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USE OF BMC FOR DUCTILE STRUCTURAL MEMBERS

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ABSTRACT

Application of Brittle Matrix Composite (BMC) for anti-seismic retrofit of reinforced concrete structures is considered. By means of numerical simulation we examine structural performance of a new type of BMC, called Engineered Cementitious Composite (ECC). The analysis shows that using an ECC that has been tailored for high tensile ductility leads to improved performance of retrofit elements loaded by intense shear.

INTRODUCTION

Brittle matrix composites (BMC) have been in the spotlight of interest of material scientists and engineers for over a decade, and great progress has been achieved. For many reasons, short fiber cementitious composites with moderate fiber volume fraction (between 1 to 2%) have been of particular interest to civil engineers. These reasons include ease of production and processing, isotropic mechanical properties and, compared to concrete, improved fracture toughness, fatigue resistance, modulus of rupture etc. Another important feature of these materials that has been recently explored, is the possibility to alter their mechanical properties by changing their composition. As an example, one can name the Engineered Cementitious Composites (ECCs) recently developed by researchers at the ACE-MRL at the University of Michigan [1]. These materials have been, by means of a conscious micromechanical design, tailored for pseudo strain-hardening behavior under tensile and shear loading.

In spite of the achievements in the material development, the number of applications of these new composites in structural members in the field of civil engineering and architecture has been, so far, very limited. Therefore, the task facing researchers and engineers now, is to demonstrate that the excellent properties of materials like ECCs can be successfully translated to a structural level, and that these materials can be, despite their higher cost, efficiently utilized in civil infrastructure or building structures. Furthermore, it can be expected that, once a particular application is studied in more detail, new requirements for material performance may emerge. This would be a valuable feedback to material engineers, giving them a direction, where to focus further attention in material development.

In the present paper, we consider a structural application of short fiber cementitious composites for anti-seismic retrofit of existing multistory reinforced concrete (RC) frame structures. The structural performance of an ECC type pseudo strain-hardening composite is examined by means of numerical simulation.

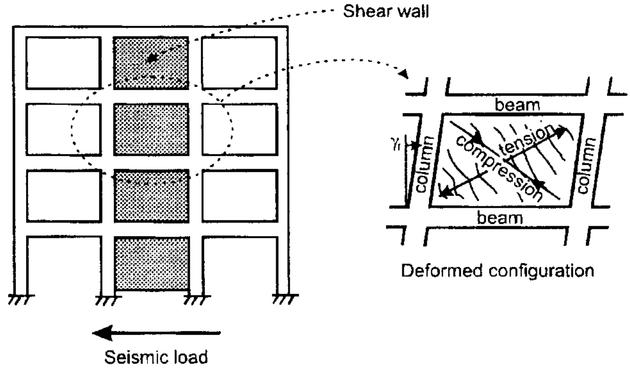

Figure 1 Multistory RC structure and retrofit shear wall

STRUCTURAL REQUIREMENTS AND MATERIAL CHOICE

Anti-seismic retrofit method

Extensive damage to older reinforced concrete buildings during recent strong earthquakes both in Japan and the U.S. highlighted the need for improving the seismic performance of such structures. It is usually necessary not only to increase their strength, but also improve their ductility and ability to dissipate energy in case of a strong earthquake. Furthermore, it is often desirable for economic and other reasons that such a retrofitting is done in short time and without heavy construction work.

A retrofit method is now being considered [2], in which a light but ductile structural shear wall is constructed between columns and beams of an existing multistory RC frame structure. Under horizontal seismic loads, the frame structure deforms and induces shear loading into the wall. The wall acts as braces in the frame structure, with one diagonal subjected to compression and the other to tension. It is expected that, if a strong earthquake occurs, the wall undergoes inelastic deformation, namely cracking normal to the tensile diagonal (see Figure 1). Energy is dissipated during this inelastic process, which further improves the seismic resistance of the whole 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. The panels are connected to each other and to the original RC structure by frictional and dowel joints, respectively.

