A Plate Type Edge-Lift Anchor: Shear Reinforcement Influence on Failure Loads

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Synopsis: The published standard [1] and Industry code of precast practice [2] do not provide a recommendation for calculating the shear capacity of edge-lifting anchors (which are normally placed in the edge of thin wall elements) and are commonly used in the precast industry throughout Australia. These anchors experience loads under tension or combined tension and shear during the lifting process. A load applied perpendicular to an anchor is commonly referred to as a shear load. When a shear load is applied, the anchor reinforcement, typically a shear bar, will provide the majority of the concrete breakout strength. This paper is an evaluation of anchor shear reinforcement test data for edge lift plate anchors in thin walled elements. References and comparisons are made with the formula in ACI 318M-08 [3], which was developed for cast-in anchors, and comparisons of the predicted capacity and the tested shear concrete breakout capacity of the edge lift anchors, with shear reinforcement, is made. Data is presented on 137 tests; the variables tested include concrete compressive strength at time of testing, anchor width, panel thickness, shear reinforcement embedment depth and shear reinforcement diameter. What this paper shows is that the mechanical interactions of a typical shear bar design does behave in a way that can be suitably predicted by the model presented in ACI318M-08 D5.2. This paper examines the failure mechanisms of a typical shear bar and highlights potential installation issues.

Keywords: Edgelift anchors, precast, plate anchors, lifting inserts, anchor capacity, shear lifting, lateral tensile loads in precasting, shear reinforcement.

1. Introduction

Plate edgelift anchors (lifting inserts) are used to transfer lifting loads between steel and concrete. Unlike footed anchors that have been investigated by numerous researchers worldwide, edgelift plate anchors are relatively un-examined in published research. Advanced knowledge regarding anchorage to concrete is included in ACI 318M-08 Appendix D [3], CEB Design of Fastening in Concrete [5] and PCI Design Handbook [4]. Recommended design solutions are also included in ACI 318-05 Appendix B [6] that were meant to ensure the ductile behavior of cast-in-place anchors. The ACI standard requires the actual tensile capacity of an anchor be greater than or equal to the calculated tensile strength of an idealized concrete cone surface, which is then used in precast lifting design, refer figure 5. ACI-349 Guide to the Concrete Capacity Design (CCD) Method - Embedment Design Examples [7] in turn, incorporates the approaches presented in ACI 318-05 App D 2005 [6].

This paper summarizes the concrete breakout strength data (placed under shear loads applied in a load controlled manner) of 137 edgelift anchors embedded in concrete panels with various reinforcement (shear anchoring) configurations on top of the anchor, shown in figure 1 and figure 2..

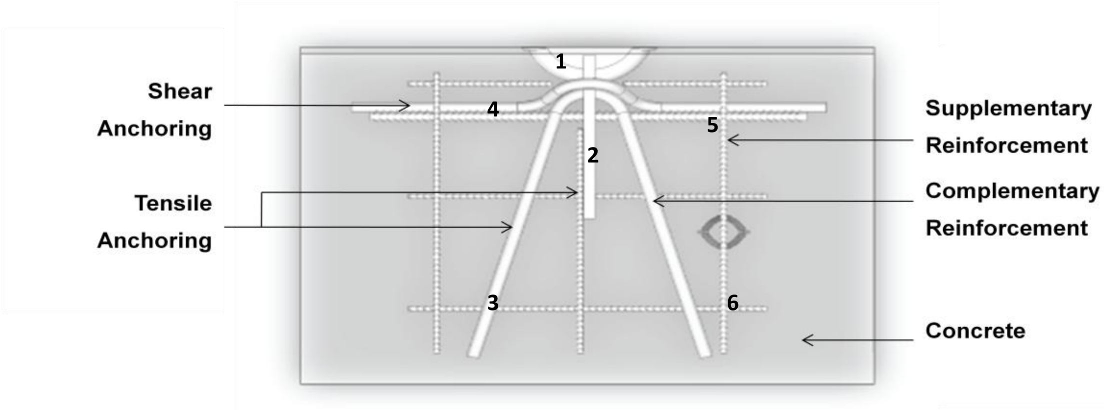

Fig 1: Plan view of a typical panel setup for edgelifting

Figure 1 shows the plate edgelift anchor and the strength contributing steel reinforcement layout, with the anchor void (1), the plate edgelift anchor (2), tension bar (3), shear bar (4), perimeter bar (5), and panel mesh (6).

