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Indoor and outdoor pullout tests for retrofit anchors in low strength concrete

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Abstract. In this study, pullout capacities of post-installed deformed bars anchored in low strength concrete using different bonding materials are investigated experimentally. The experimental study was conducted under outdoor and indoor conditions; on the beams of an actual reinforced concrete building and on concrete bases constructed at Istanbul Technical University (ITU). Ready-mixed cement based anchorage mortar with modified polymers (M1), ordinary cement with modified polymer admixture (M2), and epoxy based anchorage mortar with two components (E) were used as bonding material. Furthermore, test results are compared with the predictions of current analytical models. Findings of the study showed that properly designed cement based mortars can be efficiently used for anchoring deformed bars in low quality concrete. It is important to note that the cost of cement based mortar is much lower with respect to conventional epoxy based anchorage materials.

Keywords: adhesive; indoor; mortar; outdoor; pullout; retrofit

1. Introduction

A significant amount of existing buildings is vulnerable against earthquakes because of lack of proper engineering service and inspection, particularly in developing countries[1]. These types of substandard structures often suffer from low quality concrete and insufficient reinforcement details, as well as global weaknesses of the configuration of the structural system. These deficiencies and weaknesses require immediate measures in terms of seismic retrofitting for enhancement of strength and deformability of this type of buildings. Jacketing of structural members and integration of new shear walls to the existing structural system are among common seismic retrofit methodologies[2]. These methodologies require proper integration of new structural components with the existing structural system and the satisfactory performance of the retrofitted structure strongly depends on the effective integration of new structural members with the existing structural system. Currently, the most common and feasible method of integration is making use of chemical anchors, which are generally formed with deformed steel bars and epoxy based adhesives.

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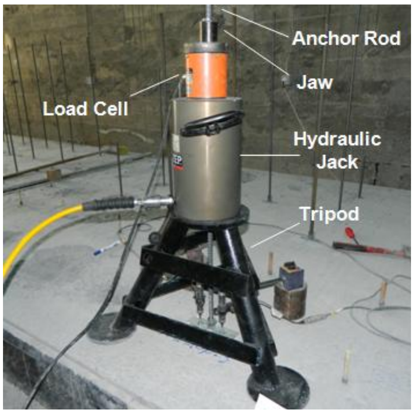
The performance of such chemical anchors mainly depends on the quality of concrete, quality of epoxy adhesive, embedment depth, environmental conditions and quality of application procedure. In literature, there are several studies on the pullout behavior of chemical anchors[3-24].

However, most of these studies have considered cases of medium to high strength concrete and epoxy based adhesives for bonding. On the other hand, while the epoxy based adhesives provide high resistance and good adhesion, they are expensive. Furthermore, a large portion of the existing substandard structures requiring seismic retrofit has been constructed with low strength concrete. While there are few studies, which have focused on cement based adhesives[3-5] or low strength concrete[6-9], none of the available studies has investigated the pullout behavior of the retrofit anchors in low strength concrete bonded using cement based materials, which are both cheaper and easier to provide in comparison with epoxy based adhesives. It should be noted here that the failure mode of the chemical anchors can be quite different when the strength of existing concrete is low[7] and in such cases the efficiency of cement based adhesives can be remarkably higher with respect to cases, where concrete strength is relatively higher. Therefore, this study aims to shed a light on the pullout behavior of chemical anchors bonded in low strength concrete using different types of bonding materials. During the installation of anchors, ready-mixed cement based anchorage mortar with modified polymers (M1), ordinary cement with modified polymer admixture (M2), and epoxy based anchorage mortar with two components (E) are used. Another unique feature of this study is the execution of a group of pullout tests on actual structural members of an existing reinforced concrete structure. After obtaining interesting and beneficial results from in-situ tests, indoor anchors were tested in the laboratory to have more consistent results under better controlled conditions and further validate obtained results. Consequently, the performance of the in-situ and laboratory pullout test results could also be presented in a comparative manner. For this purpose 10 outdoor and 18 indoor pullout tests were carried out for retrofit anchors in low strength concrete using different types of bonding materials. A group of partially bonded anchors were also included in the testing program to determine the bond strength of the adhesive and mortars in low strength concrete by avoiding cone type failure of concrete.

After presentation of experimental results with a focus on the failure patterns, the pullout capacities of the anchors are also calculated using available analytical models, and the analytical anchor capacities are compared with experimental results.



(a) Pullout test setup

(b) close-up view of displacement

Fig. 1 Test setup

2. Research significance

The unique features of this study are 1) execution of unconfined anchor pullout tests on the structural members of an existing reinforced concrete building with substandard concrete characteristics, 2) execution of laboratory controlled pullout tests to validate the results of outdoor tests and 3) usage of remarkably low-cost cement-based bonding materials for anchoring (as well as epoxy based adhesive) in case of low strength concrete. The outcomes of the study, after further validation of the results in a more general manner, may lead a considerable reduction of retrofit costs of substandard existing buildings.

3. Outline of experimental investigation

3.1 Test setup

All indoor and outdoor tests were carried out with the same test setup, which was designed according to ASTM E488M-10[25] (Fig. 1). The distances between the anchors were properly adjusted to prevent interaction between the neighboring anchors. Through this test setup, undesired confinement of the concrete around the anchor was avoided. The pullout load was applied through a central-hole jack and the applied load was measured with a central-hole load cell as shown in Fig. 1. The pullout displacement (slip) during the indoor pullout tests was measured by two displacement transducers of 25 mm stroke capacity as shown in Fig. 1. Unfortunately, pullout displacements could not be measured during outdoor tests. All parts of the pullout test setup were over designed to minimize the potential deformation of the test setup during testing.