Structural requirements

Older structures that need to be retrofitted have usually a low deformation capacity; we consider that they cannot sustain relative floor shear displacement γ_f (see Figure 1) larger than about 1%. From this observation we can draw the requirement that the ultimate ductility of the wall in terms of γ_f should be around 1%. The tensile cracking (and thus the energy dissipation mechanism) should be activated at γ_f much lower than 1%. Yet it should not start too early, as even a small earthquake or a wind load would result in necessity to replace the retrofit structure. In terms of strength, the target is to match the shear strength of a monolithic RC shear wall, which is estimated to be around 3MPa.

At the same time, the stiffness of the shear wall should not be very high. High stiffness would result in load concentration in the wall itself and in the structure around it, possibly leading to damage of the surrounding structure that we intend to protect.

Considering the load carrying mechanism of the shear wall shown in Figure 1, it is obvious that once tensile cracking occurs, most of the load imposed on the wall is carried by the compressive diagonal. The wall strength is then governed by the strength of this diagonal. If a few large throughout-thickness tensile cracks form, they may not be perfectly parallel to the maximum compression direction, thus weakening the compressive struts. The cracks may even propagate along the wall boundary cutting through the struts and causing failure. It is therefore desirable that the tensile damage occurs in a form of large number of fine discontinuous cracks.

As the wall is to resist cyclic seismic load, it has to maintain as much as possible integrity under load reversals. For this reason, it is also preferable that the damage to the wall is not localized into continuous cracks but is distributed over a large area.

It is also necessary to consider the presence of the construction joints in the wall. It may be expected that, despite careful detailing of the joints, stress concentration may occur in their vicinity, which may lead to initiation of cracking.

Choice of material

A standard material used in structural shear walls is reinforced concrete. However, its application in the present case, when the wall consists of prefabricated panels, has some difficulties. The main problem could be the presence of the construction joints, as stress concentration in their vicinity could cause a local failure. The cracking behavior in the uniform shear stress field is also not favorable. Even in the presence of steel reinforcement, continuous large cracks occur under tension and the material integrity is maintained only through the steel bars. The steel yields in the vicinity of the cracks and upon load reversal buckles, causing spalling of the concrete cover and disintegration of the panel.

Considering the properties of short fiber cementitious composites described earlier, these materials appear as possibly good candidates to be used for the shear wall.

Regarding their tensile behavior, these composites can be divided into two groups: quasi-brittle and pseudo strain-hardening. An example of the quasi-brittle composite can be an ordinary steel fiber reinforced concrete (SFRC). This material, when exposed to tensile loading, usually fails due to formation of a single crack at the load equal to the first crack strength. The failure is not brittle, though, as fibers bridging the crack ensure that traction is still transmitted across the fracture plane. This traction diminishes gradually with increasing crack width. Such a behavior is called tension softening.

In pseudo strain-hardening composites, such as ECCs, the fiber-matrix system is designed so that after the first crack occurs under tensile loading, the bridging fibers are capable of transmitting increasing traction across the fracture plane as the crack opens. This allows further increase in the applied load, which in turn triggers formation of additional parallel cracks in a process called multiple cracking. The composite in the multiple cracking state does not disintegrate, but the cracked matrix is still held together by fibers bridging the cracks. When the ability of the fibers to carry increasing load is exhausted on a certain crack plane (that, which has registered the largest opening displacement), tension softening takes place and the fracture becomes localized on this single crack plane. With a fiber volume fraction as low

