Fracture Mechanics of Concrete Structures
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ON THE PREDICTION METHOD FOR THE STRUCTURAL PERFORMANCE OF REPAIRED/RETROFITTED STRUCTURES

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Abstract
This paper emphasizes importance of a research direction related to repair and retrofit of concrete structures. The selection of an appropriate method and amounts of repair/retrofit, and evaluation of its effectiveness are questions to be answered in engineering practice. To realize efficient repair/retrofit, a prediction method for the structural performance of repaired/retrofitted structures is necessary. Two examples of studies towards the development of such prediction methods are introduced. Firstly, an analysis method for the shear capacity of RC column/beam is presented. In Japan, use of reinforced steel fiber concrete (RSFC) column/beam with no tie-hoop and its design method are under consideration. Here we present one of candidate methods to evaluate the shear capacity of RSFC column/beam. The method is extended to evaluate the effect of retrofit with FRP jacket on the improvement of the shear capacity and the results are compared with experimental data. Secondly, we present results of structural performance analysis of a panel made from pseudo strain-hardening fiber-cement composite. The panel is expected to be applied in anti-seismic shear wall retrofitting of RC buildings. The results indicate that use of the new material results in improved ductility and strength of the structural member.
Key words: repair, retrofit, structural performance, fracture mechanics

1 Introduction

What are the most important issues in the repair and retrofit of concrete structures? Identification and modeling of mechanisms leading to the loss of functions of the repair and retrofit must be given the highest priority. Through the identification and modeling of the dominant mechanisms, the parameters for the strength and loading related to the repair/retrofit are introduced.

In the engineering practice, the judgement is always required whether repair/retrofit should be carried out or not, which method should be adopted, how far the repair or retrofit should be carried out. Engineers must make the judgement so that the repaired or retrofitted structures have, for example, a load carrying capacity larger than the required value, and at the same time the cost for the repair or retrofit is minimized. Since the total cost for the repair or retrofit of infrastructures is enormous, a reliable tool for the evaluation of cost performance of the repair or retrofit is necessary for engineers to make such a decision. One of the most important research directions related to the repair/retrofit of concrete structures is the development of analytical methods which can predict the structural performance, such as the load carrying capacity, of the repaired or retrofitted structures.

The analytical method should be based on the mechanism governing the failure of the repair or retrofit. After the identification and modeling of the mechanism, the strength parameters are determined from material tests and the load parameters are calculated by numerical analysis for the prescribed boundary conditions.

We demonstrate the concept discussed above in two examples: prediction method for shear capacity of RC column/beam and analysis of panels to be used as anti-seismic retrofitting shear wall.

2 Shear capacity of RC column/beam

One of the fundamental concepts of RC column/beam design is to keep the shear capacity larger than the flexural capacity to avoid the brittle shear failure and to ensure high ductility under bending mode of deformation. Earlier experiments of reinforced columns under cyclic loading showed that the transition from catastrophic shear failure mode to ductile mode can be achieved by use of SFRC instead of regular concrete with tie-hoops. The reasons for the high ductility are that the fibers increase the column shear capacity which leads to bending mode of failure. Furthermore, they prevent spalling of the covering concrete and buckling of longitudinal reinforcement.

Hence, a practical use of reinforced steel fiber concrete (RSFC) columns/beams with no tie-hoops and their design method are currently under consideration in Japan. To this end it is necessary to be able to evaluate the shear capacity of RSFC columns/beams which varies with the size of the column/beam and the properties of the fiber. In the next subsection, a method for evaluation of the shear capacity, which is proposed for implementation in the design code, is introduced. The method is further extended to evaluate the increase in the shear capacity due to retrofit by FRP jacketing.

2.1 Evaluation of shear capacity of RC column/beam

The shear failure of RC column/beam is governed by the propagation of a "shear crack" which is a tensile crack inclined to the member's axis. Although the final failure is caused by the compressive failure near the ligament ahead of the shear crack, it results from the propagation of the shear crack. In a RSFC column/beam with no hoops, the shear crack propagation is controlled by the stress transmitted by steel fibers across the crack.

