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Embed plates with anchor reinforcement under eccentric shear loading

Deepak Suthar[1], Akanshu Sharma[2], Rolf Eligehausen[3], Adeola Adediran[4]

[1] PhD Candidate, Lyles School of Civil and Construction Engineering, Purdue University, West Lafayette, IN, USA (dsuthar@purdue.edu)
[2] Jack and Kay Hockema Associate Professor in Civil and Construction Engineering, Lyles School of Civil Engineering, Purdue University, West Lafayette, IN, USA (akanshu@purdue.edu)
[3] Professor Emeritus, University of Stuttgart, Stuttgart, Germany (eligehausen@gmx.de)
[4] Senior Principal Engineer, Bechtel National Inc., USA (akadedir@bechtel.com)

ABSTRACT

Embed plates (EP) in nuclear power plant construction are often used to resist different types of loading conditions like shear, tension, combined tension, shear loading, etc. Under shear loading, embed plates close to an edge tend to fail in a concrete edge breakout which is a brittle failure mode as it is primarily governed by distance of the embed plate from the edge. Anchor reinforcement is provided around the embed plates subjected to shear loading towards the edge to improve the ultimate capacity and ductility of embed plates. The design of anchor reinforcement for nuclear structures is governed by ACI 349-23. As per ACI 349-23, the anchor reinforcement is required to be developed in both the concrete edge breakout body and the concrete body. These requirements are difficult to satisfy for anchor reinforcement due to the limited edge distance of embed plates failing in concrete edge breakout. When eccentric shear loading is applied, anchor reinforcement is required to be designed for the highest-loaded anchor with the same amount of anchor reinforcement provided for all the anchors as the highest-loaded anchor. This leads to a very high amount of reinforcement requirements. The primary reason for this overdesign is the lack of any experiments. In this work, embed plates with 2×3 configuration were tested under eccentric shear loading with different amounts of anchor reinforcement. The reference test without anchor reinforcement was designed to fail in concrete edge breakout. Anchor reinforcement of 10 mm (#3), 12 mm (#4), and 16 mm (#5) diameters were used in the form of hoops enclosing edge reinforcement. For a light amount of anchor reinforcement (#3 rebar), a 32% increase was observed in the capacity of the embed plates compared to the reference. For a medium amount of reinforcement (#4 rebar), approximately a 66% increase in the capacity was observed. For a high amount of anchor reinforcement (#5 rebar), a relative increase of 74% was observed. The test results were compared with the predicted values from the ACI 349-23 model and it was observed that a very conservative model is utilized which leads to over-design and underutilization of embed plate capacity.

INTRODUCTION

Embed plates undergo shear loading in various conditions in a nuclear power plant when resisting vertical loads or lateral loads under operational conditions or design conditions. These embed plates are critical in ensuring a safe and reliable connection and transfer of forces from the superstructure to the concrete body. Failure of these embed plates can critically impair the operational capabilities of a nuclear power plant or, in a severe case, cause failure of critical components.

Under shear loading, especially close to an edge, embed plates can fail in concrete edge breakout which is a brittle failure and leads to a sudden detachment of the component from the concrete body without giving any prior warnings (*Eligehausen et al., 2006*). Therefore, it is important to avoid concrete edge breakout and also utilize the available capacities of the embed plate in a safe and reliable manner.

Anchor reinforcement is provided around embed plates close to an edge to improve its load capacity and prevent a brittle concrete failure (*Ramm and Greiner, 1991; Peternsen and Zhao, 2013; Schmid, 2010; Sharma et al, 2017a*). Figure 1 shows the behavior of a typical embed plate with anchor reinforcement. When shear loading is applied, concrete initially resists all the applied load until concrete edge breakout capacity is reached. This results in the formation of breakout cracks which intercept the anchor reinforcement leading to a redistribution of forces between concrete and anchor reinforcement. At peak load, the full capacity of reinforcement and the partial capacity of concrete resist the load (*Sharma et al., 2017b*). ACI 349-23 governs the design of anchorages in nuclear power plants in the United States. The currently acceptable model for the design of embed plates with anchor reinforcement requires the development length of anchor reinforcement in the breakout body and in the concrete body. This requires a long rebar length in the breakout body which is rarely available in most practical cases. In addition to that, the final capacity is higher of the concrete edge breakout capacity or the yield capacity of anchor reinforcement which are developed in both the breakout body and the concrete body.