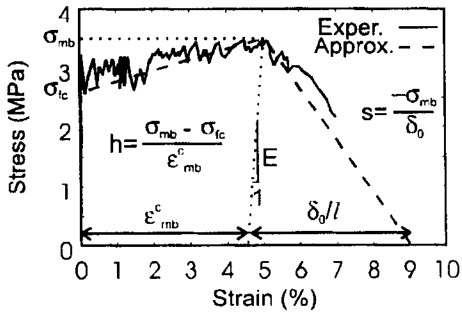

Figure 2 Uniaxial tensile stress-strain curve for PVA-ECC

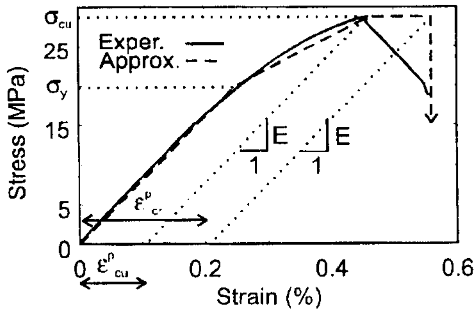

Figure 3 Uniaxial compressive stress-strain curve for PVA-ECC

as 2%, Li [1] has designed ECC that was capable of sustaining increasing load up to overall tensile strain of more than 5%.

As there are two cracking regimes (distributed hardening cracking and localized softening cracking), fracturing ECCs dissipate a large amount of energy, which results in a high fracture energy that ranges in tens of kJ/m^2.

After reviewing the above properties, it can be expected that ECCs should perform well if used for the shear wall panel. Their ability to undergo multiple cracking should result in a favorable cracking pattern with fine discontinuous cracks distributed over large volume of the panel. Their hardening behavior under tension should also make them less sensitive to stress concentration by delaying formation of localized cracks. ECC's high fracture energy should ensure a high effectiveness of the retrofitting structure as far as energy dissipation is concerned. The composites' ductility should be reflected in high ductility of the shear wall.

In order to verify these expectations, we examine the performance of an ECC shear wall panel by finite element method.

ANALYSIS

Material model

Before analyzing the structure, it is necessary to define a material model. The model has to reflect the dominant mechanisms that govern the material behavior. Due to low fiber volume fraction, short fiber length, and random fiber distribution and orientation, an uncracked ECC may be modeled as a homogenous isotropic material. The inelastic tensile behavior is attributed to multiple cracking in the hardening regime, and later formation of localized softening cracks.

As the cracks in the hardening regime have sub-millimeter widths and spacing of less than 1cm, it is not possible to model each crack separately. Instead, we model the composite in the multiple cracking state as an equivalent continuum with additional strain, called cracking strain, which represents the crack spacing and openings. Experimental studies suggest that the hardening cracks almost do not close, which means that the cracking strain is inelastic. Consequently, we employ incremental theory of plasticity. Based on experimental observations, we consider that a set of multiple cracks is formed in ECC on planes normal to the direction of maximum principal stress, when its magnitude reaches the first crack strength σ_{fc}. This is represented by employing the Rankine yield function. This model implies that

Table 1 Material parameters

Mat'l #	E (MPa)	ν (-)	σ_{fc} (MPa)	h (MPa)	ε^{c}_{mh} (%)	s (MPa/mm)	σ_{y} (MPa)	σ_{cu} (MPa)	ε^{p}_{cu} (%)	ε^{p}_{cr} (%)
1	8,000	0.2	2.6	0.18	5.0	-0.583	20.0	28.7	0.1	0.2
2	8,000	0.2	2.6	–	0.0	-0.433	20.0	28.7	0.1	0.2
3	8,000	0.2	2.6	0.18	5.0	-0.583	20.0	28.7	0.1	∞

several sets of multiple cracks can occur at the same point, each oriented at different angle. We use a kinematic hardening rule in order to reflect the assumption that initiation and response of a set of multiple cracks oriented in a certain direction is affected neither by the normal stress acting in the direction parallel to the cracks nor by multiple cracking in perpendicular direction.

The hardening relationship between the cracking strain and the stress normal to the crack planes is obtained from a uniaxial tension test, and is approximated by a straight line with constant slope h (see Figure 2).