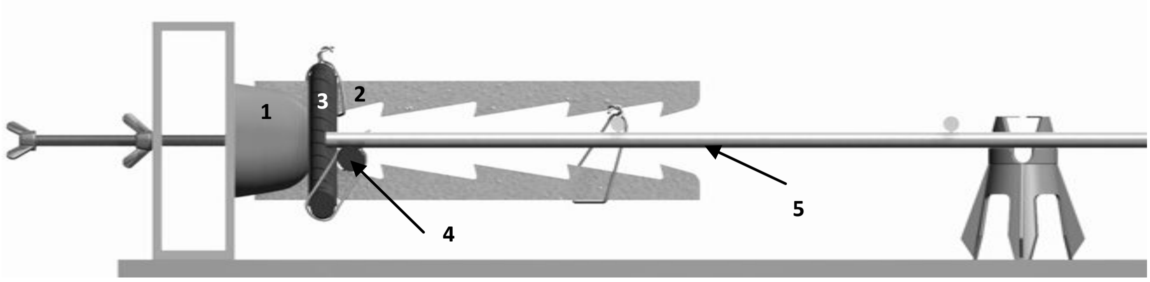

Fig 2: Side view of a typical panel setup for edgelifting

Figure 2 shows a side view of a typical test setup of the plate edgelift anchor (2) placed centrally in a 150mm panel showing the anchor void (1), shear Bar x 90mm height (3), perimeter Bar (4), and panel mesh (5). The edgelift plate anchor, if fitted with no shear bar, will have very little capacity in shear. The concrete breakout strength, in the case of no shear bar fitted, is gained from the embedment of the anchor from the panel near face to the long edge of the anchor, which with a 80mm centrally placed anchor is a 150mm panel is only 35mm cover.

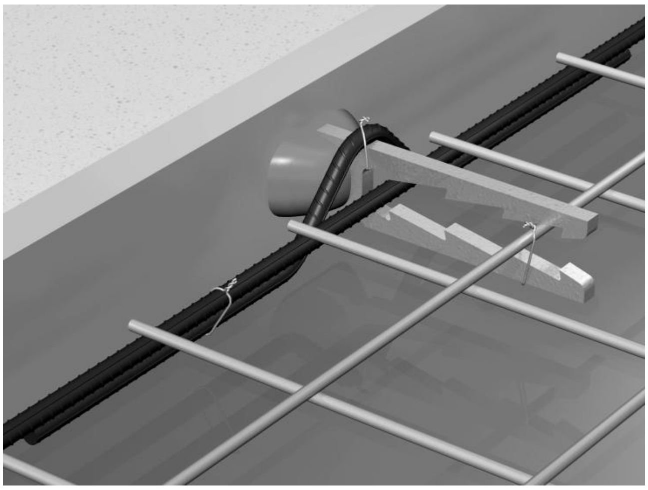

Fig 3: View of a typical anchor setup for thin panel edgelifting

Figure 3 highlights the orientation of the shear bar and the positions where it is tied to the panel reinforcement.