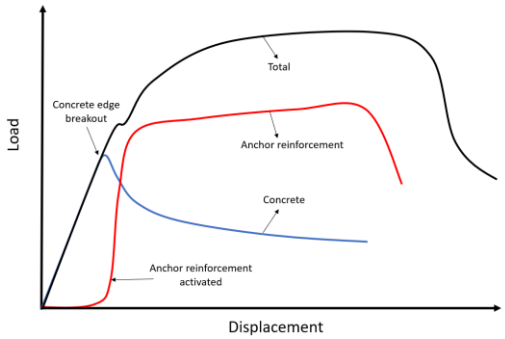


Figure 1. Typical behavior of anchorage with anchor reinforcement under shear loading.

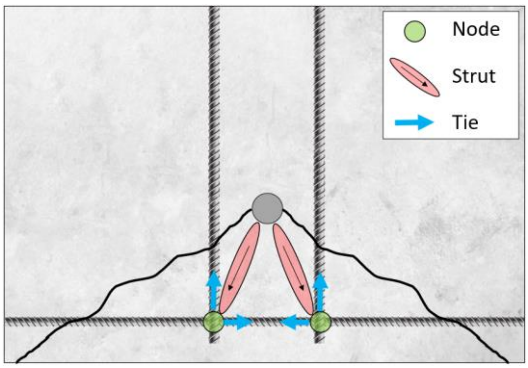


Figure 2. Strut-and-tie model for anchors with anchor reinforcement under shear loading.

Figure 2 shows a strut-and-tie model for anchorages failing in concrete edge breakout with anchor reinforcement. After the formation of the concrete edge breakout, the struts develop between the anchor and the anchor reinforcement hook which encloses the edge reinforcement. The hook acts as a CTT node and two ties develop with anchor reinforcement and edge reinforcement. The failure of the strut-and-tie model can occur in multiple modes. Failure of tie can occur due to yielding of the anchor reinforcement. Failure of the node can occur through the anchorage failure of the anchor reinforcement either in the breakout body or in the concrete body if enough reinforcement length is not available. Strut failure can also govern the overall capacity when a high amount of anchor reinforcement area is provided (*Berger 2015*). Sturt failure is not covered widely by design codes currently (*fib Bulletin 58, EN 1992-4*). ACI 349-23 and ACI 318-19 do not recognize this failure.

When eccentric shear loading is applied on the anchorages with anchor reinforcement, current design codes do not allow to utilize the full capacity of the system and require the design of anchorage with the same load on all the anchors as the highest loaded anchor. In addition to that, it also requires providing the same amount of anchor reinforcement for each anchor which results in overdesign. These conservative regulations are kept in design codes due to the lack of any available dataset for anchorages with anchor reinforcement under eccentric shear loading. This work aims to perform full-scale tests on embed plates with anchor reinforcement of different sizes under eccentric shear loading and provide data points for the development of an improved model for eccentric loading.

TEST PROGRAM

Table 1 shows the test program that was developed to investigate the behavior and check the performance of cast-in embed plates with different amounts of anchor reinforcement under eccentric shear loading. The scope of the tests was limited to a single configuration of embed plates with a 2×3 configuration, 3 anchors in the front row and 3 anchors in the back row. The anchor reinforcement was provided in the form of closed hoops enclosing the edge reinforcement. Even though hoops were used, anchor reinforcement on the same side as the embed plate will contribute and the reinforcement on the other side does not contribute towards the resistance. Figure 3a shows the typical layout of reinforcement in the specimen before concrete casting. Two embed plates were tested, one on each long side of the slab, with two anchor reinforcement hoops provided for each line. The center-to-center distance between the anchor line and stirrup was kept at 50 mm (2 in.).