The response of a set of multiple cracks changes to softening when the cracking strain reaches a critical level, denoted as ε^{c}_{mh}. This value is also determined from a uniaxial tension test as the cracking strain at the peak load. It is assumed that upon entering the softening regime, fracture becomes localized into a single crack, response of which is controlled by tension softening relationship of normal bridging stress and normal crack opening displacement. This relationship is obtained from the post-peak part of the uniaxial stress-displacement curve. As shown in Figure 2, the tension softening relation can be approximated by a straight line with slope s. As the localized cracks are typically few in number and exhibit large opening displacements, we represent them using a discrete crack model. To this end we employ a finite element with embedded displacement discontinuity. A detailed description of the analytical model is provided in [3].

Note that for a quasi-brittle composite, which lacks the presence of hardening multiple cracking, we employ only the softening crack model. The criterion for initiation of cracking is then that of the maximum principal stress being equal to the first crack strength. The crack direction is normal to the maximum principal stress direction.

ECCs exhibit a moderate hardening under uniaxial compression (Figure 3). We model this behavior by strain-hardening plasticity. The stress-strain relationship is approximated by a tri-linear curve, defined by yield strength σ_{y}, compressive strength σ_{cu}, plastic strain when σ_{cu} is reached denoted as ε^{p}_{cu}, and plastic strain when crushing occurs ε^{p}_{cr}. It is assumed that once the crushing strain is reached, the material cannot bear any load. Currently, experimental knowledge about ECCs' behavior in multiaxial stress state is very limited [4]. For this reason and for the sake of simplicity, we assume that the behavior under multiaxial compression can be described by Von Mises yield function and isotropic hardening rule.

Under combination of tension and compression, we assume, again for the lack of experimental information, that both inelastic phenomena – tensile cracking and compressive plasticity – can exist simultaneously but independently of each other. This is ensured by keeping the cracking strain and the Von Mises plastic strain as independent variables. The reduction of compressive strength by lateral tension is reflected by the shape of Von Mises yield surface, but the effect of lateral compression on tensile response of the cracked material is neglected.

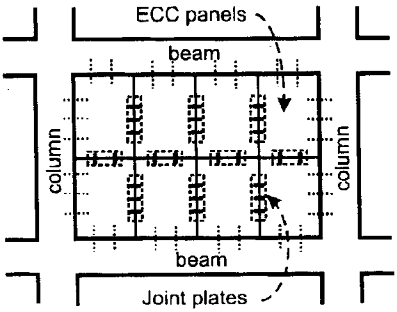

Figure 4 Anti-seismic retrofit shear wall

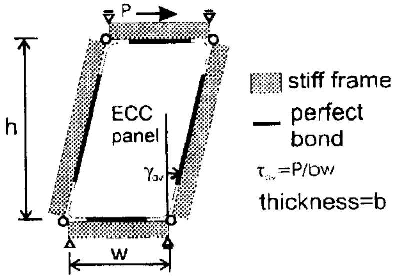

Figure 5 Simplified model of shear wall panel

Material parameters

An ECC consisting of cement paste, 2% by volume of PVA fibers, and additives has been developed particularly for use in the shear wall panel [2]. The experimental uniaxial load-displacement curve of this material is shown in Figure 2 (solid line). From this curve we obtained the material parameters, which are listed in Table 1 (Material #1). Under uniaxial compression the ECC exhibited a behavior shown in Figure 3; corresponding material parameters are also given in Table 1.

Structural model

We consider that a retrofit shear wall consisting of several panels has been constructed between columns and beams of an RC structure, as indicated in Figure 4. We assume that due to horizontal earthquake load, the RC frame undergoes a shear deformation as indicated in Figure 1. Such an assumption is close to the configuration of a scaled-down structural experiment that is to be carried out in the near future [2], so the analytical results may serve as a prediction for the experimental output. This simplifying assumption also allows us to analyze only one of the panels, while the effect of the surrounding RC structure and other panels is represented by a 'frame' of very stiff elements, as shown in Figure 5. Note that in this arrangement, the average shear strain γ_{av} is equal to the relative floor shear displacement γ_f. The panel dimensions (height h=680 mm, width w=300 mm and thickness b=75 mm) are also chosen so as to agree with the planned experimental setup. In the present analysis we assume that no reinforcing steel bars or mesh are used.

