A Plate Type Edge-Lift Anchor: Influence of Reinforcing Configurations on Failure Loads

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Synopsis: AS3850 Tilt-up Concrete Construction [1] is the current standard for design of lifting anchors and bracing in Australia. However, this standard does not provide a recommendation for calculating the capacity of edge-lifting anchors (normally placed in the edge of thin wall elements) which are commonly used in the precast industry throughout Australia. These anchors may experience a load under tension or combined tension and shear during the lifting process.

The American Concrete Institute (ACI)318-08 Building Code Requirements for Structural Concrete [2], the Precast Concrete Institute (PCI) Design Handbook – Precast and Prestressed Concrete [3], and Comité Euro-International Du Béton(CEB)Design of Fastenings in Concrete [4] include provisions for general anchorage configurations (such as multiple face lift anchors) rather than what are typically seen in edge-lifting anchors. Not only are anchor configurations for edge lifting anchors different from those described in these standards, but the reinforcement around the anchor can vary significantly to those denoted in standards.

This paper is an evaluation of pull out test data for edge lift anchors in thin walled elements. Using the formula in the ACI 318-08 [2], developed predominantly for footed anchors, comparisons of the predicted capacity and the test pull out capacity of the edge lift anchors is made. Data is presented on 154 tests; the variables tested include concrete compressive strength at time of testing and the provision and arrangement of reinforcement. In addition to the edge lift anchors, 90 face lift footed anchors were also tested and some of the more relevant data is presented for these tests.

Keywords: Edgelift anchors, precast, plate anchors, lifting inserts, anchor capacity.

1. Introduction

Edgelift anchors (lifting inserts) are used to transfer lifting loads between steel and concrete. Unlike footed lifting anchors that have been investigated by numerous researchers worldwide, including work focused on anchoring either to reinforced or pre-stressed concrete [5,6,7], edge lift anchors are relatively un-examined in published research. The most advanced knowledge regarding anchorage to concrete is included in ACI 318-08 Appendix D [2], CEB Design of Fastening in Concrete [4] and PCI Design Handbook [3]. Recommended design solutions are also included in ACI 318-05 Appendix B that were meant to ensure the ductile behavior of cast-in-place anchors [8]. The ACI standard requires the tensile strength of the anchor to be less than or equal to the tensile strength of an idealized concrete cone surface Figure 1. ACI-349 Guide to the Concrete Capacity Design (CCD) Method - Embedment Design Examples [9] in turn, incorporates the approaches presented in ACI 318-05 Appendix D 2005 [8].

This paper summarizes the pull out failure data (under direct tension loads applied in a load controlled manner) of 154 edgelift anchors embedded in concrete panels with and without reinforcement provided near the anchor. Various configurations of reinforcement were tested in conjunction with the edge lift anchors; including with or without a shear bar, with or without panel mesh and with or without a perimeter bar. The configurations were chosen on the basis of common standard practice and recommendation. Grade 350, 16 mm thick edge lift plate anchors were tested in direct tension by pull out tests of the anchors in 150mm thick, 2m x 2m panels. The tests were conducted using normal weight Portland cement concrete with a compressive strength at the time of testing of at least 10 MPa and up to 40 MPa. The minimum strength recommended for lifting is 15MPa but lower compressive strengths were included as a lower bound that may occur in practice.

The pull out failure loads were compared to the predicted capacities as determined by design provisions provided in ACI 318-08 Appendix D (2008) [2] which have been developed from the basis of extensive footed anchor tests.

