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ELEVATING MATERIAL DUCTILITY TO STRUCTURAL PERFORMANCE OF STEEL ANCHORING TO ECC

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Abstract

Concrete fracture failure is often observed at steel/concrete interaction zones, such as steel anchoring to concrete and hybrid structure connections, under mechanical loading. Steel anchoring to concrete is the focus of this investigation due to its wide application in civil infrastructure. The use of High Performance Fiber Reinforced Cementitious Composites (HPFRCC) at anchoring zone should suppress such failure mode and enhance the overall structural performance. However, this proposed material solution must be demonstrated and appropriate design equations should be provided before its adoption in practice. In this article, a class of HPFRCC -- the micromechanically designed Engineered Cementitious Composite (ECC) with a tensile ductility around three hundred times that of normal concrete is shown to eliminate the brittle fracture mode by developing extensive multiple micro-cracks in ECC. This modification in behavior led to higher load capacity and structural ductility, thus enhancing overall structural response. A preliminary design equation for the shear capacity of anchors in ECC accounting for material ductility of ECC will be derived. The enhancement in structural response through material ductility engineering is expected to be applicable to a wide range of engineering structures where steel and concrete comes into contact.

1. INTRODUCTION

Fracture of concrete is a dominant failure mechanism when steel anchor and concrete interact mechanically due to the brittleness of concrete and high stiffness of steel. This undesirable failure mode can also occur in other steel/concrete interaction zones, such as hybrid steel/concrete structures involving steel beams which penetrate into concrete columns. This article will try to address fracture failure problem of steel anchoring to concrete. A large number of RILEM round robin study of steel anchor bolt pullout from concrete [1] demonstrate experimentally and numerically that concrete fracture is the governing failure mode. In the 1995 Kobe earthquake, for instance, it was observed that failure of an exposed column base (Figure 1) was due to the fracture of the surrounding concrete near the steel bolts [2]. From the aforementioned findings, fracture failure of the brittle concrete at the steel anchor/concrete interaction zones clearly compromises the safety of the structures.

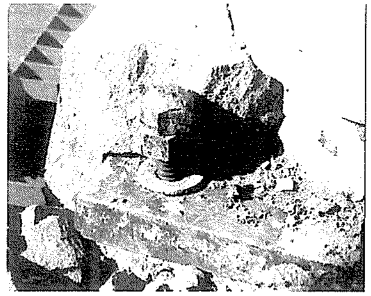

Figure 1: Fracture failure of concrete near steel bolt near column base of a structure during the Kobe earthquake [2]

To avoid the brittle failure of steel anchor in concrete, a number of methods including deep embedment length and dense confining reinforcement are used in practice, yet not very satisfying due to associated high cost. A more elegant approach is to directly impart tensile ductility into the concrete material to minimize or suppress the fracture mode of failure altogether in steel anchoring zone.

A ductilized concrete material, named engineered cementitious composites (ECC) [3], offers a potential material solution to steel anchor/concrete fracture problems. A typical tensile stress-strain curve of ECC is shown in Figure 2. As can be seen, ECC exhibits a tensile strain capacity in the range of 3-6% (300-600 times that of normal concrete or FRC) [4, 5]. It attains high ductility with relatively low fiber content (2% or less of short randomly oriented fibers) via systematic tailoring of the fiber, matrix and interface properties, guided by micromechanics principles. Associated with its high ductility in tension and shear [6], ECC reveals a high damage tolerant behavior under severe stress concentration induced by steel concrete interaction in a number of recent experiments, such as ECC panel shear-joint test [7], RCS connection (with ECC in joint zone) test [8] and precast infill panel (made with ECC) test [9]. These tests suggest the feasibility of adopting ECC in steel anchor/concrete connection to avoid fracture failure, thus leading to significant improvements in the overall structural response.