3.2 Materials

In this study, three different types of commercial bonding materials were used; Meyco Thixo 100 (M1) is a shrinkage compensated ready mixed anchorage mortar which includes mineral fillings, silica fume and polymer reinforced special cement, Meyco Flowcable (M2) is a shrinkage compensated powder admixture material that contains modified polymers and is used with any type of portland cement, Concresive 1406 (E) epoxy based repair, anchorage and adhesive mortar with two components; resin and hardener. The mix-proportions of the cement based mortars (M1 and M2) are shown in Table 1. For determination of mechanical characteristics of bonding materials, prisms of 40x40x160 mm dimensions were prepared using cement based anchorage mortars (M1 and M2) and the epoxy adhesive (E). After flexural tests of these prisms under three point bending, compression tests were also carried out on the remaining parts of the prisms[26]. It should be noted that the prisms were only prepared in the laboratory during the installation of the indoor pullout anchors. Furthermore, the prisms were kept in the laboratory, until the testing day to represent the actual environmental conditions of the anchor specimens properly. Bending and compression tests of the prisms were carried out at the pullout test day of indoor anchors. The flexural and compression characteristics of the cement based mortars and epoxy adhesive used for indoor applications are presented Table 2. During application and curing periods, the outdoor and indoor temperatures were around 15-20 ℃ and 20-25 ℃, respectively. The relative humidity was around 55-65 % and 75-85 %, outdoor and indoor, respectively.

Table 1 Mix-proportions of cement based mortars

Mortar	Product	Cement (CEM I 42.5R)	Water	Water/Binder
M1	20 kg	-	5 kg	0.25
M2	6 kg	100 kg	34 kg	0.32

Table 2 Mechanical characteristics of indoor anchors adhesive and mortars

Material type	Flexural tensile strength (MPa)	Compressive strength (MPa)
M1	7.37	81.0
M2	6.59	55.0
E	31.62	46.7

Table 3 Rebound hammer test results on columns

Column Name	Rebound Hammer Values			
	Median	Average	Std. Deviation	Coeff. of Variation (%)
S101	33	32.7	0.95	2.90
S102	32	31.5	1.51	4.79
S103	32	32.0	1.56	4.89
S104	31	31.4	1.26	4.03
S201	29	29.2	1.32	4.51
S202	33	32.7	1.16	3.55
S203	28	28.4	1.96	6.88

3.3 Outdoor tests

Outdoor tests were executed on an actual reinforced concrete frame building which had been built in 1993, in Kucukcekmece, Istanbul, Turkey (Fig. 2). The building is a two storey reinforced concrete frame structure with low strength concrete, and it represents a large portion of the existing substandard building structures that are vulnerable against seismic actions. The outdoor pullout tests were carried out around the 14th day after installation of anchors on beams of the second floor (Fig. 3). For assessing the quality of concrete, 7 cores of 95 mm diameter were extracted from 7 columns (4 at the first storey and 3 at the second storey) and were tested under compression using a 5000 kN capacity Amsler testing machine. At the end of the tests, the average core compressive strength was determined as 12.3 MPa with a standard deviation of 1.3 MPa and a variation coefficient of 10.9%. The equivalent standard cylinder (150x300mm) concrete compressive strength was determined as 9.8 MPa using the procedure given by EN 12504-1:2009[27]. For further checking the variation of concrete quality throughout the building, rebound hammer tests were carried out on a number of columns, from which core samples were taken, and beams on which pullout tests were executed. The rebound hammer number was evaluated according to EN 12504-2:2012[28]. As seen in Tables 3 and 4, the averages of the numbers on the columns were between 28.4 and 32.7, and between 29.4 and 31.2 on beams. Consequently, it was concluded that the distribution of concrete quality throughout the building is quite uniform, and the average of core test results can represent the structure reasonably well.

Fig. 2 The in-situ test building, Kucukcekmece, Istanbul, Turkey.

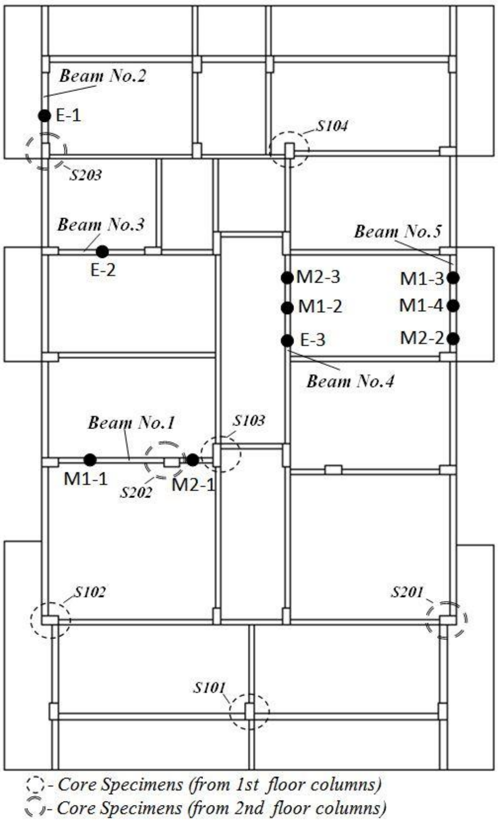


Fig. 3 The locations of the outdoor pullout tests.