The surface reinforcement was also provided in the form of hoops as shown in Figure 3a. It was ensured that no surface reinforcement, in the direction of the shear loading, was provided in the projected shear edge breakout region for the reference case as shown in Figure 3b. For cases with anchor reinforcement, only anchor reinforcement hoops get activated without activating any surface reinforcement which could have potentially contributed towards the total resistance. For both tests, anchor reinforcement bars were not connected and were discontinued in the middle so that the effect of one test on another could be avoided as shown in Figure 3c. All the reinforcement bars used in this study were Grade 60 rebars (Yield Strength of 420 MPa). To calculate the stress state of anchor reinforcement, strain gauges were used at the location of crack interception calculated using a theoretical concrete edge breakout shape.

The embed plate consists of a 50 mm (2 in.) thick baseplate with (6) 25 mm (1 in.) diameter studs of length 125 mm (5 in.) welded in a 2×3 configuration. ASTM A108 low carbon steel studs were used which were fillet welded to the ASTM A36 carbon steel baseplate. The slab specimens used for testing were 1800 mm×1550 mm (72 in.×62 in.) in size and 550 mm (22 in.) in thickness. The concrete pouring of the specimens was performed upside down, as shown in Figure 3a, to achieve good bond conditions between the anchor reinforcement and concrete. Later, the specimens were flipped using lifting hoops. The average concrete cylinder strength measured after 28 days was 40.1 MPa (5817 psi).

Table 1: Summary of the test program.

Series	h_{ef}	c_1	s_1=s_2	d_a	e_x	**Anchor Reinf.**	**Rebar area**
1	175 mm (7 in.)	125 mm (5 in.)	150 mm (6 in.)	25 mm (1 in.)	75 mm (3 in.)	-	-
2						6-10 mm (6-#3)	426 mm^2 (0.66 in^2)
3						6-12 mm (6-#4)	774 mm^2 (1.2 in^2)
4						6-16 mm (6-#5)	1200 mm^2 (1.86 in^2)

Where, h_{ef} is the effective embedment depth of the studs; c_1 is the edge distance of the front anchor row; s_1, s_2 are the spacing between the studs in each direction; d_a is the diameter of the studs; e_x is the effective eccentricity provided to the embed plates.