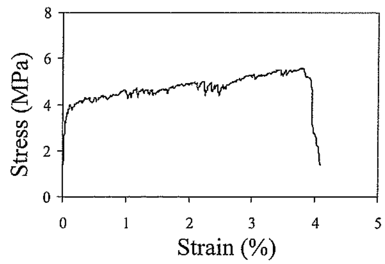

Figure 2: Typical tensile stress-strain curve of ECC

Typically, loading transfer between steel anchor and concrete is by shear, tension, or both. As can be seen from Section RD.4.1 of ACI Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02) (hereafter ACI 318-02 or ACI 318R-02) [10], for shear loading, concrete pryout is major failure mode if the anchors are far from a free edge, characterized by concrete fracture on the opposite side of loading direction and potential crushing on the other side, as shown in Figure 3 (a). For tensile loading, concrete breakout is the dominant failure mode, normally in a cone

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shape, as illustrated in Figure 3 (b). If a mixture type of loading exists, the actual shape of failure can be more complicated, yet concrete fracture may be expected. In this research, only the shear loading mode was investigated. First, tests on anchoring to ECC under shear loading were conducted in comparison with anchoring to normal concrete. The results will then be presented along with its implication on the ACI 318-02 [10]. Finally, general conclusions will be drawn from the investigation results.

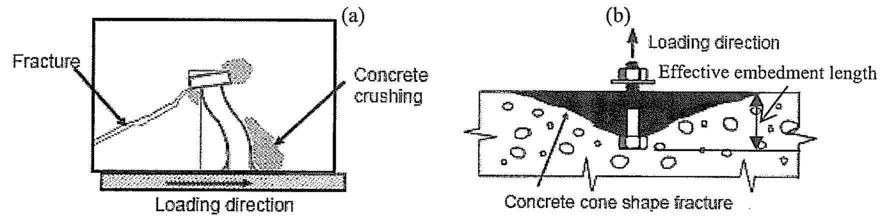

Figure 3: (a) Concrete pryout failure under anchor shear loading; (b) concrete breakout cone shape failure under anchor tensile loading

2. EXPERIMENTAL PROGRAM

The concrete materials used in this study are shown in Table 1, where the ductility of concrete, ECC 1 and ECC 2 are varied to investigate its influence on the structural response while the compressive strength remains about the same. In addition, ECC 3, with lower compressive strength in comparison with ECC 2, was conducted to take into account the influence of compressive strength on the structural performance of steel anchoring to ECC.

Table 1: Material properties and mix proportion in concrete and ECCs

Material	No.	ε_u	f_c'	C	S	CA	FA	W	SP	Fiber
concrete	2	0.01*	52.3±3.6	1	1.3	1.3	0	0.36	0.01	0
ECC 1	2	0.5±0.2	60.6±3.8	1	1	0	0	0.36	0.0075	0.016
ECC 2	3	2.5±0.3	60.0±2.1	1	0.8	0	1.2	0.53	0.03	0.02
ECC 3	2	2.5±0.4	46.0±0.4	1	0.8	0	1.2	0.58	0.03	0.02

(No.: number of specimens tested; ε_u: uniaxial tensile strain capacity (%); f_c': compressive strength (MPa); *: assumed value; ±: standard deviation; C: type I Portland cement except type III cement for ECC 1; S: silica sands F110 for ECC1, 2 and 3, ASTM C778 sand for concrete; CA: coarse aggregate with max size 19 mm; FA: type F fly ash; W: water; SP: superplasticizer, W R Grace Daracem ML320 except Glenium 3200HES from Master Builders for ECC 1; PVA fiber: KURALON K-II REC15, developed by Kuraray Co., LTD (Japan) in collaboration with ACE-MRL)

The geometry of the pushout specimen is shown in Figure 4. Two substrate slabs, with a dimension of 305mm x 305mm x 152 mm, were connected with a wide flange steel beam W8X40 with two shear studs welded on each side of the beam. The geometry is adopted from Ollgaard et al [11]. The headed stud has a diameter of 19mm and effective embedment length 64mm. Totally, 9 pushout specimens were tested, as shown in Table 1. Testing was conducted on a 2200 kN capacity Instron testing machine. Four LVDTs were mounted on the steel beam at the level of the shear studs to measure the slip between the beam and concrete/ECC slabs. The loading surface was ground for uniform load distribution before testing, and a ball support was used to maintain the alignment of the specimens.