Table 4 Rebound hammer test results on beams

Beam No	Specimen	Median	Average	Std. Deviation	Coeff. of Variation (%)
1	OD_M1_1, OD_M2_1	30	29.5	1.08	3.66
2	OD_E_1	30	29.4	1.17	3.99
3	OD_E_2	31	31.2	2.10	6.72
4	OD_M1_2, OD_M2_3, OD_E_3	30	29.8	2.15	7.21

Table 5 Mixture proportion of ready-mix low strength concrete for indoor tests (kg/m^3)

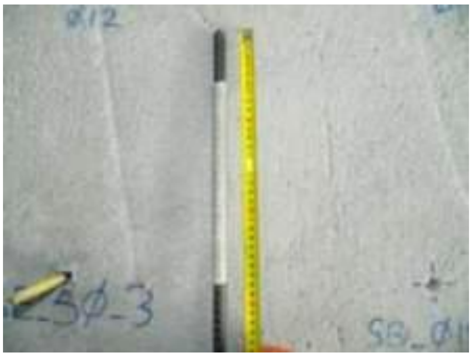
Cement	Water	Sand	Gravel	Admixture
155	155	1084	1058	2.22

3.4 Indoor tests

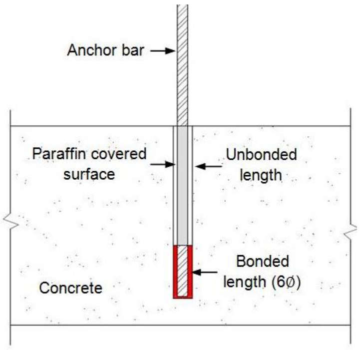
Indoor tests were carried out at Istanbul Technical University, Structural and Earthquake Engineering Laboratory. It was aimed to compare the performances of the anchors installed under outdoor and indoor conditions. The anchors were installed in concrete blocks with dimensions of

Table 6 Mechanical characteristics of anchor bars

Specimen	Yield strength f_{ya}, MPa	Tensile strength f_{uta}, Mpa	Ultimate elongation ε_{su}
Outdoor Test Anchor Bars	484	603	0.21
Indoor Test Anchor Bars	464	581	0.20



(a) covered with paraffin



(b) view of partially bonded anchor

Fig. 4 Partially bonded anchor

3000×2000×300 mm. No reinforcement was placed in these blocks. Specially designed ready-mix low strength concrete was used for the construction of blocks. The mix-proportion of the concrete is given in Table 5. The slump of the fresh concrete was 130 mm. The cylinder concrete samples were kept under the same condition as the concrete blocks. The compression and splitting tensile tests of concrete cylinder samples were carried out around the days of anchor tests approximately 180 days after casting. The average compressive strength and splitting tensile strength were 14.2 MPa and 1.9 MPa, respectively. The elastic modulus of concrete was determined as 23500 MPa through concrete compression tests. For determination of all mechanical characteristics of indoor concrete blocks, at least three identical specimens were tested. The pullout tests of indoor anchors were carried out 28 days after installation of anchors.

3.5 Anchoring application details

All anchors were 12 mm diameter deformed reinforcing bars (Ø 12). Two different groups of 12 mm diameter deformed bars were used as anchor bars during indoor and outdoor tests. The mechanical properties of the anchor bars determined through uniaxial tension tests[29] are given in Table 6.

All anchor bars were embedded into anchorage holes of 200 mm depth (≈16 times anchor bar diameter). However, the length of bonded part of the partially bonded anchors was 60 mm (5Ø) to reach the bonding strength of adhesive and mortars by avoiding cone type failure of concrete. For achieving debonding in case of partially bonded anchors, the upper parts of the anchors were covered with paraffin (Fig. 4) and the holes of the anchors were filled with less amount of adhesive/mortar to fill only the bottom 60 mm long part. The diameters of anchorage holes were 10 mm wider than the anchor bar diameter in case of specimens M1 and M2 (cement based mortars) and 6 mm wider than the anchor bar diameter for E specimens (epoxy based adhesive).

Table 7 Test results of outdoor anchors

Specimen	Hole diameter (mm)	Failure mode	Tensile load (kN)	Average tensile load (kN)	Std. deviation	Coeff. of Variation (%)
OD_M1_1	22	Bond failure	62.1	63.2	1.8	2.9
OD_M1_2	22	Bond failure	61.4			
OD_M1_3	22	Bond failure	63.9			
OD_M1_4	22	Bond failure	65.5			
OD_M2_1	22	Bond failure	43.8	57.0	13.1	22.9
OD_M2_2	22	Steel failure	69.9			
OD_M2_3	22	Bond failure	57.3			
OD_E_1	18	Steel failure	57.0	63.9	6.0	9.4
OD_E_2	18	Steel failure	68.1			
OD_E_3	18	Steel failure	66.5			

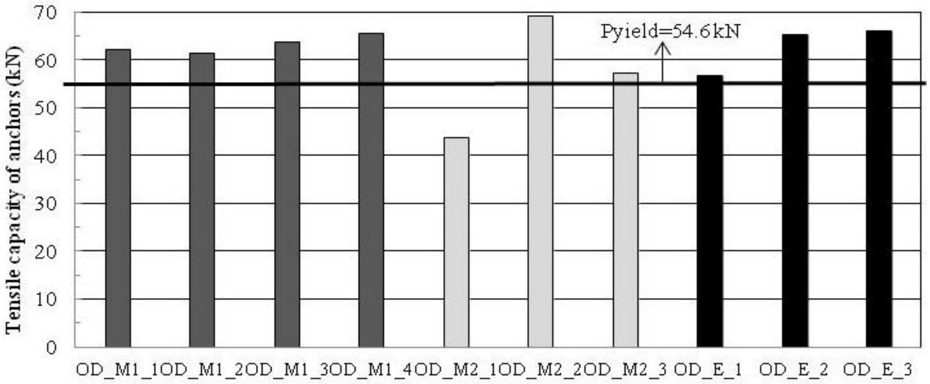


Fig. 5 Tensile capacities of outdoor anchors

a) OD_M1_1 b) OD_M1_3 c) OD_M2_3

Fig. 6 Bond failure mode of cement based type anchorage mortar specimens under tensile loads

For representing actual anchoring procedure properly, workers who normally work for anchoring tasks carried out all phases of anchoring application. The procedure included the drilling of the anchor holes, cleaning of the holes with a wire brush and compressed air, wetting of the anchor hole in case of cement based anchorage mortars to minimize the shrinkage of the mortar, pplication of sufficient amount of bonding material, placing the anchor bars into the hole by rotating to minimize the air gaps, and curing.