To judge whether the rapid propagation of a shear crack occurs before the flexural capacity is reached or not, finite element analyses are carried out. An inclined straight shear crack is considered along which the tension-softening relationship is assumed; see Fig. 1. The location and inclination angle of the shear crack are prescribed for given values of parameters, such as shearing span, based on experimental data and the FEM analysis which does not assume the location of cracks and localization of cracking is judged at each step of the computation (Matsuoka et al., 1998).

The tension softening relationship for SFRC is simplified as a linear function, which is defined by two parameters: critical stress transmitted by fibers and the slope of tension softening curve. These parameters are determined from bending tests of small beams; see Nanakorn et al. (1996) and Nanakorn and Horii (1996).

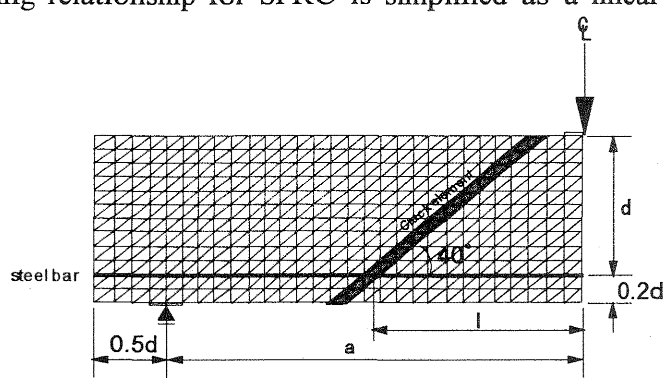

Fig. 1 Finite element mesh for analysis of shear crack growth

Figure 2 shows the result of the finite element analysis for RSFC and RC beams. No tie-hoop is installed in both cases. The variation of the load with the increasing crack depth (crack length projected on the cross section) is presented. In the studied RC beam, the increase in the load is not significant and the load does not exceed the flexural capacity, which implies the occurrence of the shear failure.

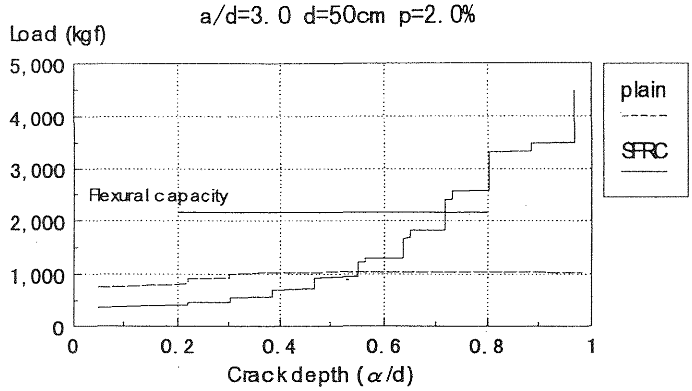

Fig. 2 Load vs. crack depth for RSFC and RC beams

In the RFSC beam, the increase in the load with increasing crack depth is remarkable as the crack depth exceeds one half of the cross section size. The load exceeds the critical load for the bending failure, which ensures the failure in the bending mode.

2.2 Increase in the shear capacity due to FRP retrofit

FRP jacketing is one of methods to increase the shear capacity and the ductility of existing columns/beams. Figures 3 show the process of FRP jacketing to an existing RC column. Glass fibers are cut and mixed with unsaturated polyester sprayed on the column surface. The increase in the shear capacity achieved by the FRP jacketing is analyzed by extending the method explained in the previous subsection. In the two dimensional analysis, the FRP cover is modeled as a sheet overlaid on the column and connected along the both edges

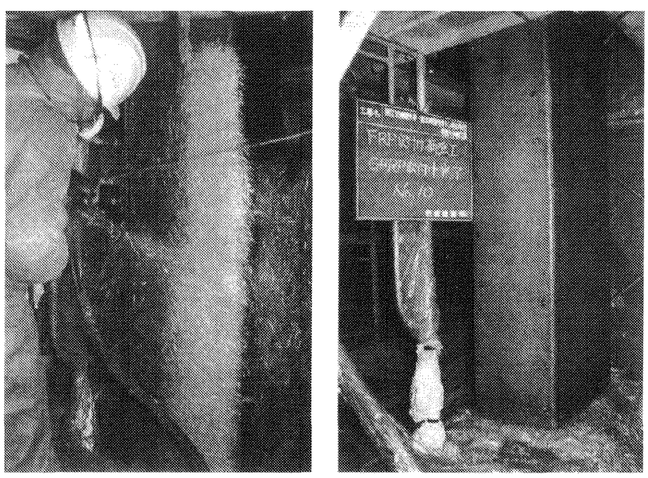

Fig. 3 Process of FRP reinforcement (left) and FRP retrofitted RC column (right).

of the column. This implies that the bond between the FRP cover and the surface of the concrete is neglected.