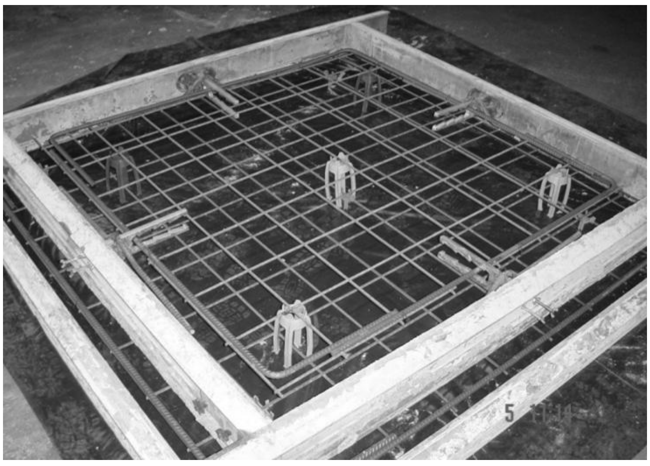

Fig 4: Typical shear panel setup prior to concrete pour

Figure 4 shows a panel setup with 4 x plate edgelift anchors, shear bars placed over the anchors, perimeter bars and SL82 panel mesh. Various configurations of shear bar reinforcement were tested in conjunction with the edge lift anchors; which included panel mesh and perimeter bar. The configurations were chosen on the basis of common standard industry practice. N Class reinforcement steel, 16 mm and 10mm thick edgelift plate anchors were tested by pull out tests in a lateral tensile direction on anchors placed in 100mm, 125mm, 150mm and 175mm thick panels measuring 2m x 2m perimeter. The tests were conducted using normal weight Portland cement concrete with a compressive strength at the time of testing of between 8MPa and up to 45MPa, detailed in table 1. The minimum strength recommended for lifting is 15MPa but lower compressive strengths were included as a lower bound may occur in practice.

The tested breakout strengths were compared to the calculated capacities as determined by design provisions provided in ACI 318M-08 Appendix D 5.2 [2]

2. Predictive Strength Equations

The calculation to estimate the concrete breakout strength of an anchor in tension is presented in ACI318M-08 D5.2, and summarised in figure 5.

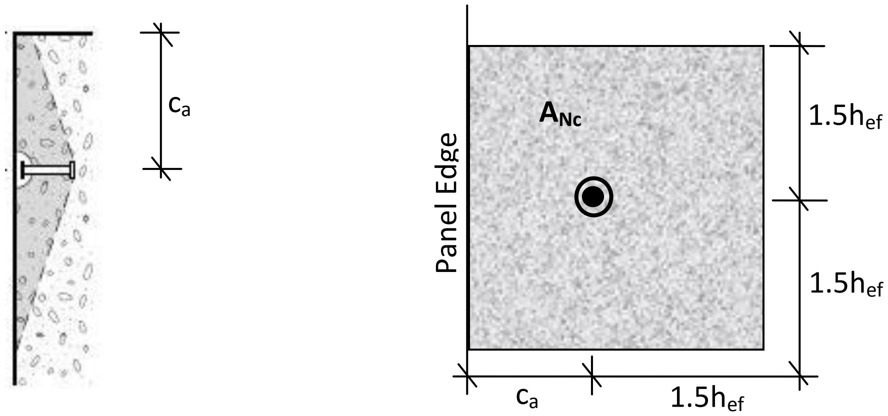

Fig 5: Idealised single edge truncated failure cone

This model is used to estimate nominal concrete breakout strength, N_{cb}, of a single anchor with edge modification factors applied. The assumed brittle fracture surface is as depicted in figure 5. h_{ef} is the effective embedment, where in the case of shear bars, h_{ef} is a function of panel thickness, anchor width and shear bar height, H.

The nominal reduced concrete breakout strength, N_{cb}, of a single anchor loaded in tension is presented in ACI318M-08 D5.2.1, as:

$$N_{cb} = \frac{A_{NC}}{A_{NCO}} . \Psi_{ed,N} . \Psi_{C,N} . \Psi_{cp,N} . N_b \qquad \text{EQ-1}$$

As distance to the edge, c_a , is less than 1.5 h_{ef}, then the edge distance reduced projected concrete failure area, A_{NC}, (see figure 5) is:

$$A_{NC} = \left(c_a + 1.5\, h_{ef}\right).(2\, x\, 1.5\, h_{ef}) \qquad \text{EQ-2}$$

And the unrestricted project concrete failure area, A_{NCO},: (ACI318M-08 D5.2.1)

$$A_{NCO} = 9\, .\, h^2{}_{ef} \qquad \text{EQ-3}$$

Then the tensile edge modification factor, $\Psi_{ed,N}$, to applied is: (ACI318M-08 D5.2.5)

$$\Psi_{ed,N} = 0.7 + 0.3\ \frac{c_a}{1.5\ .h_{ef}} \qquad \text{EQ-4}$$

And the basic concrete breakout strength of a single anchor, N_b , can be calculated as: (ACI318M-08 sec D5.2.2)

$$N_b = \ k_c\ .\ \lambda\ .\ \sqrt{f_c'}\ .\ h_{ef}^{1.5} \qquad \text{EQ-5}$$

Where: λ is the modification factor for lightweight concrete and not applicable for these tests, $\Psi_{C,N}$ (refer 318M-08 D5.2.6) is taken as 1.25 for uncracked concrete, and $\Psi_{ep,N}$ (ACI318M-08 D5.2.7) is taken as a 1, as it is a modification factor for post-installed anchor edge reduction and not applicable to these tests.

