Pullout simulation of post installed chemically bonded anchors in UHPFRC

Fabien Delhomme^{1,*}, Michael Brun¹

¹Univ Lyon, INSA-Lyon, GEOMAS, F-69621, France

Abstract. An experimental and numerical study was completed in order to examine the mechanical behaviour of post-installed bonded anchors in ultra-high performance fibre reinforced concrete with a compressive strength higher than 130 MPa. The aim was to analyse the failure mechanisms in static pullout tests and to suggest a simple numerical model, which can be employed in a design stage, to reproduce the global behaviour of the anchor. The experimental observations show that a combined pullout and concrete cone failure occurred for an embedment depth of 40 mm and a steel rod failure for an embedment depth of 100 mm. The numerical model was set up using Abaqus software, by adopting the concrete damage plastic model and a surface-based cohesive behaviour for the interface concrete-anchor. The obtained failure modes and ultimate loads are in good agreement with experimental results. A minimum embedment depth of 50 mm was assessed to prevent a pullout failure of the anchor.

1 Introduction

In civil engineering or in industrial building, fastening of structural or non-structural members in reinforced concrete is often performed by means of anchors. Anchors are divided into two general categories: cast-in-place and post-installed. The use of post-installed anchors allows for a greater flexibility in planning and design of concrete structures. Nowadays, post-installed anchors are currently not only used in repair and retrofit applications but also in new construction with the development of precast concrete products. These anchors can be classified into three categories: expansion anchors, undercut anchors, and bonded anchors. Expansion and undercut anchors transfer tension loads to the concrete through the embedded end of the anchor. Adhesive anchors transfer tension loads from the embedded element to the concrete through bond between the anchor rod and the adhesive and between the adhesive and the concrete along the entire bonded surface. A typical bonded anchor consists of a threaded rod inserted into a drilled hole in hardened concrete and filled with a bonding agent such as epoxy, polyester or vinylester.

The behaviour of post-installed anchors in normal strength concrete (concrete grade lower than or equal to C50/60) has been extensively investigated and their design is well-know and standardized [1]. The use of ultra-high performance fibre reinforced concrete (UHPFRC) is becoming more and more frequent with the use of precast elements. The incorporation of steel fibres into the concrete improves the concrete performances in terms of ductility and tensile strength.

Therefore, it should have an impact on the mechanical behaviour of post-installed anchors. An experimental campaign was led in order to provide additional experimental data on the tensile behaviour of bonded anchors in UHPFRC of compressive strength higher than 130 MPa. The tested parameters are the embedment depth, the edge distance and the state of cracking of the concrete sample [2]. In this paper, only the tests results in the uncracked concrete without edge effect are presented and modelled.

2 Background

A lot of research has already been carried out in order to analyse the behaviour of post-installed anchors in concrete; the failure modes of the post-installed bonded anchors in normal strength plain concrete were thoroughly studied [3, 4]. The different failure mechanisms are the following:

- Steel rupture of the anchor for large embedment depths.
- Concrete cone breakout failure for shallow embedment depths (h_{ef}) typically observed between 3 and 5 times the rod diameter [5-9]. Hydrostatic stress conditions seem to appear in a small zone very close to the end of the anchor. Starting from this anchor/concrete interface, circumferential microcracks grow towards the concrete block surface. At the same time, radial microcracks appear, from the anchor hole to the concrete block edges. In case of circumferential microcracks joining the surface, a concrete cone breakout happens. This failure mode is described by the

* Corresponding author: fabien.delhomme@insa-lyon.fr

concrete cone [3, 5] or concrete capacity design (CDD) models [6-9].

- An extraction of the anchor at low embedment depths. A concrete cone failure surface appears near the surface of the block and a failure at the interface between anchor and concrete in the remaining portion occurs.
- Splitting failure may occur with anchors located near the edges or in a thin thickness slab [10]. The typical splitting mechanism is a radial cracking at the end of the anchor and a growth of vertical cracks until they reach conical cracks at the end of the anchor. In the same time superficial cracks growing towards the edges of the concrete block.