Figure 4 shows numerical results in terms of load vs. crack depth for an RC column retrofitted with FRP jacket and for an unretrofitted one. Horizontal lines correspond to the critical load observed in experiments. In the experiments, the unretrofitted column exhibited shear failure, while the retrofitted one failed in bending mode. For the unretrofitted column, the load increases almost linearly with the crack depth. Continuous growth of the shear crack with the load less than the flexural capacity indicates the occurrence of shear failure. The computed load exceeds the experimental value when the crack depth is 80% of the column depth. In reality the shear failure occurs due to the compressive failure around the ligament ahead of the shear crack, which is not considered in the numerical analysis.

For the column retrofitted with FRP, the rate of the load to the crack depth increases drastically after the crack depth exceeds about 75% of the column depth, and the load exceeds the maximum load in the experiments. The termination of the shear crack growth indicates the increase in the shear capacity and the prevention of shear failure. In the experiments the shear failure did not occur and the column failed under bending mode.

In Fig. 5, the relationships between the load and the displacement measured in experiments are plotted. The column

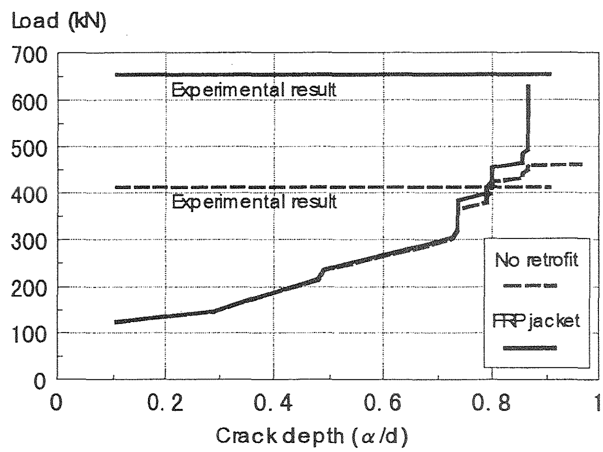

Fig. 4 Load vs. crack depth for RC columns with and without FRP jacketing

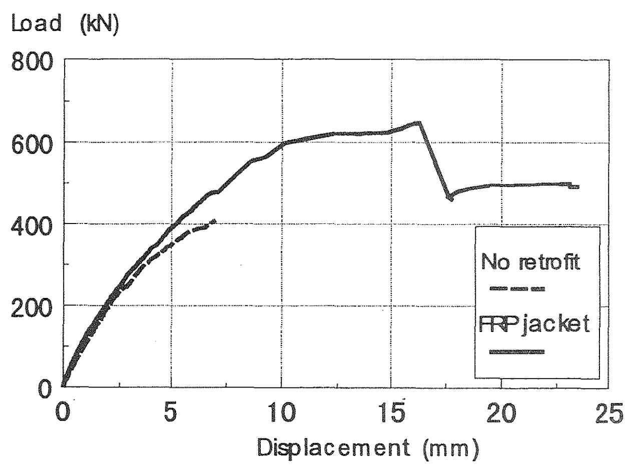

Fig. 5 Load vs. displacement measured in tests

retrofitted with FRP shows high ductility with bending mode. The discontinuous reduction of the load and increase in the deformation at the displacement of 27mm is due to the failure of the FRP cover. FRP retrofit is effective in the improvement of the shear capacity while additional means are required to realize higher ductility. Currently use of wire mesh wrapping is examined, with results indicating improvement of ductility.

It is demonstrated that the fracture mechanics based FEM analysis presented in this paper can predict the increase in the shear capacity by the FRP jacketing. This method provides a tool to realize the optimal design of repair/retrofit.