The panels are connected to each other through frictional joints [2]. The joints are realized by steel plates protruding into the panel along its center plane, as indicated in Figure 4. The whole joint is clamped by prestressed bolts. The joints between the panels and the RC structure are realized by steel dowels. Both types of joints are modeled by assuming a perfect bond between the panel and the rigid frame along parts of the panel boundary, as shown in Figure 5.

We are interested in evolution of cracking, failure mode, and estimation of the panel overall shear strength τ_{av}^{max} (maximum average shear stress τ_{av}) and ductility γ_{av}^{max} (average shear strain γ_{av} when τ_{av}^{max} is reached). Consequently, we consider only monotonic loading.

The problem is discretized by a uniform mesh consisting of 608 quadrilateral 4-node isoparametric elements and solved by the finite element method using the material model described earlier.

Performance of ECC panel

Computed overall behavior of the panel in terms of average shear stress τ_{av} and average shear strain γ_{av} is shown by the solid line in Figure 6. The graph shows initially linear behavior up to τ_{av} equal to about 2 MPa (Point A). Figure 7 suggests that the slope change of the τ_{av}-γ_{av} curve is attributed to tensile cracking along the construction joints. It should be noticed, though, that the cracking is not localized but spreads away from the locations of tensile and shear stress concentration. No compressive yielding occurs at this loading stage.

Figure 8a shows that further increase in load causes formation of three diagonal bands of distributed tensile cracks. These cracks are still in the hardening regime. Figure 8b shows that at this loading stage compressive yielding starts in isolated areas due to stress concentration near the ends of the joint plates. Figure 8c shows that most of the load is carried by the compressive diagonals I-IV, II-V and III-IV. The fact that the slope of the τ_{av} -γ_{av} curve is almost constant (point B in Figure 6), suggests that stiffness of these diagonals is not much affected by the evolution of distributed tensile cracking.

Failure of the panel occurs suddenly at a load level τ_{av}^{max} equal to 8.11 MPa and γ_{av}^{max} equal to 0.53%, as indicated by point C in Figure 6. It is seen in Figure 9a that the failure is attributed to local compressive crushing of ECC near the top end of the left vertical joint and bottom end of the right vertical joint. Figure 9b shows that since loading level B, the tensile cracking did not much evolve spatially, still taking place mainly in the three diagonal bands, but increased in intensity. However, the maximum principal cracking strain reaches only about 1%, which is well bellow the critical value ε^{c}_{mb} =5%, at which localized softening crack occurs. Therefore we can claim that tensile cracking occurs in the form of distributed discontinuous fine cracks. These cracks are bridged by fibers, which still provide the cracked material with ability to carry increasing tensile load.

Performance of quasi-brittle material

In order to demonstrate that the panel behavior described in the previous section is a result of using pseudo strain-hardening ECC material, we analyzed an identical panel, which, however, was made of a quasi-brittle composite. For the sake of comparison, we assumed that elastic properties and compressive behavior of both materials were identical. The only difference was in tension, where the quasi-brittle material lacked the hardening branch and was assumed to exhibit tension softening and discrete crack immediately after reaching the first crack strength σ_{fc}. Material parameters for the quasi-brittle composite are listed in Table 1, Material #2.

The quasi-brittle panel was analyzed using a finite element mesh consisting initially of 616 quadrilateral 4-node isoparametric elements. In the loading process, some of these elements were automatically changed into elements with displacement discontinuity to represent formation of softening cracks. It should be noted that the mesh was constructed so as not to constrain propagation of the discrete softening cracks, path of which was earlier estimated by using a smeared crack model.