Few results on the behaviour of post installed bonded anchors in steel fibre reinforced concrete are available. Among these, Gesoglu et al. [11] carried out monotonic tension tests on 12 and 16 mm diameter adhesive anchors at embedment depths ranging from 40 to 160 mm. Normal strength and high strength concrete were used with, respectively, a compressive strength of 30 or 50 MPa. In some mixtures, steel fibres of 60 mm long and 0.8 mm in diameter were used with a fibre content of 0.8 and 1 % by volume. With the addition of steel fibres the damage of concrete was significantly reduced. In case of a concrete cone failure, in steel fibre reinforced concretes, the concrete cones were composed of three or more pieces rather than one unique piece. The use of steel fibres in concrete did not affect the pullout capacity of the anchors, but the failure type of some anchors shifted from concrete cone to pullout mode or combined pullout and cone mode. Moreover, the displacement at maximum load was generally higher in steel fiber reinforced concrete. For deeper embedment depths, the influence of steel fibres on the displacement at the ultimate load was significantly reduced. Saad et al. [12] carried out tests in high performance steel fibre reinforced concrete with headed cast-in-place anchor. Concrete strength ranged from 27.4 to 58 MPa, four steel volume fractions of steel fibres ranging from 0.4 to 1.6 %, three anchor diameters (8, 10 and 12 mm), and four embedment depths from 25 to 62.5 mm were used. The majority of the specimens failed by concrete cone failure. The addition of steel fibres to concrete increased the concrete cone breakout capacity of the anchors. In fibrous concrete, the concrete cone angle was increasing and the failure cones were smaller than in concrete specimens without fibres.

3 Test setups and procedures

Three embedment depths of 40, 60 and 100 mm are tested. The 40 mm embedment depth was tested in order to have a pullout failure. The main characteristics and parameters of the realized tests are given in Table 1.

The tested anchors were torque controlled bonded anchors HIT-Z threaded steel rod of 12 mm in diameter (ϕ_{anchor}) (Table 2). The anchors were post-installed by drilling a 14 mm diameter hole with a handheld percussion drilling machine. The anchors were installed

in the cleaned hole with a urethane acrylate methacrylate resin. The adhesive cured for a minimum of 24 hours at room temperature before testing.

One anchor was installed in a concrete block (285 x 340 mm and 150 mm high), which was large enough to avoid splitting failure and edge effects when the anchors were positioned in the centre. The ultra-high performance fibre reinforced concrete used was the SMART-UP BCV type supplied by Vicat Company. The mix proportions are given in Table 3. The specimens are cast into wood forms according to the recommendation of the manufacturer (mixing of premix, addition of water and superplastizer, addition of short and long fibres). The concrete is mixed by small amounts (15 l) to ensure an efficient concrete mix and a homogenous fibre distribution. The UHPFRC samples are poured vertically from one location at the centre of the formwork without any vibration (self-placing concrete). The anchors were installed on the upper surface of the block (side without formwork) following the casting direction. After removal, one day after casting, the concrete samples were cured into sealed plastic bags, so no additional water gain was possible during hydration. The anchors were tested at least 28 days after casting. The mean bending flexural strength was controlled before each test on three 40x40x160 mm prismatic rectangular samples and the mean compressive strength on six 40x40x80 mm prismatic rectangular samples. The elastic modulus was measured on cylindrical samples of 220 mm high and 110 mm diameter.

The samples were tested on a hydraulic press and fixed in order to have no confining pressure (unconfined tension test) applied around the anchorage (Fig. 1). The quasi-static tensile loading was displacement controlled with a loading rate of 1 mm.min⁻¹. The load was applied until the failure of the anchor. Three identical samples of each configuration were tested.

The data recorded, at acquisition rate of 2 Hz, were the following:

- The displacement between the anchor and the upper surface of the concrete block with two LVDTs (S1 and S2). The mean displacement was used in the analysis of results,
- The load applied with a load sensor.