3 Anti-seismic retrofit by structural shear wall made from pseudo strain-hardening cementitious composite

3.1 Anti-seismic retrofit method

A retrofit method is being considered (Kanda et al., 1997), in which a seismic resistance of a multistory RC frame structure is improved by enhancing its ability to dissipate energy. To this end, a light ductile structural shear wall is constructed between columns and beams of the existing structure (Fig. 6a). The wall is to be designed so that under a strong seismic load it undergoes inelastic deformation without losing load-carrying ability. Energy is dissipated during this inelastic process, which prevents damage to the original frame structure. After fulfilling its function, the damaged shear wall is removed and replaced by a new one.

In order to facilitate the construction and replacement process, the wall consists of an assembly of prefabricated panels (Fig. 6b), connected to each other by bolted steel plates and to the original RC structure by steel

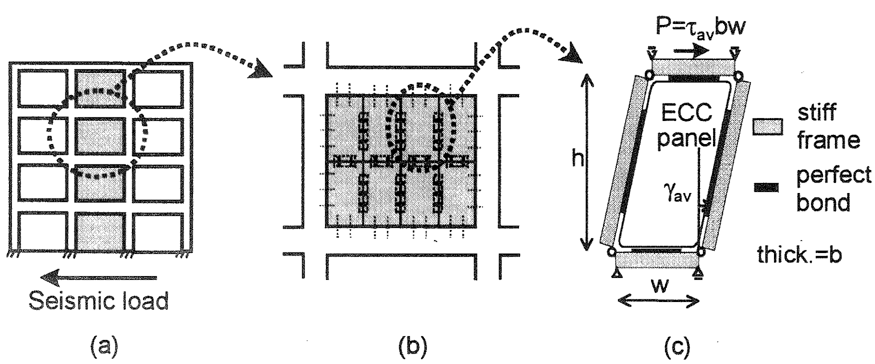

Fig. 6 Retrofit shear wall and its simplified model

dowels (Kanda et al., 1997).

A novel material, called Engineered Cementitious Composite (ECC), will be used to manufacture the shear wall panels. ECCs consist of cement-based matrix reinforced by a relatively small amount of short random fibers. A major advantage of ECCs is that their mechanical properties can be controlled by means of micromechanics-based design (Li, 1997). For example, using polymeric fibers, composites that exhibit extensive multiple cracking and pseudo strain-hardening behavior prior to softening and fracture localization under tensile and shear loads have been developed.

As discussed in the previous section, even use of a quasi-brittle SFRC, which lacks the multiple-cracking ability, greatly improves shear resistance of RC columns/beams, permitting to reduce or even eliminate conventional shear reinforcement. When using ECC, even better performance can be expected. Therefore, it is considered that the retrofit shear wall panels are made of plain ECC without any conventional steel reinforcement.

3.2 Finite element analysis

3.2.1 Computational model

Kabele et al. (1997) investigated the mechanical performance of the retrofit by means of finite element analysis. They assumed that under seismic load, each of the panels that formed the shear wall was undergoing the same overall shear deformation γ_{av}, which was equal to the relative floor drift γ_f. Thus, only one of the panels was analyzed (Fig. 6c), whereas the effect of the surrounding RC structure and other panels was represented by a stiff hinged frame. The joint steel plates and dowels were represented by perfect bond between the ECC panel and the surrounding frame. A monotonic loading was assumed.

Considering that the dominant mechanism of ECC mechanical behavior is the distributed multiple cracking followed by fracture localization, an analytical model developed by Kabele and Horii (1997) was utilized to represent the ECC's behavior under general bi-axial stress state. In this model, ECC in the tensile multiple cracking state is represented as a continuum undergoing inelastic deformation (cracking strain). Incremental theory of plasticity with Rankine yield function and linear kinematic hardening are employed. A discrete crack model with a linear tension-softening relationship is used for localized cracks that occur once the material's tensile strength is reached. ECCs' response under compression is represented by plasticity with von Mises yield function and isotropic hardening rule. The post-yield stress-strain relationship is approximated by a bi-linear curve, with initial hardening followed by a perfect plasticity. It is assumed that once the equivalent plastic strain associated with the von Mises yield function attains critical value ε_{cr}^{p} the material cannot bear any load. Under combination of tension and compression, it is assumed that both inelastic phenomena – tensile cracking and compressive plasticity – exist simultaneously but independently of each other. This is ensured by keeping the cracking strain and the von Mises plastic strain as independent variables.