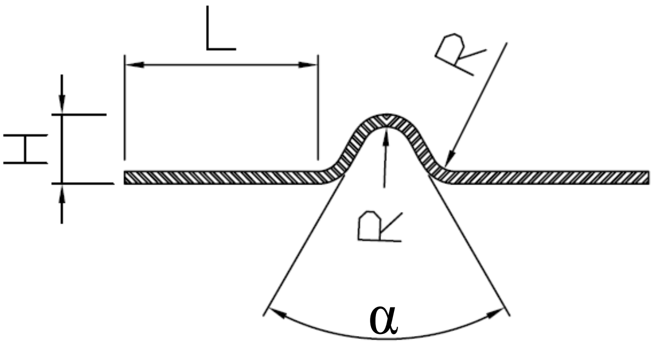

Fig 6: Typical shear bar design

A typical shear bar is defined with variables such as height, H, (which is a function of embedment depth and related to the plate anchor) leg length, L, (which is related to the stress development length) and the bend angle, α, (which is required to clear the void and allow the shear bar to sit in a position that has been designed and tested). The mandrel diameter used to bend the shear bar, if cold bent, is limited to at least 4 x the diameter of the bar, in accordance with AS4671.

When a load is applied to the anchor, the shear bar is subject to bending, especially at the center of the bridge which is in contact with the anchor.

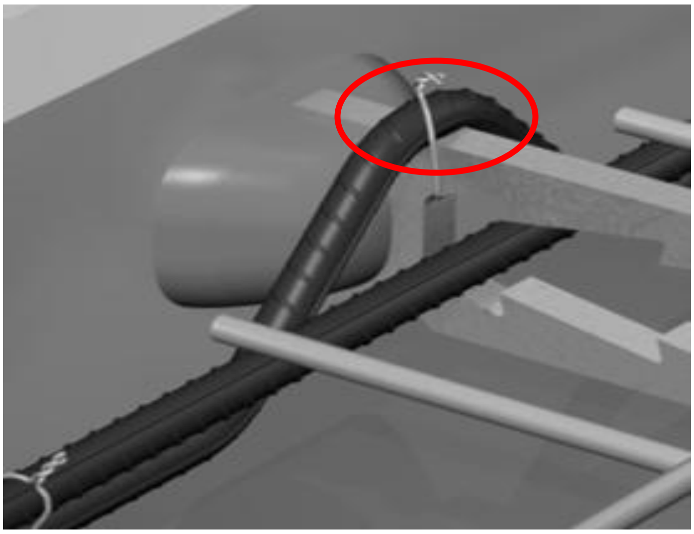

Fig 7: The shear bar bridge is subjected to bending when a shear load is applied

When the shear bar is being installed into the panel formwork it is important for the installer to ensure that the bend radius sits in contact with the anchor, and is adequately tied in. In the case where there is a gap between the anchor and shear bar at this point, and a shear load is applied, the concrete will crack around the head of the anchor, allowing the anchor to move. Special care was taken during the test setup to ensure the shear bar was suitably tied in. This was done by first tying in the shear bar bridge prior to tying in the shear bar legs.

3. Experimental Program

3.1 Test Parameters

Plate-type edge lift anchor pullout lateral tensile tests (137 off) were conducted at concrete compressive strengths and shear bar embedment heights that would precipitate a concrete cone failure. The shear bar anchors were a combination of N16 and N12 reinforcement bars, with either a 90mm or 60mm bend height, refer H in figure 6. Plate anchors used were manufactured from 10mm and 16mm plate, with a profile as shown in figure 9 and figure 10. They were cast in thin (100mm, 125mm, 150mm and 175mm thick) panels all with panel reinforcement using SL82 and a N16 Perimeter bar, described in more detail below.