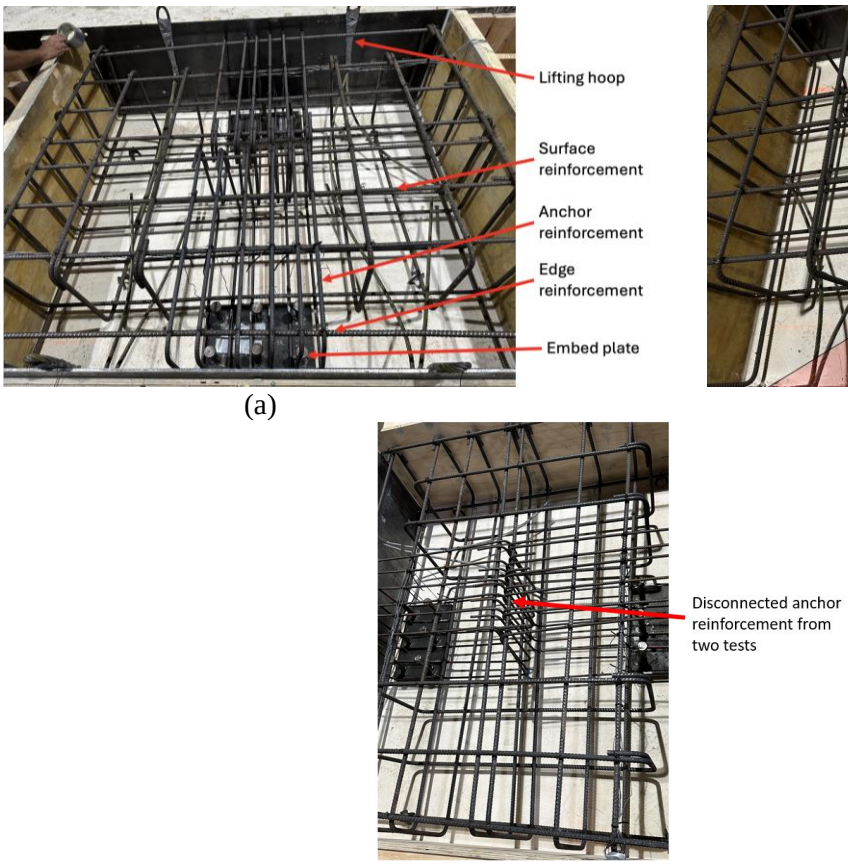
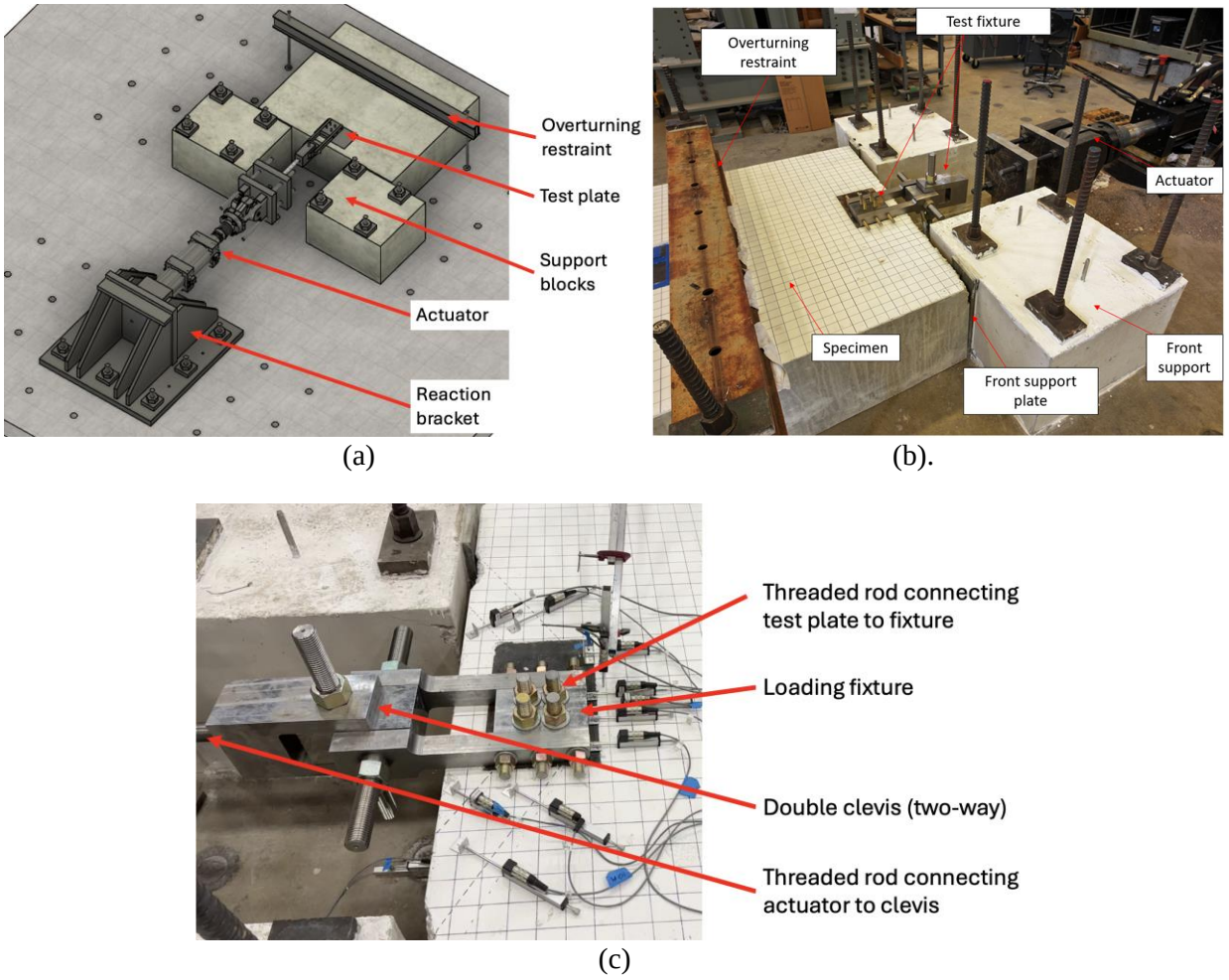