Figure 6 shows that after the first bend-over point, the quasi-brittle panel showed much softer behavior than the ECC panel. It is seen in Figure

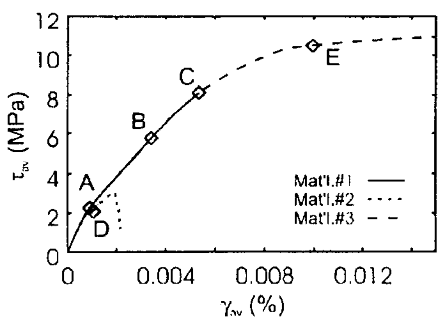

Figure 6 Computed τ_{av}-γ_{av} curves

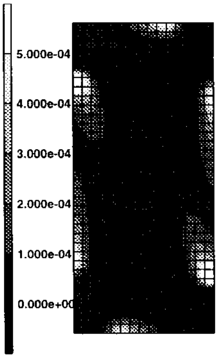

Figure 7 Max. cracking strain (Mat'l#1, load level A)

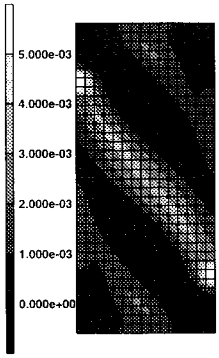

Figure 8a Max. cracking strain (Mat'l#1, load level B)

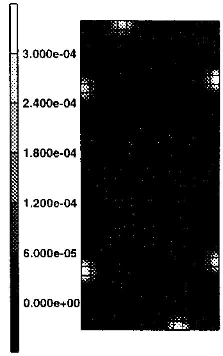

Figure 8b Eq. plastic strain (Mat'l#1, load level B)

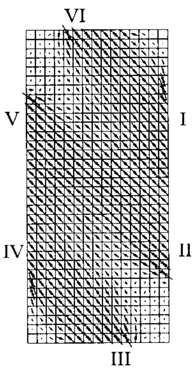

Figure 8c Principal stresses (Mat'l#1, load level B)

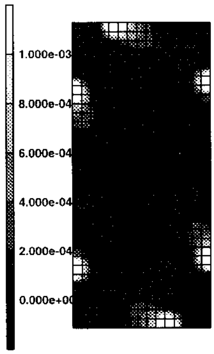

Figure 9a Eq. plastic strain (Mat'l#1, load level C)

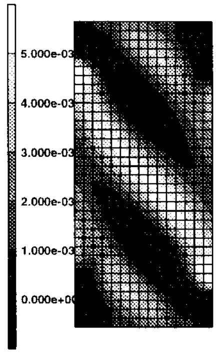

Figure 9b Max. cracking strain (Mat'l#1, load level C)

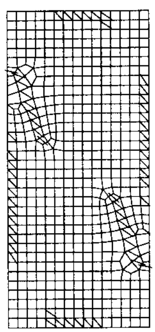
Figure 10 Softening cracks (Mat'l#2, load level D)

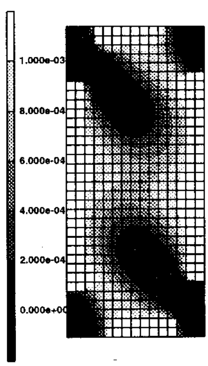

Figure 11a Eq. plastic strain (Mat'l#3, load level E)

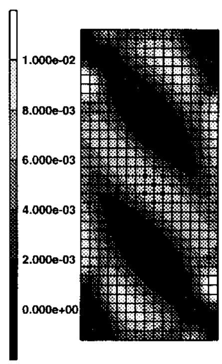

Figure 11b Max. cracking strain (Mat'l#3, load level E)

10 that the change in slope of the load-displacement curve at load γ_{av} equal to about 2 MPa can be attributed to formation of six localized shear cracks propagating along the joints and diagonally toward the center of the panel. The panel failed due to extension and interconnection of these cracks at τ_{av}^{max} equal to 3 MPa and γ_{av}^{max} equal to 0.19%. No compressive yielding occurred up to failure.