2. Predictive Strength Equations and Their Historical Development

Two similar methodologies exist to predict brittle tension failure of an anchor, being the 45^o cone method and the concrete capacity design (CCD) method, being a square-pyramidal failure surface with 35^o inclination. For the 45^o cone method the concrete strength of an anchor is calculated assuming a conical surface (Fig. 1) taking the slope between the failure surface and the concrete surface as 45^o. As the depth of embedment of the lifting insert increases, the area of the conical section increases proportionately up to the point of full embedment. Following this capacity guideline and test data, Nelson Stud Welding [10] stated that an embedment depth of 8 to 10 times the anchor shank diameter, for footed anchors, was required for the concrete breakout strength to be larger than the tensile strength of the steel in headed anchors. In relation to plate edgelift anchors the minimum stress area of the anchors cross section can be matched to follow this design recommendation. Cannon et al. [11] proposed to calculate anchors subjected to tension, shear and combined loads. The resulting recommendations included using a conical failure surface to calculate the tensile strength and were adopted in ACI 349-80 [12]. The design strength of concrete for anchorage was based on a uniform tensile stress of $ø_t$ (4 $\sqrt{f'_c}$).The resistance factor, $ø_t$ was 0.65. PCI (1978) adopted the conical failure surface to predict a brittle failure of the concrete and this method was retained in PCI 5th Ed (1999) [13,14]. However, PCI later adopted the provisions in ACI 318-02 Appendix D (2002), which are based on Concrete Capacity Design (CCD), to calculate the tensile strength of anchors assuming uncracked concrete [15,3]. In the CCD method the concrete strength of a single anchor is calculated assuming a four-sided pyramid failure surface, with a slope between the failure surface and the surface of the concrete member of 35^o. The more recent versions of ACI 318 Appendix D 2008) [2] and 2005 [8] use this approach.

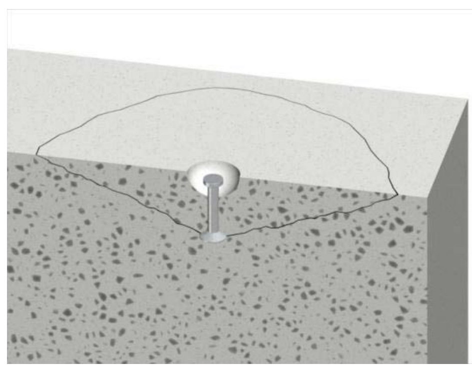


Figure 1. Conical failure surface of earlier editions of PCI Handbook [12]

A comprehensive state-of-the-art of cast-in-place and post installed anchors are included in ACI 355 Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D(1997) & CEB (1994) [16,17] These reports summarise the basis for current, general anchorage provisions of embedded anchors subjected to tension and tension plus shear interaction.

In the PCI editions, there have been several formulations to compute the tensile strength of an anchor. A conical failure surface for an anchor in tension was adopted up to PCI (1999), as presented in Table 1, with a resistance factor of 0.85 [13].As discussed earlier, PCI (2004) [3] changed the approach to a four sided pyramid cone, adopting a similar formulation to ACI (2008, 2005) [2,8], though working with coefficients related to uncracked concrete. The results given by PCI (2004) then correspond with results given by the ACI318-08 Appendix D (2008). Concrete failure occurs, when not influenced by edge conditions, when the minimum of either the pull out strength or breakout strength is reached before the steel yield (or ultimate fracture) strength is reached. The expressions used to calculate the pullout and breakout strengths are presented in Table 1, presenting the 5% fractile formula (which is used as the nominal strength formula) for PCI 5th and distinguishing between the 5% fractile (nominal strength) formula and the average formula for the ACI 318-08 (CCD method), since the average formulae of CCD may be found elsewhere [6]. PCI 6th adopted the ACI 318 formulas in the particular case of uncracked concrete. The nominal strength (5% fractile) formula used in ACI 318 Appendix D for anchoring, such as Wollmershauser [18] reported, presents a 90% confidence that 95% of the anchor ultimate loads exceed the 5% fractile value. The formulas are summarized in the Table 1 for the various editions of codes and handbooks.

Table 1. Predictive Capacity Formulae for Steel Failure, Pull-out and Breakout [2, 3, 14]

	Concrete Failure Pull-out	**Concrete Failure Breakout**
PCI 5th Edition	No model included	$12.6h_{ef}(h_{ef}+d_h)\lambda\sqrt{f'_c}$
ACI 318-08 (5% fractile)	$8A_{brg}f'_c\psi_{c,p}$	$24\lambda\sqrt{f'_c}(h_{ef})^{1.5}\psi_{c,N}$
ACI 318-08 (Average)	$13A_{brg}f'_c$	$40\lambda\sqrt{f'_c}(h_{ef})^{1.5}$
PCI 6th Edition (5% fractile)	$11.2A_{brg}f'_cC_{crp}$	$3.33\lambda\sqrt{\frac{f'_c}{h_{ef}}}(9h_{ef}^{2})$