4 Pullout test results

Table 1 gives the main results of pullout tests. The standard deviation on the 3 tests gives the uncertainty on load and displacement. For tests h60 and h100, with a nominal embedment depth of respectively 60 and 100 mm, the anchors failed via steel rupture with no visual damage of concrete observed at the block surface (Fig. 2-a). For a nominal embedment depth of 40 mm (tests h40), the failure mode was a combined pullout and concrete cone failure (Fig. 2-b). The maximum tensile load was 52.7 kN with a displacement of 1.4 mm. The

Fig. 1. Pullout test setup (dimensions in mm).

Table 1. Main characteristics and results of pullout tests.

Test	Mean effective embedment depth (mm)	Mean concrete compressive strength (MPa)	Mean concrete tensile strength (MPa)	Failure mode	Ultimate load N_{max} (kN)	displacement at ultimate load (mm)
h100	101.8	162	38	Steel rod failure	66.2 ± 0.1	1.00 ± 0.01
h60	61.5	140	44	Steel rod failure	64.7 ± 1.5	1.19 ± 0.09
h40	43.0	149	43	Pullout failure	52.7 ± 1.7	1.43 ± 0.09

Table 2. Mechanical properties of steel and concrete.

Steel rod	Yield strength (MPa)	520
	Ultimate strength (MPa)	650
	Elastic modulus (GPa)	210
Concrete	Mean compressive strength (MPa)	150 to 165
	Mean flexural strength (MPa)	39 to 48
	Elastic modulus (GPa)	43.5

Table 3. Mixture proportions of BCV for 1 m³.

Premix (kg)	Water (kg)	Water-cement ratio	Superplasticizer (kg)	Short steel fibre 12.7/0.175 mm (kg)	Long steel fibre 20/0.3 mm (kg)
2086	210	0.19	28.7	60	100

failure occurred at the interface between the resin and the rod. The concrete cones were often in many pieces rather than in a complete cone. The height of the concrete cone was approximately 15 mm. The mean bond stress, resulting from the strains of rod and concrete and the interface damage concrete-anchor, was calculated in considering a uniform stress because of the low embedment depth. For tests h40, the average bond stress was 32.5 MPa, was obtained from Eq.1.

$$\tau = \frac{N_{max}}{\pi \cdot d \cdot h_{ef}} \quad (1)$$

where τ is the bond stress and N_{max} the experimental ultimate tensile load.

Fig. 2. (a) Steel rupture for tests h100 (effective embedment depth = 100 mm) – (b) Combined pullout and concrete cone failure for tests h40 (effective embedment depth = 40 mm).

5 Finite element analyses

5.1 Finite element model

The objective of the modelling is to provide a simple model able to reproduce the global behaviour of the anchor, i.e. the load-sliding curve and the cracking of the concrete. The numerical simulation is performed with the Abaqus software [13]. A non-linear incremental static analysis is conducted for the simulation of pullout tests. The proposed model is an axisymmetric model although the block cross-section is rectangular. In fact, the block cross-section is large in comparison to the anchor diameter and the concrete compressive strength is very high, therefore no damage near edges is observed. The radius of the circular cross-section was considered as the lowest distance between the anchor and the edge of the block. The block and the anchor are modelled with 4-node, bilinear, axisymmetric, quadrilateral elements (CAX4R). After a sensitivity analysis, the maximum concrete elements size is chosen as equal to 4x4 mm (coarse mesh) at the edges and the minimum size is set to 2x2 mm (refined mesh) around the anchor.

5.2 Boundary and loading conditions

The anchor is clamped at its free end and a constant displacement is applied on the support width of the block. The thickness of the glue is no considered and a surface-based cohesive behaviour is used to model the concrete-anchor interface. The contact parameters are calibrated with the experimental test h40 (bond failure). No contact is considered between the end of the anchor (horizontal surface) and the concrete because of the very low tensile strength of the resin. All the boundary conditions are depicted on Fig. 3.