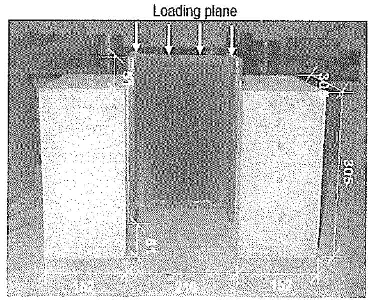

Figure 4: Geometry of pushout

3. RESULTS AND DISCUSSIONS

In concrete pushout tests, as loading approached the peak value, large cracks (crack width about 2 mm) formed in the concrete near the shear studs and developed rapidly throughout the entire specimen as the peak load was reached. As revealed in Figure 5, concrete specimens fractured into several pieces after testing, with fracture clearly initiated from near the head of the shear studs. The high stress concentration induced by the stiff steel stud combined with the brittle nature of concrete led to the rapid development of macro cracks, resulting in the catastrophic failure of concrete pushout specimens.

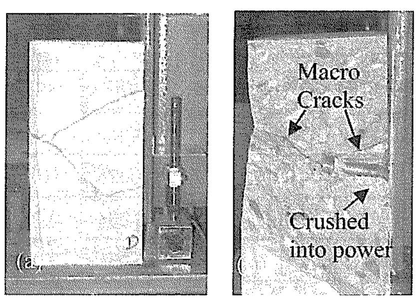

Figure 5: Half of concrete pushout specimen after test. Macro cracks observed on the (a) outside and (b) inside of the half specimen (natural fracture surface along shear stud)

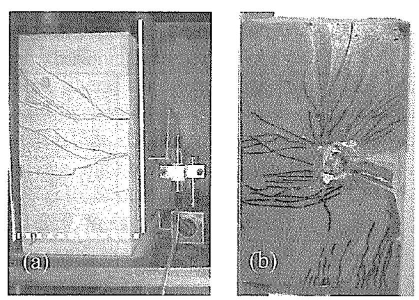

Figure 6: Half of ECC pushout specimen after test. Microcracks observed on the (a) outside and (b) inside of the half specimen (crack width ~ 40 μm, magnified by magic ink pen for clarity, cut section along shear stud indicated as white line in (a))

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Conversely, ECC pushout specimens showed a ductile failure mode due to its extreme tensile ductility. During the initial loading stage, no cracks could be observed from the specimen surfaces. As the load approached peak load, a few micro-cracks appeared on outside surfaces. While loading continues at a stable level, more micro-cracks radiated from the shear stud and developed outwards, as shown in Figure 6. For specimens ECC 2 and ECC 3, the peak load is generally associated with the localization of one of the micro-cracks into a fracture, even though fracture of the steel shank eventually led to a drastic load-drop. This suggests that the ductility of the ECC and the steel stud are fully utilized. Due to the relatively low ductility, ECC 1 specimens develop less micro-cracks in comparison with ECC 2 and 3 and failed due to localized fracture after ECC exhausted its strain capacity, yet its overall structural capacity and ductility is still much better than that of concrete.

Closely related to its favorable failure mode, the structural performance of the steel anchor/ECC pushout specimens was greatly enhanced compared to steel/concrete specimens in terms of load capacity and structural ductility, revealed in Figure 7 and Table 2. It should be noted that ECC 3 demonstrated much higher load capacity than concrete despite lower compressive strength. This suggests that material ductility in ECC plays a significant role in improving the structural response of the steel anchor/ECC connections given adequate compressive strength. The two ECC 3 specimens show quite different load capacity of 173kN and 148kN, respectively. There might be some alignment issue or welding problem for the lower one.

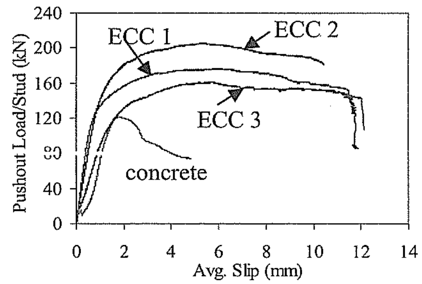

Figure 7: Representative pushout load – slip curves for different concrete materials shows much higher load

Table 2: Material properties and associated structural response in concrete and ECCs