A_{brg} – bearing area of foot
f'_c – 28 day concrete compressive strength
$\Psi_{c,p}$ – concrete crack modification factor (1.4 for non-cracked) for pull-out strength
$\Psi_{c,N}$ - concrete crack modification factor (1.25 for non-cracked) for breakout strength
h_{ef} – effective embedment
d_h – diameter of the anchor head
λ – light-weight concrete modification factor

3. Experimental Program

3.1 Test Parameters

Sixty (60) footed anchors of Series 7 with a 50 mm effective embedment depth were cast in two reinforced concrete panels 2 m x 2 m x 150 mm thick with 30 anchors in each panel. The reinforcing was SL82 mesh and an N16 perimeter bar located 50 mm from the edge of the panel. These anchors were tested in direct tension as the concrete matured in order to precipitate concrete cone failures; tests were conducted at compressive strengths ranging from 18 MPa to 26 MPa, with an average of 21 MPa. Concrete compressive data for all series is shown in Table 2. All anchors of Series 7 failed due to concrete cone failure. The footed anchors were arranged with sufficient edge distances such that concrete capacity was not reduced due to edge effects.

Thirty (30) footed anchors of Series 6 were cast in unreinforced concrete blocks of 2m x 2 m x 0.6m deep with 2 anchors per block. The anchors of Series 6 were tested in direct tension once the concrete had matured. The compressive strength was 42 to 46 MPa, with an average compressive strength at time of testing (which was at 28 days) of 43 MPa (Table 2). This was to ensure the footed anchors failed due to steel tensile failure rather than a concrete cone failure. The footed anchors of series 6 were of varying embedment depth; 120 mm, 170mm and 240 mm effective embedment depth. All anchors of Series 6 failed due to steel tensile failure of the anchor. Series 6 anchor tests are mentioned here for context of the series notation; however, the data was not used in the analysis presented in this paper and is part of an ongoing research programme.

Plate-type edge lift anchor pull out tests (154 of) were conducted at concrete compressive strengths and embedment lengths that would precipitate a concrete cone failure. The edge lift anchors were 252mm, 272mm and 295mm effective embedment depth, 16 mm plate, with a profile as shown in Figure 2. They were cast in thin (150 mm thick) panels with varying reinforcement arrangements in the panels and around the anchors, described in more detail below.

The test specimens were identified in series of panels which were constructed with the same reinforcement provided in the panels. Series 1 test panels had no reinforcement in the panels (as seen in Figure 3(a)). Series 2 had N16 shear bars placed over the notch of the anchor and a centrally placed N16 perimeter bar which extended the length of the panel and was lapped at the corners of the panels (as seen in Figure 3(b)).Series 3 had no shear bar and had centrally placed SL82 mesh with centrally placed N16 perimeter bar. Series 4 had an N16 shear bar, centrally placed SL82 mesh and a centrally placed N16 perimeter bar. Series 5 had an N12 shear bar, centrally placed SL82 mesh and a centrally placed N16 perimeter bar. Details are summarised in Table 3 and shown in Figure 3.

Normal strength concrete was used throughout all series of the tests; being 14 mm coarse aggregate, 0.6 water/cement ratio, and nominal grade 40MPa design strength supplied by a commercial ready-mix company. The range of concrete compressive strengths at time of test was 10.1MPa to 40MPa, with an average of 21MPa. Full concrete compressive data for all series is shown in Table 2.

Table 2 – Concrete Compressive Data for Test Series

Test Series	f_{cm} mimimum (MPa)	f_{cm} maximum (MPa)	f_{cm} average (MPa)
1	18	26	21
2	16	28	23
3	10.1	36	18
4	15	40	22
5	15	35	23
6	42	46	43
7	18	26	21

