situations where site conditions are very restricted. One is a shear strengthening technique for wall members using retrofitted plate-anchored reinforcing bars (PHb method), while the other is a shear strengthening and ductility improvement technique for column members using carbon fiber composite panels (CF-panel method). It is clarified through experiments that the proposed strengthening design method is able to predict member performance after strengthening.

In Japan, there is a high probability of the occurrence of large-scale earthquakes in the near future including Nankai Trough Earthquake and Tokyo Inland Earthquake. The authors hope that newly developed techniques contribute to the effective utilization of the nation’s infrastructure and help improve national resilience.

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