Material	ε_u	f_c'	h_{ef}	V_{cp}	V_m	V_m/V_{cp}	S_c	W_c
concrete	**0.01***	**52.3±3.6**	64	125.9	125.5±5.4	**1.00**	2.0±0.2	~2000
ECC 1	**0.5±0.2**	**60.6±3.8**	64	135.5	173.4±4.3	**1.28**	5.9±0.2	~60
ECC 2	**2.5±0.3**	**60.0±2.1**	64	134.8	192.3±11.7	**1.43**	6.4±1.3	42±20
ECC 3	**2.5±0.4**	**46.0±0.4**	64	118.1	160.9±17.7	**1.36**	5.8±0.3	37±21

(ε_u: uniaxial tensile strain capacity (%); f_c': compressive strength (MPa); *: assumed value; h_{ef}: effective embedment length of anchor (mm); V_{cp}: nominal pryout strength (KN); V_m: measured shear strength (kN); S_c: slip capacity of headed stud (mm), which is the relative displacement between stud and concrete/ECC slab corresponding to peak load; W_c: crack width at peak load (μm); ±: standard deviation)

3.1 On design equation and strength reduction factor

Table 2 shows the calculated and measured structural response of concrete and ECCs, including nominal pryout strength V_{cp}, measured shear strength V_m, slip capacity S_c and crack width at peak load W_c. V_{cp} is calculated based on the design equation (1) in section D.6.3 of ACI 318-02 [10]. For all ECC and concrete specimens, the anchor design shear strength is governed by equation (1), which is lower than nominal anchor steel shear strength $A_{se}f_{ut}$ (180 kN).

$$V_{cp} = 34 \sqrt{f_c'} h_{ef}^{1.5} \tag{1}$$

where: A_{se} = Cross sectional area of a headed stud (mm^2);
f_{ut} = Tensile ultimate strength of a headed stud (MPa);
f_c' = Specified compressive strength of concrete (MPa);
h_{ef} = Effective embedment length of anchor in concrete (mm).

Revealed in Table 2, the measured shear strength of concrete pushout specimen V_m is almost the same as its calculated nominal pryout strength V_{cp}, hence the ratio of V_m over V_{cp} is one. However, the ratios for all ECCs are much higher than one, i.e., 1.28, 1.43, 1.36 for ECC 1, 2, and 3 respectively. Hence, a highly tentative design equation (2) of anchor nominal pryout strength in ECC is proposed below based on these ratios in limited available test results.

$$V_{ECCp} = 34\, \alpha \sqrt{f_c'} h_{ef}^{1.5} \tag{2}$$

where: α = strength modification factor due to concrete material tensile ductility, which may be linear interpolated from the ratio of V_m/V_{cp} according to their ductility and compressive strength, as shown in Table 2.

For current ECC field applications, such as ECC link slab project, a strain capacity of 2% and compressive strength of 60 MPa is considered adequate for the particular application [12]. Thus, it can be deduced that a strength modification factor of 1.4 may be suitable for the calculation of nominal pryout strength of a similar headed stud in ECC. If the material ductility or compressive strength is significantly out of the range of the test data, additional test needs to be conducted to obtain the appropriate modification factor. Alternatively, advanced numerical methods such as Finite Element Analysis, incorporated with appropriate constitutive mode of ECC [13], should be a cost effective approach in order to get a deep understanding of the anchor/ECC shear behavior and obtain a more complete design equation.

It should be noted that the embedment length of stud is not varied in the test and therefore in the equation. However, when the embedment length is four to five times the stud diameter the pryout failure may be precluded for concrete [14]. Other failure mode, such as anchor steel shear failure may occur. In case of ECC, this ratio should be even lower due to material tensile ductility. In fact, in this study, the ratio between these two parameters is about 3.4, very close to the limit aforementioned. This is indicated by the small difference between measured shear strength V_m of ECCs and nominal anchor steel shear strength $A_{se}f_{ut}$ (180 kN).

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The proposed design equation for anchor in ECC is very preliminary; however, it gives more room for structural designer to design anchoring connection. With the introduction of new ductile material ECC, one can now trade off between compressive strength and material ductility. For example, as can be seen in Table 2, even though the compressive strength of ECC 3 is lower than concrete, the structural response of ECC 3 is still much better than concrete. Therefore, it may be advantageous to design a lightweight (hence relative lower compressive strength) but high ductility ECC [15] into structure, which may be able to reduce the cost of construction on the foundation part, yet retain decent anchor shear strength. This concept needs to be further confirmed by experimental investigation.