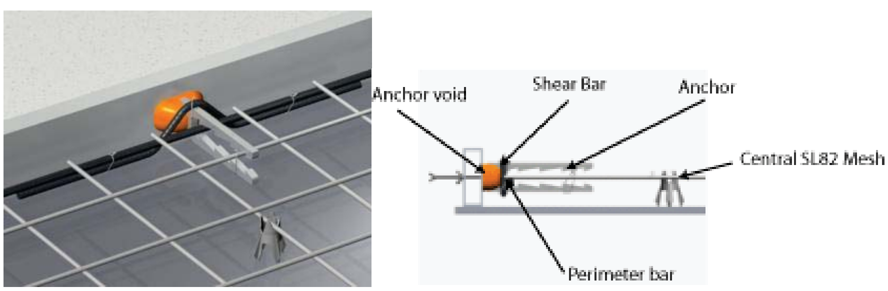


Figure 2. Reinforcement layout

Table 3. Reinforcement Configurations for Test Series

Test Series	N16 Shear bar	N12 Shear bar	Central SL82 mesh	N16 Perimeter bar
1	Nil	Nil	Nil	Nil
2	Yes	Nil	Nil	Yes
3	Nil	Nil	Yes	Yes
4	Yes	Nil	Yes	Yes
5	Nil	Yes	Yes	Yes
6	Nil (not applicable)	Nil (not applicable)	Yes	Yes
7	Nil (not applicable)	Nil (not applicable)	Yes	Yes

The preparation of the specimens for testing is shown in the Figure 3. Figure 3(b) shows a typical 2 m x2m x 150 thick panel formwork with N16 perimeter bar and 16mm x 295 effective embedment depth plate edge lift anchors in the form. As can be seen, this panel had two test anchors which was the typical arrangement. If, after testing one of the anchors, it was observed that cracking had propagated then the second anchor, whilst still tested, was excluded from the results presented in this analysis and paper.

(a) Series 1anchor with no reinforcement

(b) Series 2 prior to installation of shear bar

(c) Series 3 Test Panel prior to installation of perimeter bar with SL82 mesh

(b) Series 4 or 5

Figure 3 Typical Test Panels Prior to Casting

The anchors were loaded under load-control at a rate of 20 kN/min via a hydraulic jack with a load cell. The test data recorded for each specimen included load-displacement (of the anchor relative to a fixed point on the test panel or block) and load-time. The panels with edge lift plate anchors were tested horizontally and supported off the floor on timber gluts whilst the panel reacted against a steel frame with an open span of 1.8 m as the load was applied to the anchor. The spacing of the reaction frame for the anchors was outside the predicted failure zone for the concrete by at least 450mm as shown in Figure 4. The foot anchors embedded in the face of the panels and blocks were tested at the same loading rate in direct tension. The load was applied to the footed anchors via a tripod reaction frame with the legs of the reaction frame placed at a distance from the anchor of least three times the effective embedment depth of the anchor.

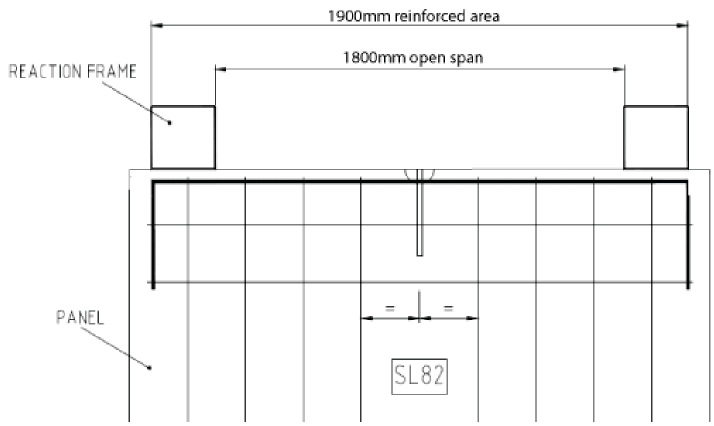


Figure 4. Panel plan indicating open span to the reaction frame (for edge lift plate anchor tests)

3.2 Test Results and Preliminary Analysis

From the analysis presented in Table 4 of the footed anchor tests which failed due to cone failure of the concrete (Series 7 test specimens), it can be said that the ACI 318-08 average concrete capacity approach (four sided pyramid) better predicts the behaviour of the concrete failure load due to the similar average value to the PCI 5th equation, but with a smaller standard deviation. This is since the model of the four-sided pyramid that forms a slope of 35^{o} with the horizontal surface better simulates the failure surface and therefore failure load when compared with the PCI (5th edition) 45 degree model. It should be noted that in all of these pull out tests, edge effects were not a factor in the failure. These test results align with previously published data [5, 6, 7]