5.3 Constitutive laws and materials properties

In Abaqus, different types of concrete constitutive models are provided: a smeared crack model, a discrete crack model and a damage plasticity model (CDP), which has proved very powerful for modelling concrete structures [14–16]. To model the crack propagation in UHPFRC, CDP model is used in this study, assuming that fibers are uniformly distributed in the matrix.

Fig. 3. Geometric and boundary conditions of the model (dimensions in mm).

Therefore, the UHPFRC is modelled as a homogeneous material with the CDP model [17, 18]. The main assumptions of this model are the following:

- two damage mechanisms: tensile cracking and compressive crushing of concrete [19],
- material stiffness is reduced by two damage parameters (tension and compression) (Fig. 4),
- the yield function is specified according to reference [17] and the flow potential is a hyperbolic function [13],
- the plastic flow is non-associated.

Experimental data obtained with a similar UHPFRC concrete (2 % amount of fibre), for compression and tensile tests (compressive strength of 150 MPa and tensile strength of 9 MPa), were used to define the strain-stress curves in tension [20]. In this study, as the compressive stress in the pullout tests is always smaller than the compressive strength, the compressive damage is not taking into account. Moreover UHPFRC material can be considered as quasi-elastic behaviour until the compressive strength [21, 22], therefore an elastic compressive behaviour was assumed. The use of tensile stress-strain laws leads to mesh-dependent results as the crack tends to localise in one element width. To mitigate this problem, the equivalent stress crack opening displacement law was used to model tension stiffening in this study.

The damage parameter ranges from 0 representing no damage to 1 representing complete failure. The tensile damage D_t is calculated with Eq. 2.

$$D_t = \frac{1 - \sigma_t}{\sigma_{t0}} \quad (2)$$

where σ_t is the traction related to the crack opening displacement curve in the softening range and σ_{t0} the tensile stress at failure.

The other model parameters are the following (Table 4):

- Dilation angle ψ is a measurement of how much volume increase occurs when the material is

sheared. For a Mohr–Coulomb material, dilation is an angle that generally varies between zero (non-associated flow rule) and the friction angle (associated flow rule). A default value of 38° was considered [23, 24].

- Flow potential eccentricity ε is a small positive number that defines the rate at which the hyperbolic flow potential approaches its asymptote. A default value of 0.1 was considered. The plastic-damage model assumes non associated potential flow (Drucker-Prager hyperbolic function).
- σ_{b0}/σ_{c0} is the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress with a default value of 1.16.
- K_c is the ratio of the second stress invariant on the tensile meridian to the compressive meridian with a default value of 2/3.
- Viscosity parameter μ is used for the visco-plastic regularization of the concrete constitutive equations. Szczecina et al. [25] recommend a maximal value of 0.0001 to avoid convergence difficulties in implicit analysis with material models exhibiting severe degradations. A value of 0.0001 was considered in this study.

To model the steel of the anchor, an ideal elastic-plastic model, satisfying Von Mises yield criterion, an associated flow rule and isotropic hardening, was used to define the constitutive behaviour. The yield and tensile strengths are given in Table 2.

Fig. 4. Uniaxial stress-strain curve with damage (a) in tension and (b) compression [16].

5.4 Anchor – concrete block contact interaction

The surface-based cohesive model in Abaqus models the delamination for small thickness interfaces in terms of traction versus separation. A linear elastic traction-separation law prior to damage is assumed. The failure of the cohesive bond is characterized by progressive degradation of the cohesive stiffness (Fig. 5).

An ascending linear branch until a maximum value is reached (t_0) indicates the damage initiation; afterwards a descending branch describes the evolution of the damage. An elastic constitutive matrix relates nominal tensile stresses and nominal strains. Displacements are directly considered rather than strains. An uncoupled elastic constitutive matrix is chosen, as expressed in Eq 3.

$$\mathbf{T} = \begin{Bmatrix} t_n \\ t_s \\ t_t \end{Bmatrix} = \begin{bmatrix} k_{nn} & 0 & 0 \\ 0 & k_{ss} & 0 \\ 0 & 0 & k_{tt} \end{bmatrix} \begin{Bmatrix} \delta_n \\ \delta_s \\ \delta_t \end{Bmatrix} \quad (3)$$

Where $\mathbf{T}$ is the nominal traction stress; t_n , t_s and t_t are the normal and the two shear tractions respectively; δ_n , δ_s , δ_t are respectively the displacements related with the normal and transversal directions; k_{nn} , k_{ss} and k_{tt} are the stiffness coefficients.