Figure 3. (a). Arrangement of the anchor reinforcement around embed plate before concrete casting, (b). Projected concrete edge breakout location with no anchor reinforcement for reference tests, (c). Discontinued anchor reinforcement hoops between two tests.

Test Setup

All tests performed in this study were conducted in Bowen Laboratory, Purdue University. The tests were monotonic static shear in a displacement control protocol at a constant rate of 1 mm/min (0.04 in./min). Before testing, a white matte paint was applied on the top surface of the test slab with a 50 mm×50 mm grid pattern for improved visualization of cracks and post-test evaluation. Figure 4 shows the overview of the test setup which includes a steel bracket with an actuator of 1000 kN (220 kips) capacity and 250 mm (10 in.) stroke length attached horizontally. The steel bracket is post-tensioned to the strong floor. The height of the line-of-action of the actuator from the ground was equal to the slab thickness and half of the test fixture thickness. The actuator is connected to the anchor pullout fixture through a 44 mm (1-¾ in.) high-strength threaded rod. The pullout fixture consists of a two-way custom clevis capable of providing hinging action in out-of-plane and in-plane rotation of the embed plate during tests as shown in Figure 4c. The clevis is further connected to the test plate using two arms which are further connected to the loading plate. The loading plate is connected to the embed plate using four 32 mm (1-¼ in.) threaded rods.

To hold the specimen and prevent any overturning, a heavy steel section was provided at the back as shown in Figure 4b. The steel section was connected to the strong floor using two DYWIDAG bars. It should be noted that the DYWIDAG bars were post-tensioned to the strong floor, but the steel section was not post-tensioned to the strong floor and was connected through the protruding length of the bars. The steel section was wrench-tightened to the specimen through DYWIDAG bars. To provide the eccentricity, the specimen was moved to the side with respect to the line of action of the actuator. Two post-tensioned concrete blocks were provided in the front of the slab specimen to provide horizontal support. Two 150 mm (6 in.) wide steel plates were provided in the front between the support blocks and specimen to provide rigid front support. A clear distance of more than twice the back anchor row distance was provided on both sides of the test plate to avoid any support interference.



(a) (b).

(c)

Figure 4. Test setup used in this work (a). Schematic view, (b). Actual view in lab, (c). Testing fixture used to load the embed plate.

RESULTS AND DISCUSSION

Table 2 summarize the test results. Figure 5 shows the load-displacement curves for all the tests performed. A significant increase in the ultimate load and ductility was observed for all the tests with anchor reinforcement compared to the reference test without any anchor reinforcement.