Figure 5. Panel of Series 7 with failed footed anchor

Table 4. Assessment of tensile strength due to concrete formula of PCI 5th [13] and ACI 318-08 [2] for tests that failed in the concrete. (SERIES 7)

Concrete Capacity Formula - Footed Anchors	Test /Predicted	Number of tests
ACI318-08 Average concrete capacity	.55 - 1.65	60
PCI 5th Edition Concrete formula	.62 - 2.2	60
ACI318-08 5% fractile, Concrete formula	.63 - 2.1	60

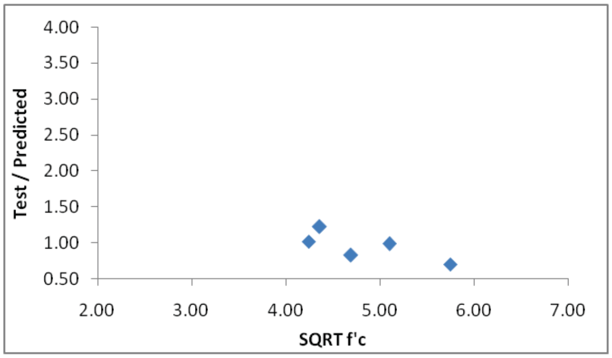
The edge lift anchor test data was compared with the predicted capacity as determined using the ACI 318-08 average capacity formula as a mechanism of comparison to the well-established relationship for foot anchors presented in literature and verified in the tests of Series 7. The ratio of the test failure load and the predicted load as per ACI 318-08 is shown in Table 5 and Figures 6 (a) through (e) for each series of panel tests with edge lift anchors. The ratio of test failure load to predicted failure load is plotted against the square root of the concrete compressive strength since the predicted strength is a function of and directly proportional to the square root of compressive strength.

For the two series of panels with edge lift anchors and no central mesh reinforcement in the panels; Series 1 and Series 2, the following observations were made: The addition of shear and perimeter bars (Series 2) resulted in a slightly increased absolute failure load for comparable tests and is indicated by a slightly higher average ratio of test/predicted compared to Series 1. Since the manufacture of panels without central mesh is impractical, the number of tests conducted was small; however, the test results are valuable as an indicator that the provision of the perimeter bars is likely to be beneficial to the capacity of the anchor. Thus this detail (N16 perimeter bar) along with central panel mesh of SL82 was subsequently used in Series 3, 4 and 5.

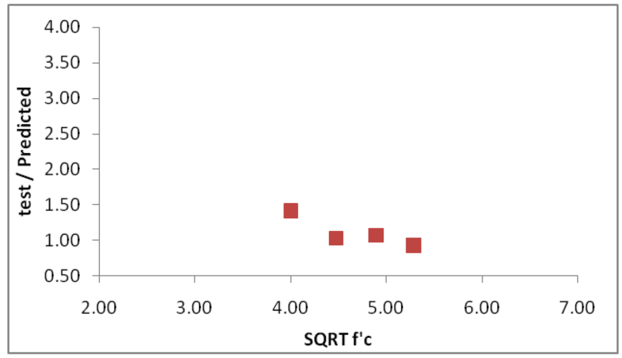
For the three series of panels with central mesh reinforcement and N16 perimeter bar in the panels; Series 3, Series 4 and Series 5, the following observations were made: Series 3, the edge lift anchors with no additional N12 or N16 shear bar reinforcement, has a significantly higher capacity than the unreinforced panels as indicated by the value of test/predicted ratio average of almost 2.5. The two series with additional shear bar of either N12 or N16 had similar average and range of test/predicted apparently less than the panels without shear bars; however, Series 3 had more tests conducted at lower concrete strengths and it is these test results which appear to magnify the average ratio and the standard deviation of the data for Series 3. Further analysis of the data for trends is being undertaken. Additionally, it should be noted that the anchors were tested in tension only and at this stage of the research programme, shear tests and combined tension/shear tests have not been completed. Analysis of the impact or potential benefit (or otherwise) of the shear bars can only be made after further testing in shear and combined loading has been finalised. These tests will simulate the anchor loading as the panels are tilted from the horizontal position to the vertical position where the anchor is engaged in tension.