4. Experimental results

According to ACI 318-11, Appendix D[30], under tensile loads, anchors display six types of failure modes: steel failure, pullout failure, concrete breakout, concrete splitting, side-face blowout, and bond failure. The failure modes of the outdoor and indoor anchors are determined considering ACI 318-11[30].

4.1 Outdoor test results

The outdoor pullout test results are presented in Table 7 for three different types of anchors investigated in this study. As seen in this table, the average tensile strengths of different types of anchors varied between 57.0 and 63.9 kN. The interesting result of the pullout tests is the similarity of the average tensile strengths of the anchors of different types. While the average tensile capacities of the M1 and M2 type of anchors were 63.2 and 57.0 kN, the average pullout capacity of the E type anchors was 63.9 kN. Although the observed failure modes were different; namely steel fracture and bond failure; the difference in the failure mode did not cause a significant change in tensile capacities. This can be explained with approximately similar levels of the tensile capacities of the anchor bars and bond failure capacity of the anchors, which can be calculated considering the bond strength of the adhesive or anchorage mortar used. The tensile pullout capacities of all specimens are presented in Fig. 5 together with the tensile load corresponding to the yield strength of the anchor bars.

The tensile strengths and failure modes of four M1 type anchors were quite consistent. The tensile strengths of these specimens varied between 61.4 and 65.5 kN with an average of 63.2 kN. All M1 type anchors failed through the bond failure (Fig. 6 a,b). As seen in Fig. 5, the tensile capacities of all M1 anchors were higher than the tensile force corresponding to yielding of the anchor bar.

Three M2 type anchors were tested under pullout forces. The tensile strengths of these specimens varied between 43.8 and 69.9 kN, with an average of 57.0 kN. As seen in Fig. 6, two of M2 anchors resisted higher forces than the tensile force corresponding to yielding of anchor bars, whereas the tensile capacity of OD_M2_1 anchor was lower than that. The relatively earlier failure of this anchor is attributed to potential problems due to difficulties of the application on site. The successful performance of M2 type anchors are further validated through indoor tests as explained below. Nevertheless, OD_M2_1 anchor could resist the tensile load of approximately 44 kN (81 % of the tensile force corresponding to yielding of anchor bar) and failed through a combined bond failure with a small cone formation and slip of remaining part of the anchor (Fig 6c). As seen in Table 7 and Fig. 6, while OD_M2_2 and OD_M2_3 anchors reached the yield strength of anchor bars, the failure modes were different. OD_M2_2 anchor failed because of the fracture of the anchor bar, after yielding of the anchor bar, whereas bond failure mode was observed for OD_M2_3. The difference between the failure modes of anchors is deemed natural considering the site conditions during testing and potential variation of concrete quality within the building. Like M1 type anchors, the E type anchors also showed a consistent performance with little scatter in tensile strength. They all failed because of fracture of the anchor bars after yielding. The tensile capacities of these anchors varied between 57.0 and 68.1 kN with an average of 63.9 kN. The variation of tensile strength is attributed to the variation of mechanical characteristics of anchor bars and potential marginal differences of the effective diameter of bars. These findings as well as further validation of these results through indoor tests as explained below clearly demonstrate the

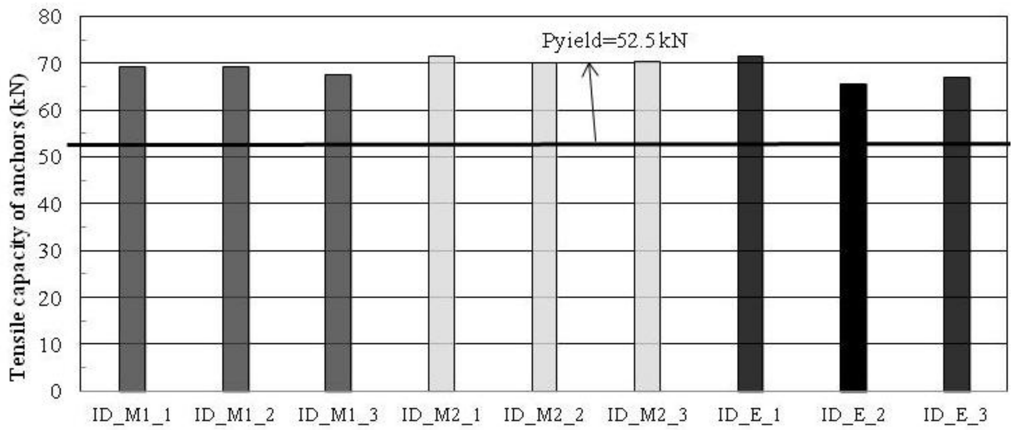


Fig. 7 Tensile capacities of indoor anchors

Table 8 Test results of indoor anchors

Specimen	Hole diameter (mm)	Failure mode	Tensile load (kN)	Average tensile load (kN)	Std. deviation	Coeff. of Variation (%)
ID_M1_1	22	Steel failure	69.1			
ID_M1_2	22	Steel failure	69.3	68.6	1.0	1.4
ID_M1_ 3	22	Steel failure	67.5			
ID_M2_ 1	22	Steel failure	71.6			
ID_M2_ 2	22	Bond failure	70.1	70.6	0.8	1.2
ID_M2_ 3	22	Steel failure	70.3			
ID_E_ 1	18	Steel failure	71.6			
ID_E_ 2	18	Steel failure	65.5	68.0	3.2	4.7
ID_E_ 3	18	Steel failure	66.8			

efficiency of cement based anchorages for retrofit applications on existing substandard buildings, that had been constructed with low strength concrete.