Table 2: Summary of the peak load and failure mode obtained for each test series

Series	Anchor Reinforcement	Peak load	Average peak load	Relative increase	Governing Failure mode
1	-	267.7 kN (60.15 kips) 264.2 kN (59.37 kips)	265.9 kN (59.76 kips)	1.00	CEB
2	6-10 mm (6-#3)	347.8 kN (78.17 kips) 356.6 kN (80.13 kips)	352.2 kN (79.15 kips)	1.32	CEB-RY-SB
3	6-12 mm (6-#4)	402.5 kN (90.45 kips) 480.9 kN (108.08 kips)	441.7 kN (99.26 kips)	1.66	CEB-RY-SB
4	6-16 mm (6-#5)	454.5 kN (102.13 kips) 473.5 kN (106.41 kips)	464.0 kN (104.27 kips)	1.74	CEB-RY-SB

Where, CEB: Concrete edge breakout; RY: Majority rebar yielding; SB: Secondary breakout.

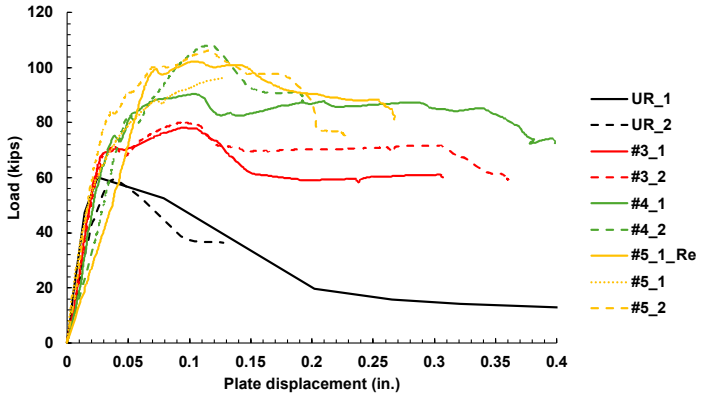


Figure 5. Peak load (kips) vs plate displacement (in.) for all the tests performed.

Series 1

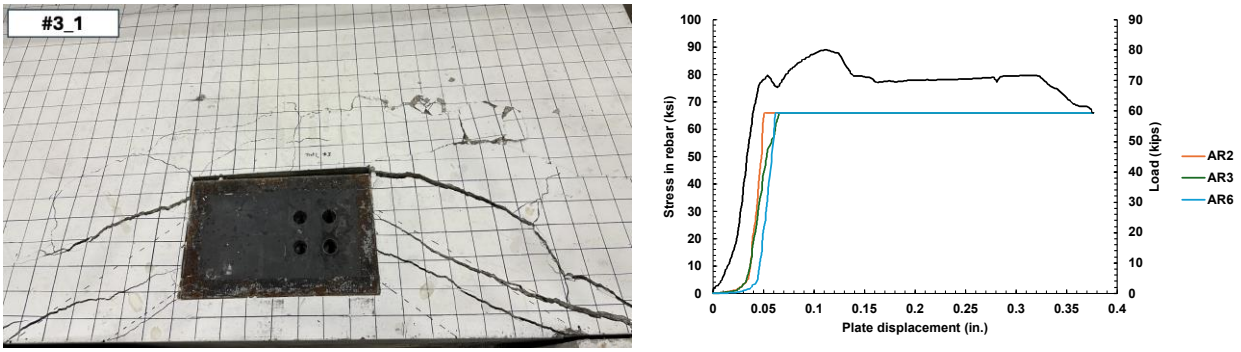
For unreinforced cases, an average of 265.9 kN (59.76 kips) load was observed with an asymmetric concrete edge breakout as shown in Figure 6a. It can be noted that the failure mode was brittle indicated by a drop in the load after peak load was reached as shown in Figure 5. Initially, the concrete edge breakout from the front row was observed on the eccentric side. This is followed by a failure crack from the back anchor row with an asymmetric failure shape. After completely removing the breakout body, as shown in Figure 6b, it can be verified that the governing failure crack occurred from the back anchor row. The predicted value from ACI 318-18/ACI 349-23 was 269.2 kN (60.5 kips) which is very close to the experimental values.