According to Molina et al. [24], k_{ss} and k_{tt} are obtained with the experimental bond-slip relation for a 40 mm embedment depth (test h40) in Eq.(4). The stiffness of the normal traction is given in Eq.(5).

$$k_{ss} = k_{tt} = \frac{t_s^0}{\delta_s^0} \quad (4)$$

$$k_{ss} = k_{tt} = \frac{t_s^0}{\delta_s^0} \quad (5)$$

where t_s^0 is peak value of the shear stress and δ_s^0 the experimental displacement corresponding to t_s^0 .

A maximum separation criterion is assumed to model the damage initiation (Eq. 6).

$$\max \left\{ \frac{\langle \delta_n \rangle}{\delta_n^0}; \frac{\langle \delta_s \rangle}{\delta_s^0}; \frac{\langle \delta_t \rangle}{\delta_t^0} \right\} = 1 \quad (6)$$

where δ_n^0 , δ_s^0 and δ_t^0 represent the peak values of the contact separation, when the separation is either purely along the contact normal or purely in the first or the second shear direction, respectively.

The damage evolution describes the rate at which the cohesive stiffness is degraded once the corresponding initiation criterion is reached. A decreasing linear evolution based on the energy that is dissipated as a result of the damage process is used. The fracture energy is equal to the area under the experimental traction-separation curve.

The parameters of the anchor-concrete block contact interaction are given in Table 4.

Fig. 5. Interfacial cohesive law.

Table 4. Parameters of the CDP model and the cohesive element.

ψ (°)	ε	σ_{b0}/σ_{c0}	K	μ	t_s^0 (MPa)	δ_s^0 (mm)	k_{ss} (N.m ⁻²)	k_{nn} (N.m ⁻²)
38	0.1	1.16	2/3	0.0001	37	6.8	45.5	45500

5.5 Results and comparison with experimental data

Fig. 6 shows the sliding of the anchor (relative displacement between the concrete block and the anchor) following the tensile load applied. Experimental measures for tests h40 (pullout failure) and h100 (steel failure), and numerical results for different embedment depths are plotted. For an embedment depth of 40 mm (h40) with a pullout failure, the FE model accurately reproduced the global behaviour of the anchor. The bond stage up to the ultimate load and the sliding stage are in good agreement with the experimental test. The ultimate load and respective displacement, which are the main data in a design stage, are accurately estimated. Fig. 7 is depicted the concrete cracking for test h40 at the ultimate displacement (8 mm). A concrete cone appears with a depth of approximatively 15-20 mm as in the experimental observations.

For an embedment depth of 100 mm (h100) with a steel failure, the behaviour of the anchor up to the yield stress, i.e. the anchor stiffness, is satisfactory reproduced. Fig. 8 shows that the failure is localized at the surface end of the rod as in experiments. A parametric study on the embedment depth enables us to conclude that a yielding of the steel rod is obtained rather than an anchor bond failure for an embedment depth higher than 50 mm (equal to 4. ϕ_{anchor}).

Fig. 6. Comparison between experimental (dotted lines) and numerical results.

Fig. 7. Concrete damage for test h40 for a displacement of 8 mm.

Fig. 8. Von Mises stress (in MPa) for test h100 at the ultimate load (steel yield value = 520 MPa).