4.2 Indoor test results

The indoor pullout test results are presented in Table 8. As seen in this table, average tensile strengths of the anchors varied between 68.0 and 70.6 kN. The tensile capacities of all anchors (either cement or epoxy based) were higher than yielding capacity of the anchor bars (Fig. 7). Except ID_M2_2, all anchors failed due to fracture after yielding, whereas bond failure mode was observed for ID_M2_2 after yielding of the anchor bar.

The tensile load-slip relationships of the cement based (M1 and M2) and epoxy based (E) indoor anchors are presented in Figs. 8-10. The curves given in Figs. 8-10 clearly demonstrate the ductile steel bar behavior in tension both for cement and epoxy bonded indoor anchors. As an exception, in case of ID-M2-2, bond failure mode was observed after the anchor reached the ultimate tensile load. Nevertheless, although the failure mode was different from other indoor

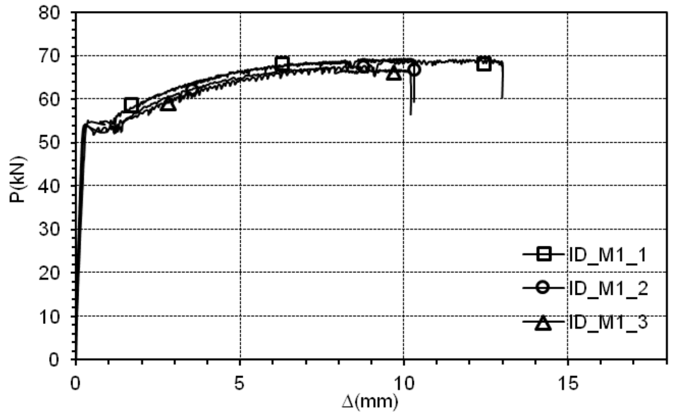


Fig. 8 Load-displacement relationships of the M1 type indoor anchors

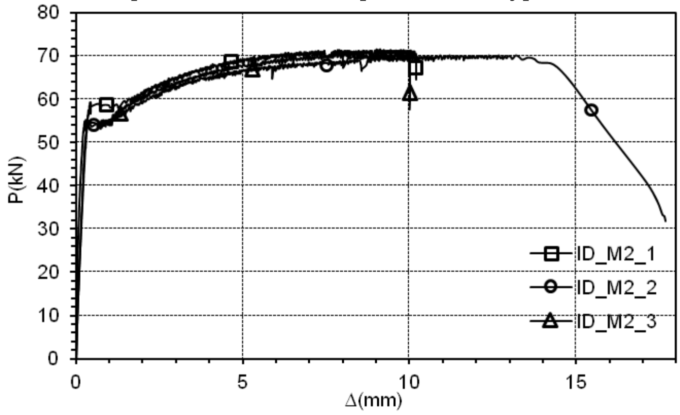


Fig. 9 Load-displacement relationships of the M2 type indoor anchors

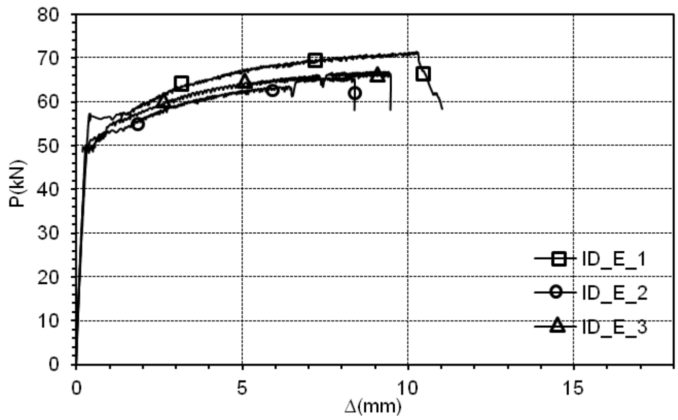


Fig. 10 Load-displacement relationships of the E type indoor anchors

anchors, the tensile capacity of ID-M2-2 had no significant difference with respect to other cement and epoxy based indoor anchors. As seen in Table 8 and Figs. 7-10, the tensile capacities, failure modes and tensile load-slip relationships of M1, M2 and E types of anchors were consistent and quite similar to each other. In case of indoor tests, scatter of test results were much less and tensile capacities of cement and epoxy based anchors were higher with respect to outdoor tests (Table 7 and 8). The difference between tensile strengths of M1 and E type anchors were about 10%, where this was around 20% for M2 type anchors. The higher scatter and lower anchorage capacities in case of outdoor tests can be attributed to variation of concrete quality during outdoor tests as well as difficulties of testing under outdoor conditions.

4.3 Tests of partially bonded anchors

For obtaining the bond strength of cement based mortars and epoxy based adhesive, partially bonded anchor pullout tests were executed. For avoiding cone type failure as well as anchor bar fracture, upper parts of the anchor bars were left unbounded. The length of bonded bottom part of the anchorage bars was only 60 mm (5∅). For determining the bond strength of partially bonded anchors, a uniform bond stress model is used. In this model, the bond stress is assumed to be constant along the bonded length of the anchor.