Table 5. Assessment of tensile strength due to concrete formula of ACI 318-08 [2] for panel tests conducted with edge-lift anchors

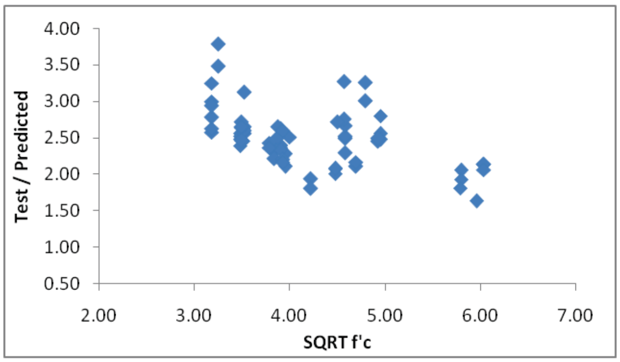
Series	N, number of Tests	Test / Predicted Range	Average Test/Predicted	Standard Deviation Test/Predicted
Series 1: No reinforcement	5	.71 - 1.22	0.96	0.2
Series 2: N16 shear bar, N16 perimeter bar	6	.92 – 1.41	1.07	0.18
Series 3: SL82 mesh, N16 perimeter bar	77	1.63 – 3.79	2.49	0.4
Series 4: N16 shear bar, SL82 mesh, N16 perimeter bar	29	1.52 - 2.69	1.90	0.3
Series 5: N12 shear bar, SL82 mesh, N16 perimeter bar	37	1.64 - 2.19	2.22	0.15



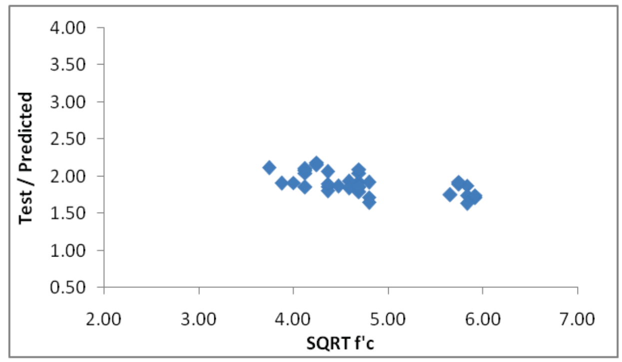
(a) Series 1



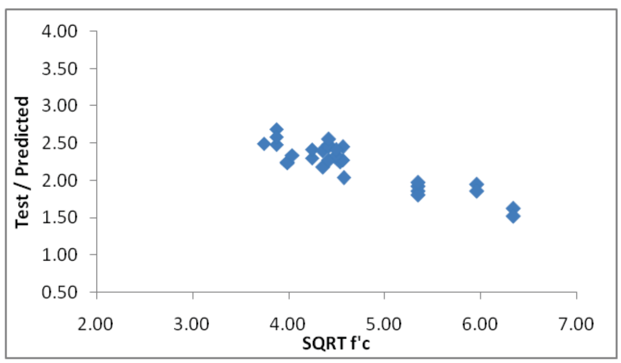
(b) Series 2



(c) Series 3



(d) Series 4



(e) Series 5

Figure 6 Ratio of Test Failure Load to Predicted for each Series of Plate Edge Lifters

4. Concluding remarks

This paper is an evaluation of pull out test data for edge lift anchors in thin walled elements. Using the formula in the ACI 318-08 [2], developed predominantly for footed anchors, comparisons of the predicted capacity and the test pull out capacity of the edge lift anchors is made. Three series of panels were reinforced with centrally placed SL 82 mesh, and the ratios of test to predicted failure load indicate that the capacity of these anchors was well in excess of the predicted failure load as per ACI 318-08, of the order of approximately 1.5 to 3.8 times.

Overall, 154 tests were conducted using edge lift anchors in direct tension; the variables tested include concrete compressive strength at time of testing which ranged from 10 to 40 MPa with an average of 21 MPa, and arrangement of reinforcement which included the provision or exclusion of perimeter bars, and shear bars (N16, N12 or nil) and central mesh reinforcement in the panel. Further analysis of the data and the trends indicated from the test results are currently underway. It is anticipated that a predictive mode,l with improved capacity to simulate and thus predict the failure of edge lift anchors, will be developed.

5. Acknowledgement

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6. References

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