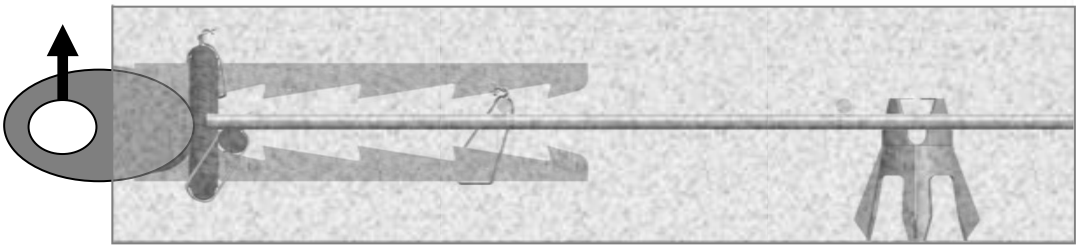

Fig 8: Shear loads applied in a lateral tensile direction

Series 1 test panels were 100mm thick, a plate edgelift anchor of 60mm width x 10mm thick, with a shear bar of N12 x 60mm height. Series 2 test panels were 125mm thick, a plate edgelift anchor of 60mm width x 10mm thick, with a shear bar of N12 x 60mm height. Series 3 test panels were 150mm thick, a plate edgelift anchor of 60mm width x 10mm thick, with a shear bar of N12 x 60mm height. Series 4 test panels were 150mm thick, a plate edgelift anchor of 60mm width x 16mm thick, with a shear bar of N12 x 60mm height. Series 5 test panels were 150mm thick, a plate edgelift anchor of 75mm width x 16mm thick, with a shear bar of N12 x 90mm height. Series 6 test panels were 150mm thick, a plate edgelift anchor of 80mm width x 16mm thick, with a shear bar of N16 x 90mm height. Series 7 test panels were 175mm thick, a plate edgelift anchor of 60mm width x 16mm thick, with a shear bar of N12 x 60mm height. Details are summarised in the table 2 and shown in Figure 3 and figure 4.

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 8MPa to 45MPa, with an average of 22MPa, detailed in table 1. Full concrete compressive data for all series is shown in Table 2.

Table 1 – Concrete Compressive Data for Test Series

Test Series	f_{cm} mimimum (MPa)	f_{cm} maximum (MPa)	f_{cm} average (MPa)
1	19	22	20
2	22	37	29
3	10	32	16
4	15	37	21
5	8	35	22
6	8	45	26
7	30	45	42

Fig 9: Series 7 typical panel setup

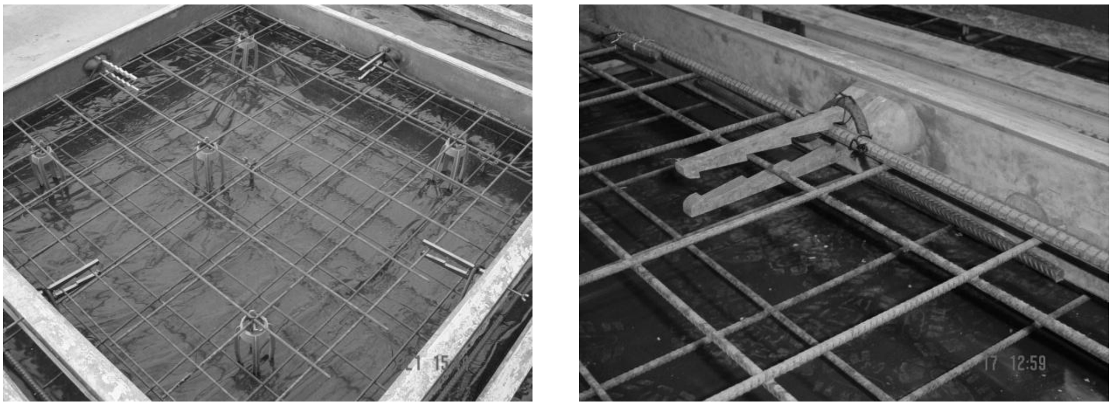

Fig 10: Series 2 typical panel setup

Figure 9 shows a typical panel setup with a 80mm plate anchor, N16 x 90mm shear bar in a 175mm panel. Figure 10 shows a typical panel setup with a 60mm plate anchor, N12 x 60mm shear bar in a 125mm panel.

Each test panel included 4 anchors. Concrete compressive strength, f_c , was recorded by means of cylinder compression tests. Were 4 anchors were setup in a panel for testing, 9 cylinder compressive strengths were recorded for each panel test, 3 at the beginning and end, and 1 after testing anchor 1, 2 and 3. The mean of these cylinder compressive strengths was calculated, f_{cm}, for each panel test, and noted in table 1.