The characteristic tensile load capacity is evaluated with Eq.(1), where K is tolerance factor corresponding to a 5 percent probability of nonexceedence with a confidence of 90 percent, F_k is characteristic value of tensile load capacity (N), $\bar{F}_{test,x}$ is mean tensile capacity (N) and $v_{test,x}$ is coefficient of variation of the population sample corresponding to test series (percent)[31]. K values are provided by ACI 355-4M-11[31] according to the number of tests (*K*=5.311).

$$F_k = \bar{F}_{test,x}(1 - K.v_{test,x}) \tag{1}$$

The nominal characteristic bond stress in tension is obtained by dividing calculated characteristic value of tensile load capacity to failure surface area. It should be noted that, the failures of M1, M2 and E type partially bonded anchors were due to debonding at the steel/mortar or steel/adhesive interface (Fig. 11). To obtain characteristic bond strength for mandatory and optional use conditions, the nominal characteristic tension bond strength is reduced by reduction factors in accordance with ACI 355.4M-11[31]. The reduction factors are used for sustained tension loading, seismic tension loading and various service conditions. In this experimental study, the nominal characteristic bond stress reduction factors are taken as 1.0 due to the laboratory

(a) ID_M1_P_1 b) ID_E_P_1

Fig. 11 Failure modes observed in partially bonded anchors

Table 9 Test results of indoor partially bonded anchors

Specimen	Failure mode	Failure surface	Tensile load capacity (kN)	Average tensile load capacity $\bar{F}_{test,x}$ (kN)	Standart Deviation (kN)	Coeff. of Variation $v_{test,x}$	Charact. tensile load capacity F_k(kN)	Charact. bond strength (MPa)
ID_M1_P_1	Bond f.	Anchor/Mortar	46.6					
ID_M1_P_2	Bond f.	Anchor/Mortar	49.5	48.0	1.45	0.030	40.4	17.9
ID_M1_P_3	Bond f.	Anchor/Mortar	47.9					
ID_M2_P_1	Bond f.	Anchor/Mortar	27.1					
ID_M2_P_2	Bond f.	Anchor/Mortar	27.5	27.5	0.35	0.013	25.6	11.3
ID_M2_P_3	Bond f.	Anchor/Mortar	27.8					
ID_E_P_1	Bond f.	Anchor/Adhesive	39.2					
ID_E_P_2	Bond f.	Anchor/Adhesive	39.1	40.5	2.39	0.059	27.8	12.3
ID_E_P_3	Bond f.	Anchor/Adhesive	43.3					

conditions, static and monotonic nature of loading. It should be noted that, the minimum characteristic bond stress values given by ACI 355.4M-11[31] are 4.5 MPa for outdoor conditions and 7.0 MPa for indoor conditions. In Table 9, the characteristic bond stresses, standard deviations and coefficient of variations (COV) of M1, M2 and E type bonding materials are presented.

5. Comparison of experimental results and current design models

In this experimental study, steel failure, concrete breakout and bond failure mechanisms are expected to occur according to ACI 318-11[30]. Therefore, to make a prediction of the anchor capacities and the failure modes of the anchors, these three failure mechanisms are explained in this part. In addition to ACI 318[30], the technical documents of *fib*-Bulletin 58[32] and EOTA-TR 029[33] are used to evaluate the findings of the experimental study. Therefore, the experimental results and predicted anchor capacities and failure modes are compared considering different technical documents.

According to ACI 318-11, Appendix D[30], for steel failure mode, the nominal steel strength of an anchor in tension, N_{sa}, can be calculated by Eq. (2). Here, $A_{se,N}$ is the effective cross sectional area of an anchor in tension, mm^2, and f_{uta} is the specified tensile strength of anchor steel, MPa. The nominal steel strength is reduced by a reduction factor of 0.80 (for ductile steel elements) to determine anchor design strength. In the technical documents of *fib*-Bulletin 58[32] and EOTA-TR 029[33], the reduction factor is 0.67.

$$N_{sa} = A_{se,N} f_{uta} \tag{2}$$

In ACI 318-11, Appendix D[30], the basic concrete breakout strength, N_b, of a single anchor unaffected by edge distance or superposition of cones is given by Eq. (3). In this model, the capacity of a single anchor is derived assuming a concrete failure prism with an angle between the concrete failure surface and surface of the concrete member (about 35 degrees) considering fracture mechanism concept[11,30] (Fig. 12). In Eq. (3), k_c is 7 for post-installed anchors and 10 for cast-in anchors, λ_a is a modification factor for lightweight concrete, f_c' is specified compressive strength of concrete, MPa, and h_{ef} is the effective embedment depth of the anchor, mm. The k_c factor is defined differently in the technical documents of *fib*-Bulletin 58[32] and EOTA-TR 029[33]. In *fib*-Bulletin 58[32], the k_c factor is 7.7 and 11.0 in case of cracked and uncracked concrete, respectively. In EOTA-TR 029[33], these values are replaced with 7.2 and 10.2. Additionally, the characteristic concrete compression strength, f_c', is determined by cylinder tests (150x300 mm) in ACI 318-11[30] and *fib*-Bulletin 58[32], and by cube tests with side length 150 mm in EOTA-TR 029[33].

$$N_b = k_c \lambda_a \sqrt{f_c'}\, h_{ef}^{\;1.5} \tag{3}$$

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} \mathrm{N}_b \tag{4}$$

The nominal concrete breakout strength of an anchor, N_{cb}, in tension is given by Eq. (4). The horizontal dimension of the failure surface of idealized concrete cone as a pyramid is about three times the embedment length (Fig. 12)[11,30]. Therefore, the projected concrete failure area of a single anchor, A_{Nco}, at the concrete surface is equal to $9h_{ef}^2$. A_{Nc}, is the modified projected area at the concrete surface determined as a function of edge distance and neighboring anchors. However, since no edge effects or neighboring anchors are present in this study, A_{Nc} is taken equal to A_{Nco}. The modification factors $\Psi_{ec,N}, \Psi_{ed,N}, \Psi_{c,N}$ and $\Psi_{cp,N}$ are related with the group effects, edge effects, cracking and splitting of concrete, respectively. In this study, $\Psi_{ec,N}, \Psi_{ed,N}, \Psi_{c,N}$ and $\Psi_{cp,N}$ modification factors are used as 1.0, 1.0, 1.4 and 1.0, respectively. The $\Psi_{c,N}$ *and* $\Psi_{cp,N}$ modification factors do not exist in the technical documents of *fib*-Bulletin 58[32] and EOTA-TR 029[33]. However, in these documents, the shell spalling factor, $\Psi_{re,N}$, is taken into account for the effect of closely spaced reinforcement. In ACI 318-11, Appendix D[30], the nominal concrete breakout strength, N_{cb}, is reduced by strength reduction factor of 0.75 to obtain anchor design strength of post-installed anchors under tension loading. The strength reduction factor is 0.67 in *fib*-Bulletin 58[32] and EOTA-TR 029[33].