Performance of ECC with enhanced compressive ductility

The analysis of the ECC shear panel showed that structural failure occurred due to exhausting the material compressive strain capacity. 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 compressive strain capacity should be increased and what would be the panel performance like. In order to provide answers to these questions we conducted the following analysis.

The panel configuration, boundary conditions and discretization are the same as those used in the earlier analysis with Material #1. It is assumed that the panel is made of Material #3 (see Table 1), which has identical characteristics as the pseudo strain-hardening Material #1, with the exception of the compressive crushing strain ε'_{cr}. The value of is ε'_{cr} is now considered to be infinite, that is, we assume that the material exhibits unlimited perfectly plastic response upon compressive stress reaching the level of σ_{cu}.

The overall behavior of the panel is shown in Figure 6. It is seen that up to the point C, the curve is identical to that obtained with Material #1. When Material #3 is used, the panel can sustain further load increase, even though the slope of the τ_{av}-γ_{av} curve is decreasing. This change is attributed to expansion of the areas where compressive yielding takes place. It is seen that the curve more or less flattens out at γ_{av} of 0.9%. Figure 11a shows that this is a result of complete yielding of the compressive diagonals. The maximum equivalent plastic strain at this loading level, which is marked by point E in Figure 6 and corresponds to the target value of γ_{av} = 1%, is 2%. The point where the maximum compressive straining occurs is again near the ends of the vertical joints. Figure 11b indicates that the tensile cracking does not expand over the whole panel, but occurs within the three diagonal bands. The maximum principal cracking strain at load E is 1.8%, which is about one third of ε^{t}_{mh}, meaning that the material still has an ample tensile strain capacity.

SUMMARY OF RESULTS AND CONCLUDING REMARKS

The analyses showed that the ECC panel exhibited extensive tensile microcracking distributed over a large volume. As a result of the material tensile hardening behavior, tensile and shear stress concentration near the construction joints was diffused, which led to significant delay in formation of large localized crack. In fact, the panel failed due to compressive crushing near the joints before any localized tensile crack occurred. On the contrary, when a quasi-brittle material was used, localized shear cracks formed at very early loading stage. Propagation of these cracks along the joints and diagonally toward the center of the panel led to failure at load and displacement level much lower than when ECC was used. No compressive yielding took place in the quasi-brittle panel.

The ECC panel failed at average shear load τ_{av}^{max} equal to 8.11 MPa and average shear strain γ_{av}^{max} equal to 0.59%. While the strength is larger than the target value of 3MPa, the panel does not satisfy the required ductility of γ_{av}=1%. The main reason is that by delaying the formation of localized cracks, failure mode shifts to compression. Although the ECC developed for this application exhibits about 1.5-times higher crushing strain than conventional concrete, it is not high enough to allow the panel to undergo the desired deformation before a compressive failure occurs. It should be also noted that when compressive failure occurred, the maximum tensile strain was only about 1/5 of the tensile strain at which softening and fracture localization occurs, meaning that the composite tensile capacity was underutilized.

The analysis of Material #3 showed that, should the desired ductility of the panel be attained, the ECC would have to be capable of at least perfectly plastic behavior up to compressive plastic strain level of about 2%. However, even then the structural failure would be governed by compression.

It should be noted, though, that the above results have been obtained under simplified loading conditions. It was assumed that the shear wall underwent a pure shear deformation and that the load from the surrounding structural members was transmitted to the panel through perfectly rigid joints. It is probable that in reality the wall will be subjected to a combination of shear and bending, which may result in more tensile straining and thus more ductile overall response. The actual joints are also not perfectly rigid, but allow some deformation to take place. This may add to the flexibility of the whole panel assembly, and also reduce the stress concentrations around the joint plates. More precise analyses are now being carried out to prove these hypotheses.

From the analytical results presented in this paper we can conclude that beneficial effect of using ECC for the shear wall panel is mainly in delaying localized tensile cracking. Due to this phenomenon, even unreinforced ECC panel fails in compression. In order to further increase the panel ductility, it is therefore necessary to enhance the composite compressive strain capacity.

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