6 Conclusions

The use of ultra-high performance fibre reinforced concrete enables to significantly improve the tensile capacity of the post-installed bonded anchors. The metallic fibres and the reduced water amount used in the concrete mixture provide a high tensile strength precluding the collapse of the anchor characterized by a concrete cone failure. In uncracked concrete, the only failure modes observed were a steel rupture, for an embedment depth higher than 40 mm, or a combined pullout and concrete cone failure. In the last case, the breaking occurs at the interface between the resin and the steel rod and the concrete cone is characterized by a low height and a low inclination angle in comparison in normal strength concrete. The reinforcement provided by the metallic fibres leads to a better distribution of the stresses into the concrete.

The finite element modelling using Abaqus software permits to reproduce the anchor behaviour under tensile stress. The surface-based cohesive and the CDP models adopted in this simulation allow to predict the global tensile behaviour of the anchor and the failure mode for bonded anchors in UHPFRC in a design stage. A minimum embedment depth of $4 \cdot \phi_{\text{anchor}}$ was estimated in order to avoid a pullout failure of the anchor. Ongoing

tests on fastenings with several anchors will allow validating further the proposed finite element model.

References

1. EOTA. *Guideline for European technical approval of metal anchors for use in concrete ETAG 001* (Brussel, 2013).
2. F. Delhomme, P. Chaudet, J. Gioratto, *BEFIB 201*, (Vancouver, 2016).
3. R.A. Cook, *J. of Str. Eng.* **119**, 2744-2762 (1993).
4. S.J. Kim, S.T. Smith, *J. of Comp. Constr.* **14**, 406-414 (2010).
5. CEB, *Fastenings to concrete and masonry structures: State-of-the-art report* (Lausanne, 1994).
6. W. Fuchs, R. Eligehausen, J.E. Breen, *ACI Struc. J.* **92** 1, 73-94 (1995).
7. AISC, *Steel Design Guide: Base plate and anchor rod design* (2006).
8. EOTA. *TR029 - Design of Bonded Anchors* (Brussel, 2010).
9. CEB/Fib. *Design of anchorages in concrete* (Lausanne, 2011).
10. T. Hüer, R. Eligehausen, *FramCos-6* (Catania, 2007).
11. M. Gesoglu, T. Özturan, M. Özel, E. Güneyisi, *ACI Struct. J.*, **102** 22, 224-231 (2005).
12. A. Saad, A. AbdulKader, A. Alaa, *Asian J. of App. Sc.* **5** 7, 485-496 (2012).
13. Dassault Systemes Simulia, *Abaqus, ver. 6.9-3*. (RI, 2010)
14. J.F. Chen, G.M. Chen, J.G. Teng, *FRPRCS-9* (Sydney, 2009).
15. S. Li, J. Chen, L. Bisby, Y. Hu, J. Teng, *Int J. Struct. Stab. Dynam.* **12** 01, 75-94 (2012).
16. G.M. Chen, J.F. Chen, J.G. Teng, *Constr. Build. Mat.* **32**, 13-26 (2012).
17. J. Lubliner, J. Oliver, S. Oller, E. Oñate, *I.J.S.S.* **25**, 229-326 (1989).
18. J. Lee, G.L. Fenves, *J. of Eng. Mech.* **124**, 892-900 (1998).
19. J.F. Chen, Y. Tao, Springer-Verlag. (2011)
20. H.M. Goran, Y. Zhenjun, M.T.H. Aram, *Constr. and Build. Mat.* **48**, 1027-1034 (2013).
21. L. Moreillon, *Shear and punching shear strength of high performance fibre reinforced concrete structures* (Univ. Paris-Est, 2012).
22. AFGC. *Ultra-High Performance Fibre-Reinforced Concretes – Recommendations* (2013).
23. J. Henriques, L. Simões da Silva, I.B. Valente, *Eng. Struct.* **52**, 747-761 (2013).
24. L. Molina, F. Javier, F. Ruiz, M. Alejandro, E.H. Montes, A. Alonso, M. Cruz, *3rd Int. Conf. on Mech. Mod. in Struct. Eng.* (Univ. of Seville, 2015).

25. M. Szczecina, A. Winnicki, *World Congr. on Adv. in Struct. Eng. and Mech.* (Incheon, 2015).