Table 2. Reinforcement Configurations for Test Series

	Panel Thickness, mm	Plate Edgelift Anchor		Shear Bar sizes, N Class		
Test Series		Width, mm	Thickness, mm	N16 x 90mm	N12 x 90mm	N12 x 60mm
1	100	60	10	Nil	Nil	Yes
2	125	60	10	Nil	Nil	Yes
3	150	60	10	Nil	Nil	Yes
4	150	60	16	Nil	Nil	Yes
5	150	75	16	Nil	Yes	Nil
6	150	80	16	Yes	Nil	Nil
7	175	60	16	Nil	Nil	Yes

Note: Each test panel included SL82 panel mesh and N16 perimeter bars.

The anchors were loaded under a load-controlled 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) and load-time. The panels were tested horizontally and supported off the floor on timber gluts whilst the panel reacted against a steel frame with an open span of 2.0m 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 1.0m, shown in Figure 11.

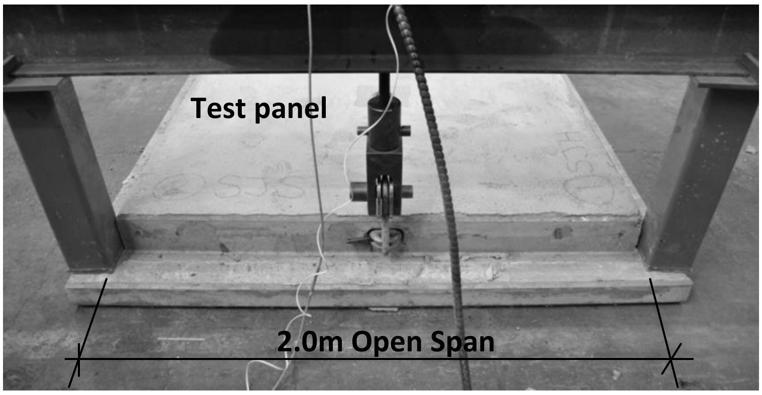

Fig 11: Shear loads applied with 2.0m open span reaction frame

3.2 Test results and Analysis

Shear reinforced plate-type edge lift anchor pullout tests (137 off) were conducted at concrete compressive strengths and shear bar embedment heights that would precipitate a concrete cone failure. The shear bar used on these edge lift anchors were N Class, 60mm and 90mm height, 16mm and 12mm diameter reinforcement installed with 60mm, 75mm and 80mm wide plate anchors, (10mm and 16mm plate thicknesses). They were cast in thin (100mm, 125mm, 150mm and 175mm thickness) panels with the reinforcement configurations described in more detail below. Details of the combination are summarised in Table 2. 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 8MPa to 45MPa, with an average of 22MPa.

Fig 12: Typical failure surface propagating to the panel thin edge

The edge lift anchor test data was compared with the predicted capacity as determined using the ACI 318M-08 concrete breakout strength calculations detailed in section D 5.2.1 including reduction factors. The ratio of the recorded breakout loads and the calculated strengths, as per ACI 318M-08 D 5.2, is shown in Table 2.

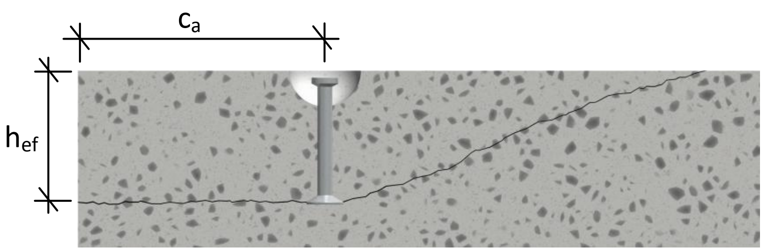

Fig 13: Idealised fracture path as stated in ACI318M-08 for a single cast-in anchor

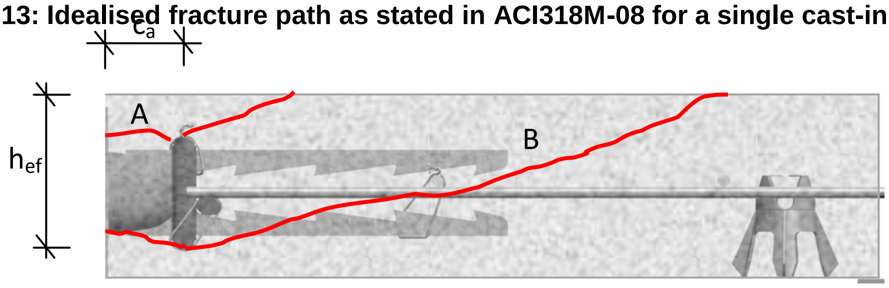

Fig 14: Estimate fracture path from test results

If crack in position A starts to propagate and reach the panel surface prior to crack B being visible on the surface, this can be an indication that the bridge of the shear bar is not seated adequately to the anchor. A panel flexure crack will also be present if crack A erupts to the surface prior to crack B.