Series 2

Concrete edge breakout from the front anchor was observed initially with small cracks followed by the initiation of failure cracks from the back anchor row as shown in Figure 7a. During that phase, the redistribution of force occurs between the concrete and anchor reinforcement. Forces in the anchor reinforcement increased until yielding was observed as shown in Figure 7b. It should be noted that the number of anchor reinforcement hoops (AR#) is based on the eccentricity side. AR1 starts from the eccentricity side and ends with AR6 on the opposite side. It was observed that anchor reinforcement closest to the highest loaded anchors picks the load initially followed by redistribution of forces among anchor reinforcement after yielding as evident from Figure 7b. An average of 352.2 kN (79.15 kips) load was observed which resulted in a relative increase of 32% compared to unreinforced cases. Eventually, after yielding all the anchor reinforcement, a secondary breakout (similar to Pryout failure) was observed on the back side of the back anchor row as shown in Figure 7a.

(a) (b)

Figure 6. Asymmetric concrete edge breakout in case of eccentric shear load without anchor reinforcement (series 1).



(a). (b).

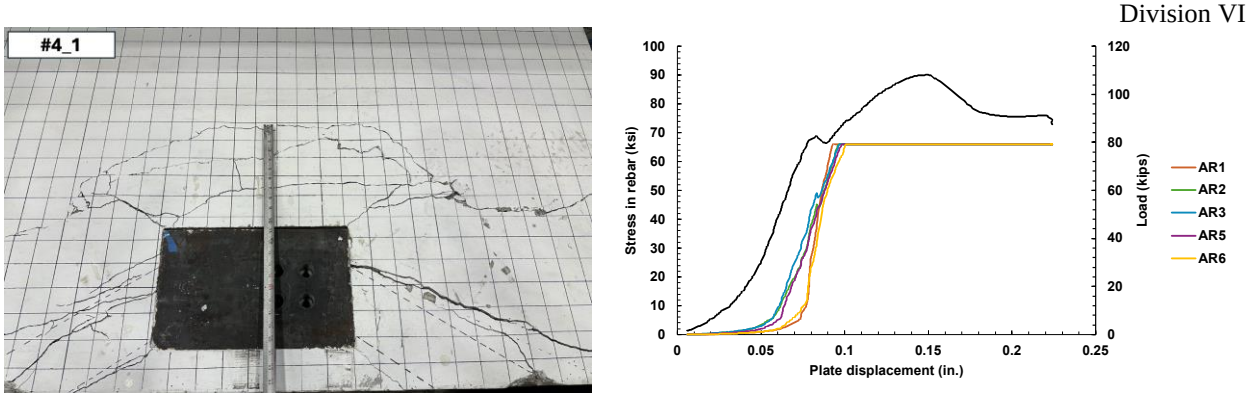
Figure 7. (a). Failure mode of series 2 tests, (b). State-of-stress as a function of plate displacement.

Series 3

For series 3, observations similar to series 2 were made with an average of 441.7 kN (99.26 kips) load as shown in Figure 8a. A relative increase of 66% was achieved compared to the unreinforced case. It can be observed in Figure 8b that the anchor reinforcement closest to the eccentricity side starts to take load first compared to the anchor reinforcement away from the loading side. This results in a redistribution among the anchor reinforcement. Eventually, all the anchor reinforcement yields followed by a pryout/secondary breakout from the back anchor row. It can be verified that the pryout failure from the back anchor row extends approximately $1.5h_{ef}$ (262.5 mm) which is in line with the currently followed norms. Pryout failure can be avoided using higher embedment depth of the studs to increase the pryout capacity from the back anchor row.