In ACI 318-11, Appendix D[29], the basic bond strength of a single anchor, N_{ba}, is given by Eq. (5). This equation is based on the assumption of uniform bond stress distribution along the bonded embedment depth. In this equation, τ_{uncr} is the characteristic bond strength in uncracked concrete (N/mm^2), d_a is the diameter of anchor bar (mm), h_{ef} is the effective embedment depth of the anchor (mm). The nominal bond strength in tension, N_a, of a single adhesive anchor can be calculated by Eq. (6). According to ACI 318-11, Appendix D[30], the modification factors in Eq. (6) are similar to the factors in Eq. (4). The nominal bond strength, N_a, is reduced by reduction factor of 0.75 to obtain anchor design bond strength under tension loading. In *fib*-Bulletin 58[32] and EOTA-TR 029[33], the reduction factor is 0.67.

$$N_{ba} = \lambda_a \tau_{uncr} \pi\, d_a h_{ef} \tag{5}$$

$$N_a = \frac{A_{Na}}{A_{Nao}} \Psi_{ec,Na} \Psi_{ed,Na} \Psi_{cp,Na} \mathrm{N}_{ba} \tag{6}$$

The theoretical design capacities for different failure modes calculated using the equations given in these technical documents are presented in Table 10. In this table, experimental tensile loads and failure modes of the anchors are also given. The table is prepared for indoor and outdoor anchors. It is clearly seen from Table 10 that, all technical documents predicted the lowest theoretical design strength for steel fracture. Furthermore, the predicted anchor strengths by all three documents are below the experimentally determined anchor strengths for cement and epoxy based anchors (except M2 type cement based outdoor anchors according to ACI 318-Appendix D[30]). This indicates that the investigated design documents are sufficiently conservative for the

Table 10 Comparison of indoor/outdoor anchors tensile test results and prediction of available models

Specimen	Exp. Average Tensile Load (kN)	Exp. failure mode	ACI 318-11,Appendix D[30]			*fib*-Bulletin 58[32]			EOTA TR 029[33]		
			Steel strength (kN)	Bond strength (kN)	Concrete breakout strength (kN)	Steel failure (kN)	Combined failure (kN)	Concrete Cone Failure (kN)	Steel failure (kN)	Combined failure (kN)	Concrete Cone Failure (kN)
ID_M1	68.6	Steel f.		100.7			89.5			89.5	
ID_M2	71.0*	Steel f.	52.6	64.0	78.3	43.8	56.9	78.2	43.8	56.9	80.2
ID_E	68.0	Steel f.		69.5			61.8			61.8	
OD_M1	63.2	Bond f.		100.7			89.5			89.5	
OD_M2	50.6**	Bond f.	54.6	64.0	65.1	45.5	56.9	64.9	45.5	56.9	66.7
OD_E	63.9	Steel f.		69.5			61.8			61.8	

* Except ID_M2_2 (Bond failure)
** Except OD_M2_2 (Steel failure)

Table 11 Theoretical capacities by considering installation safety factor

Specimen	Exp. Ave-rage Tensile Load (kN)	Exp. failure mode	ACI 318-11,Appendix D[30]			*fib*-Bulletin 58[32]			EOTA TR 029[33]		
			Steel strength (kN)	Bond strength (kN)	Concrete break out strength (kN)	Steel failure (kN)	Combined failure (kN)	Concrete Cone Failure (kN)	Steel failure (kN)	Combined failure (kN)	Concrete Cone Failure (kN)
OD_M1	63.2	Bond f.		73.9			63.9			63.9	
OD_M2	50.6*	Bond f.	54.6	46.9	47.7	45.5	40.6	46.4	45.5	40.6	46.4
OD_E	63.9	Steel f.		51.0			44.1			44.1	

* Except OD_M2_2 (Steel failure)

cases of anchors tested indoor (for all anchors) and outdoor (for E and M1 type) anchors. As shown in Table 10, although the technical documents predicted failure mode of the indoor and outdoor anchors as steel fracture, M1 and M2 type cement based outdoor anchors failed due to debonding, and the predicted bond strength is above the experimentally observed bond strengths of the M1 and M2 type outdoor anchors.

The technical documents recommend usage of smaller reduction factors for considering the influences of potential problems that may occur during various phases of the application in case of imperfect conditions. In such cases, it is specified to use a reduction factor of 0.55 to calculate bond strength and concrete breakout strength in ACI 318-Appendix D[30]. In *fib*-Bulletin 58[32] and EOTA-TR 029[33] this value is 0.48. Table 11 presents the predicted tensile capacities by considering these reduction factors for outdoor anchors. As seen in Table 11, the technical design documents are sufficiently conservative for all outdoor anchors when lower reduction factors are taken into account. However, except M2 type cement based anchors, the technical documents are not able to estimate the experimental failure modes correctly.