In addition to measured strength V_m, slip capacity of anchor in ECCs is also enhanced by about 3 times in comparison with concrete, as revealed in Table 2. This could directly impact the strength reduction factor ϕ for anchors in concrete since this factor is closely related to the ductility of the concrete failure mode. For example, in Section D.4.4 of ACI 318-02 [10], two conditions is set for anchor governed by pryout strength, condition A applies (with a higher ϕ factor 0.75) where the potential concrete failure surfaces are crossed by supplementary reinforcement, otherwise condition B applies with a lower ϕ factor 0.70. This suggests anchor in ECC with a much ductile failure mode associated with much higher slip capacity may be able to use a favorable strength reduction factor, such as 0.8, this potential additional benefit will further increase the advantage of adopting ECC in critical anchoring region over concrete.

3.2 On durability

On the durability side, the anchor in ECC can maintain very tight crack width (around 40-60 μm) up to peak load at a slip capacity of around 6 mm. In contrast, concrete will fracture at a slip capacity of around 2 mm with a much wider crack width of about 2 mm. According to the 1995 edition of the ACI Building Code [16] (Section 10.6.4), a maximum crack width of 400 μm and 330 μm for interior and exterior exposure conditions is recommended, respectively. Hence, with extreme tight crack width at an extreme large structural ductility, it is very possible that even after accidental overloading the durability and serviceability of anchoring in ECC member can still be well maintained, which is not the case for anchoring in concrete.

3.3 On group effect

When an anchor group is considered, generally, theory of elasticity is required to be used assuming the attachment that distributes loads to the anchors is sufficiently stiff since "when the strength of an anchor group is governed by breakage of the concrete, the behavior is brittle and there is limited redistribution of the forces between the highly stressed and less stressed anchors", as stated in Section RD.3.1 in ACI 318-02 [10]. On the other hand, if anchor strength is governed by ductile yielding of the anchor steel, an analysis based on theory of plasticity is permitted since significant redistribution of forces can occur among anchors, therefore resulting in much better structural performance. From ductile failure mode of anchoring in ECC shown above, it suggests that significant redistribution of forces among headed studs is highly likely and greatly enhanced anchoring behavior may be expected. This concept has been proposed by Qian and Li [17], however experimental verification is needed.

3.4 On anchoring in ECC under tensile loading

Similar to anchoring to ECC under shear loading, much improved structural performance may be expected from anchoring in ECC under tensile loading, as partially demonstrated from the results of the two dimensional anchor bolt/ECC connection pullout tests [18]. The average pullout load capacity and structural ductility of 2-D anchor bolt/ECC connection are about twice and 16 times respectively, in comparison with those of concrete specimens. It should be noted that both ECC and concrete in these specimens have about the same compressive strength. Again, this suggests that material ductility is a significant contributing factor to the structural load capacity and ductility. Furthermore, the compressive strength may not be as important as shown in design equation of anchor in ECC under shearing loading since the pullout failure is more likely dominated by tensile behavior of ECC [18]. Three dimensional anchor/ECC pullout specimens need to be conducted in order to further confirm these findings and obtain appropriate design equation and strength reduction factor.

4. CONCLUSIONS

A new approach and material solution – elevating material ductility to structural performance of steel anchoring to ECC by suppressing concrete fracture failure was proposed and experimentally demonstrated through stud anchoring to ECC pushout tests. Significant delay or elimination of fracture localization can be achieved via extensive inelastic straining offered by ECC, resulting in much improved structural response in terms of load capacity and structural ductility. Current design equations offered in section D.6.3 of ACI 318-02, while appropriate for concrete, significantly underestimates the stud anchor load capacity when ECC is used in place of concrete. New design equations for load capacity and strength reduction factor need to be developed in order to incorporate the effect of ECC material ductility. Additionally, the influence of material ductility on durability and anchor group effect and anchoring to ECC under tensile loading are also discussed and promising results is expected. While the research is still very preliminary and significant works remain, this material based solution to concrete fracture problems provide structural engineer more room to design innovative structures, and is expected to be applicable to broad classes of structural applications involving critical steel/concrete connections.

5. ACKNOWLEDGEMENTS

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