3.2.2 Mechanical performance of the ECC panel

The analyses by Kabele et al. (1997) revealed that the ECC panel exhibited extensive tensile multiple cracking distributed over three wide diagonal bands (Fig. 7). Although not shown in the figure, compressive yielding of ECC took place in small areas at the ends of both horizontal and vertical joint plates. The computation was terminated when the equivalent compressive plastic strain in one of these areas reached the critical level ε_{cr}^{p}, interpreting this state as a compressive failure. The maximum (tensile) cracking strain at that moment was about 1% near the ends of vertical joint plates and about 0.4-0.6% in most of the cracked regions. These cracking strain levels were far bellow the tensile strain capacity of the ECC used (about 5%), indicating that no softening localized cracks occurred. In contrast, analysis of an identical panel made from a quasi-brittle SFRC revealed failure due to formation of large tensile and shear cracks with no compressive yielding (Fig. 8).

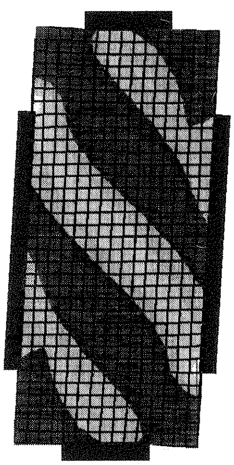

Fig. 7 Deformed shape and cracking strain distribution of ECC panel with steel plates/dowels along vertical and horizontal joints

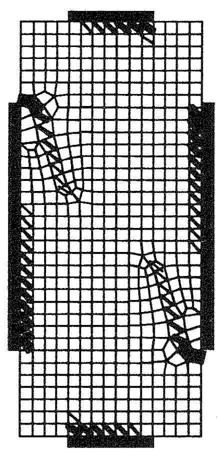

Fig. 8 Discrete cracking pattern in quasi-brittle SFRC panel

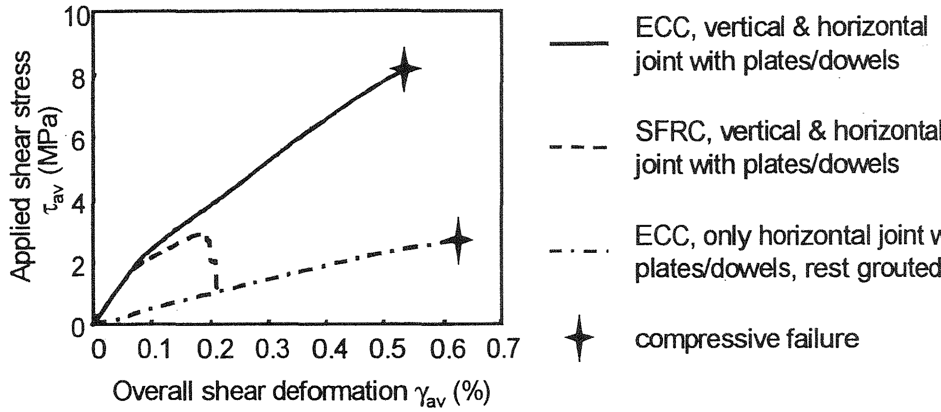

Fig. 9 Computed overall shear stress-overall shear deformation curves for retrofit panels

Fig. 9 shows that the ECC panel outperformed the SFRC one in terms of both strength and ductility. Furthermore, it can be expected that the ECC panel would perform well even under cyclic shear loading. The reason is that the tensile cracking occurs in form of distributed fine cracks (multiple cracking) and the shear panel does not lose its overall integrity.

3.2.3 Effect of ECC compressive ductility on panel performance

The analysis presented in the previous sub-section showed that failure occurred due to exhausting the ECC's compressive strain capacity. It was assumed that the failure strain, ε_{cr}^{p}, was constant equal to 0.2%. However, it is likely that when the material undergoes multiple cracking under biaxial tension-compression, the compressive capacity is reduced. On the other hand, it is possible that by further engineering the composite, its compressive ductility could be enhanced. Thus, it is of interest to know how much the panel's structural performance changes when the material compressive strain capacity is reduced or increased.