Once crack B continues to open, the load on the shear bar bridge will increase, and bending of the shear bar may occur.

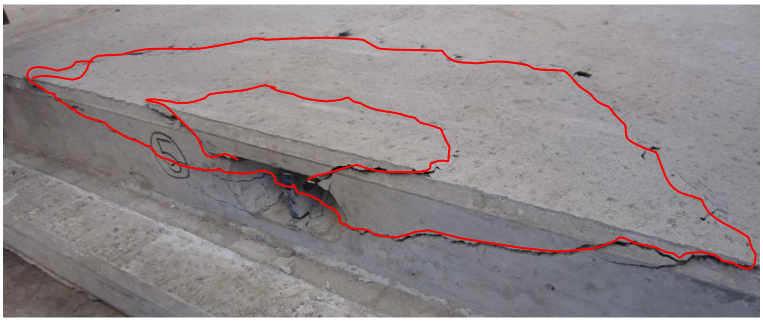

Fig 15: Actual fracture surface

Figure 15 shows typical fracture cones propagated to the panel surface. Using equations 1-5, with the variables in figure 14, the predicated values were calculated and compared with the test results and presented in table 3.

Table 3 Tensile breakout strength compared with the reduced concrete breakout capacities, N_{cb}, presented by ACI 318M-08 D5.2 [3]

Series	Number of Tests, n	Test / Predicted Range	Average Test/Predicted	Variation Coefficient, %
Series 1: 100mm Panel, 60x10mm Anchor, N12x60mm Shear bar	8	1.29 – 1.80	1.59	8
Series 2: 125mm Panel, 60x10mm Anchor, N12x60mm Shear bar	10	1.15 – 2.82	1.60	14
Series 3: 150mm Panel, 60x10mm Anchor, N12x60mm Shear bar	5	1.08 – 3.40	2.02	16
Series 4: 150mm Panel, 60x16mm Anchor, N12x60mm Shear bar	32	1.04 – 2.64	1.75	20
Series 5: 150mm Panel, 75x16mm Anchor, N12x90mm Shear bar	25	1.47 – 3.41	2.53	17
Series 6: 150mm Panel, 60x16mm Anchor, N16x90mm Shear bar	42	1.08 – 2.70	1.67	23
Series 7: 175mm Panel, 60x16mm Anchor, N12x60mm	4	1.01 – 1.42	1.14	7

Shear bar				

The results of these tests all fell above the predicted concrete breakout strengths calculated, demonstrating that the ACI model is adequate for shear capacity concrete strength predictions. The coefficient of variation for series 2, 3, 4, 5 and 6 is statistically widespread, and since these tests were conducted under controlled conditions, a typical precast manufacturing environment should ensure suitable quality control procedures to monitor the adherence to installation recommendations from anchor manufacturers. Care should be taken to ensure the design intent of the shear bar is maintained.

4. Concluding remarks

This paper is an evaluation of concrete breakout capacities for shear reinforcement of edge lift anchors in thin walled elements. Using the formula in the ACI 318-08M D5.2 [3], comparisons of the tested breakout strength and the shear test pull out capacity of the edge lift anchors is made. All panels were reinforced with a shear bar, centrally placed SL 82 mesh and centrally placed N16 perimeter bar, and the ratios of test to predicted failure load indicate that the ultimate capacity of all the shear reinforced anchors was within the predicted breakout strengths noted by ACI 318-08M.

The ratio of actual against predicted does however get close to 1, showing that under less controlled conditions environments the designed performance may be greater than actual performance if suitable quality procedures are not used to monitor shear bar design and installation.

Overall, 137 tests were conducted using shear reinforced edge lift anchors in lateral tension; the variables tested include concrete compressive strength at time of testing which ranged from 8 to 45MPa with an average of 22MPa, and arrangement of shear reinforcement which included the provision of perimeter bars 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 model 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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