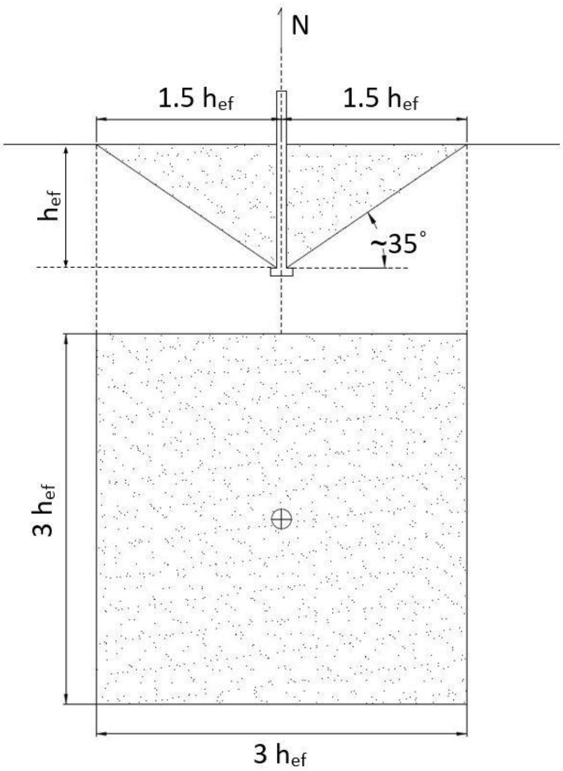


Fig. 12 Breakout failure cone and plan for tension

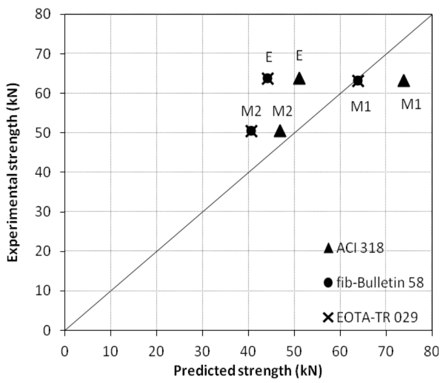


Fig. 13 Comparison of experimental tensile load and predicted bond strength capacities of outdoor anchors

The experimental tensile loads and the predicted bond strengths of outdoor anchors are compared in Fig. 13. As seen in the figure, with increasing bond strength of the bonding materials, the bond capacity is slightly overestimated by *fib*-Bulletin 58[32] and EOTA-TR 029[33] and slightly overestimated by ACI 318-11, Appendix D[29]. Therefore, an additional safety factor is recommended, particularly for ACI 318-11, Appendix D[29] in case of bonding materials with high bond strength.

6. Economical comparison of the bonding materials

Thousands of anchors are used during seismic retrofit of ordinary reinforced concrete structures. Therefore, cost of adhesive material used for anchors is among governing factors of total cost of retrofit application. Therefore, any intervention towards reduction of anchor costs may significantly reduce retrofit cost. The test results showed that both cement and epoxy based bonding materials were almost equally successful. Proven that the performances of cement based and epoxy based bonding materials are similar, a cost analysis is carried out to examine the bonding materials in terms of their economical feasibility. During the economical analyses, material costs, as well as drilling costs are taken into account. Cost per anchor is calculated considering a 12 mm diameter anchor bar, which is to be anchored in an embedment depth of 200 mm. Consequently, the cost ratios of the M1, M2 and E type anchors are (1.35; 1.00; 6.90), respectively. It is clearly seen that using cement based mortars instead of epoxy based adhesives can significantly reduce overall seismic retrofit costs of existing structures built with low strength concrete.

7. Conclusions

In this study, the pullout behavior of deformed bars that are anchored in low strength concrete is investigated. The test parameter was the type of bonding material and outdoor/indoor application/testing conditions. The experimental study was carried out outdoor on a 2 storey substandard reinforced concrete building to resemble realistically the actual buildings that need seismic retrofitting and indoor on concrete blocks produced at Istanbul Technical University Structural and Earthquake Engineering Laboratory. The obtained test results are also compared with the anchorage strength values obtained by using available technical documents. The conclusions are summarized below;

• The cement based mortars could provide pullout strengths similar to that of the anchors bonded with epoxy based adhesive. In case of indoor tests, the failures were due to fracture of anchor bars after yielding both for the cement based anchorage mortars and epoxy based adhesive. The experimental results showed that, for the considered range of effective parameters (i.e. concrete strength, reinforcement type and diameter, anchor hole depth, loading pattern etc.) properly designed cement based mortars can effectively be used for anchoring instead of epoxy based adhesives for sufficient tensile resistance. The outdoor cement based mortar and epoxy based adhesive anchors also failed after yielding of the steel bars. However, during outdoor tests, cement based mortar anchors failed due to debonding, while epoxy based anchors failed due to fracture of the anchor bars. Although failure modes were different, the tensile capacities of all of the anchors reached the yield strength of the anchor bars (with the exception of OD_M2_1).

• The indoor anchors behaved more consistently with less scattering of tensile strengths than outdoor anchors. The site conditions, variation of concrete quality and more difficult testing conditions on site might have led relatively less consistent test results for outdoor anchors.

• The technical design documents of ACI 318-Appendix D[30], *fib*-Bulletin 58[32] and EOTA-TR 029[33] are conservative for all indoor anchors. These documents are also conservative for outdoor anchors when the additional reduction factors for considering the imperfections are taken into account.

• The bond failure load was determined slightly higher than experimental loads by *fib*-Bulletin 58[32] and EOTA-TR 029[33], whereas ACI 318-Appendix D[30] significantly overestimated the bond strength for high strength bonding materials in low strength concrete. Therefore, an additional safety factor can be taken into account for calculation of bond capacity of materials with high bond strength.

• Considering the large stock of existing substandard buildings, which need seismic retrofitting, and limited financial resources that can be allocated for this task, the possibility of utilization of cement based bonding materials instead of remarkably more expensive epoxy based adhesives may bring important advantages in terms of reducing the retrofit cost, which is a considerable obstacle in front of widespread seismic retrofit applications.

• For reaching more general conclusions, particularly for seismic retrofitting applications, a wider range of effective parameters as well as effects of reversed cyclic loading conditions and aging under environmental conditions should be further investigated.

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