To that end, the analysis discussed in sub-section 3.2.2 is extended, assuming that ECC does not fail at the critical compressive plastic strain ε_{cr}^{p}, but instead, it keeps on yielding at constant stress level (perfect plasticity). Fig. 10 shows that if the maximum compressive strain that the material can sustain were reduced to one half, the overall ductility of the panel (overall shear deformation when the compressive strain capacity is reached) would decrease by about 12%. On the other hand, to increase the ductility of the

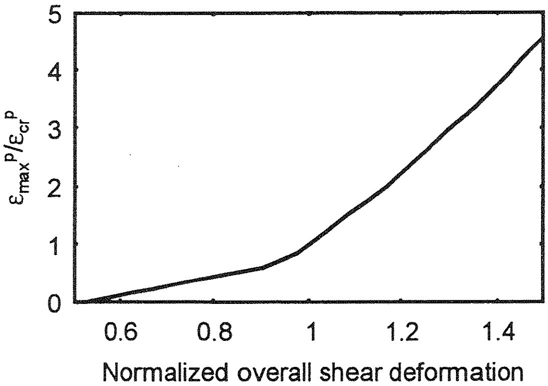

Fig. 10 Relationship between the maximum compressive plastic strain and the overall shear deformation

panel by 50%, the material's compressive strain capacity would have to be increased almost 4.5-times.

3.2.4 Effect of joint configuration on panel failure mechanism

The failure mechanism of the shear wall panel can be altered not only by varying the properties of the utilized material but also by changing the configuration and detailing of the construction joints. In addition to the configuration discussed in the previous sub-sections, where jointing steel plates/dowels were used on all four sides of the panel, a case with steel plates/dowels along the horizontal joints only was analyzed. It was assumed that the vertical joints and corner portions of the horizontal joints were filled by grout capable of transmitting only normal compressive stress.

Fig. 11 shows the computed deformed shape and tensile cracking strain (multiple cracking) distribution for the latter joint configuration. It is seen that tensile cracking was concentrated near the panel top and bottom, with localized cracks propagating along the horizontal joint plates. Local compressive failure occurred near the left end of the upper joint plate and near the right end of the lower joint plate. As Fig. 9 indicates, the panel failed at almost the same overall deformation level but much lower load, than the panel with steel plates/dowels along all four sides.

3.3 Conclusions drawn from computational results

The present analyses indicate that ductile anti-seismic retrofit elements can by designed using pseudo-strain hardening cementitious composites, such as ECC. It was observed that if formation of localized tensile cracks is delayed by the composite hardening ability, the member's performance is ultimately controlled by the compressive strength and strain capacity of the used material. Thus, further engineering of the composites toward enhancement of these properties is suggested. It is also demonstrated that the retrofit members should be configured so that the main inelastic process – tensile multiple cracking – takes place in a large volume of the element.

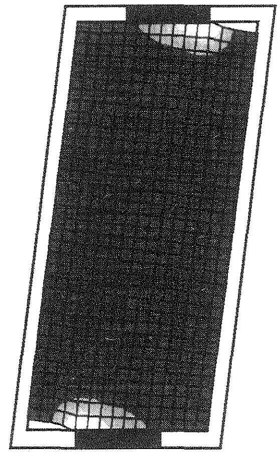

Fig. 11 Deformed shape and cracking strain distribution of ECC panel with steel plates/dowels along horizontal joints and grouted vertical joints

4 Concluding remarks

The two examples introduced in the present paper demonstrate the importance of approaching the problems of repair/retrofit from a global, structural point of view, as opposed to focusing on a limited area, such as a structural detail or a single material property. With the presented approach, researchers and engineers are able to identify phenomena that have dominant effect on the performance of the repair/retrofit and deserve investigation that is more detailed.

In the first example, the bond between FRP jacket and concrete was neglected. Its importance will be clarified by comparing numerical results with and without the bond. If the bond effect is significant (which may not be the case in this example), the research target is specified.

The second example demonstrated that both structural detailing and complex material properties decide the effectiveness of the retrofit method. The analyses furthermore revealed the material property (compressive ductility, in this case), which had a dominant effect on the ultimate structural performance of the retrofit, and therefore should be further studied and improved.

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