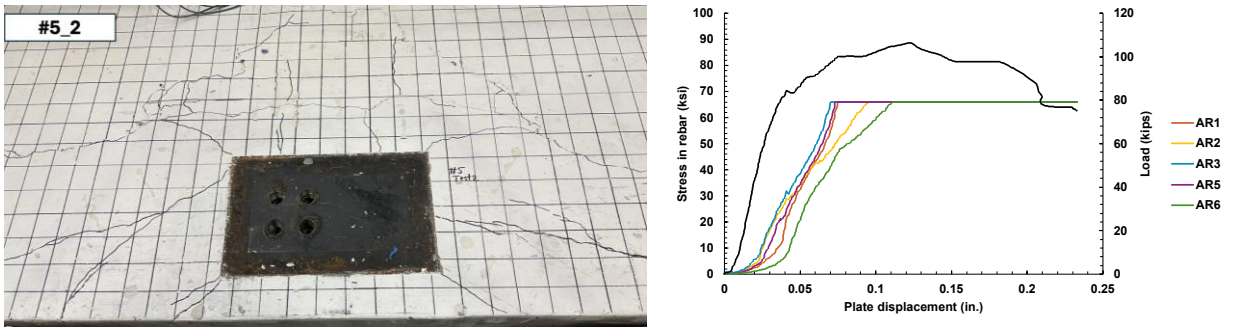
Series 4

For series 4, an average of 464 kN (104.27 kips) load was observed with similar failure observations as series 2 and series 3 which gives a relative increase of 74% compared to the unreinforced case and is shown in Figure 9a. However, a stronger redistribution among anchor reinforcement was observed as shown in Figure 9b due to larger sizes of anchor reinforcement. The pryout failure was observed eventually after the yielding of the anchor reinforcement. This is primarily due to no uplift restraint of the concrete on the back side of the embed plate. When a certain displacement level is achieved after concrete edge breakout, the breakout starts to rotate out of the plane which results in a pryout failure from the back anchor row. When compared with the calculations from fib Bulletin 58, a load of 489.5 kN (110 kips) was calculated which matches well with the experimental observations.



(a). (b).

Figure 8. (a). Failure mode of series 3 tests, (b). State-of-stress as a function of plate displacement.



(a). (b).

Figure 9. (a). Failure mode of series 4 tests, (b). State-of-stress as a function of plate displacement.

Comparison with ACI 349-23

Table 3 shows the comparison between experimentally observed peak load and peak load calculated using the ACI 349 model at the mean level. The hook development lengths were calculated for #3, #4, and #5 rebar for the given conditions and it was observed that all anchor reinforcement bars in series 2 (#3) were satisfying the requirements. Only 4 bars were satisfying the requirements for series 3 and no rebars were satisfying for series 4. Based on that, the contribution of reinforcement was calculated and it was smaller than concrete edge breakout capacity for each case. Per ACI 349-23, no contribution of anchor reinforcement toward the total resistance should be assumed. Therefore, concrete edge breakout capacity governs the design for all the cases which leads to a significant underutilization of anchorage capacity. This calls for improvements in the current model that is available to calculate the capacity of anchorages with anchor reinforcement under eccentric shear loading.

Table 3. Comparison of the average experimental load with the predicted values from ACI 349-23.

Series	Anchor reinforcement	Exp. load	Exp. failure mode	ACI 349 load	ACI 349 failure mode	Test/ACI 349
1	-	265.9 kN (59.76 kips)	CEB	269.2 kN (60.5 kips)	CEB	0.99
2	6-10 mm (6-#3)	352.2 kN (79.15 kips)	CEB-RY-SB		CEB	1.31
3	6-12 mm (6-#4)	441.7 kN (99.26 kips)	CEB-RY-SB		CEB	1.64
4	6-16 mm 6-#5	464.0 kN (104.27 kips)	CEB- RY-SB		CEB	1.72

CONCLUSION

Embed plates are used in nuclear power plants to connect various structural and non-structural components like reactor vessels, etc. to the concrete structure. Eccentric shear loading is common on embed plates under such conditions which creates asymmetric loading conditions on the embed plates. When present close to an edge, concrete edge breakout can occur as a result which is a brittle failure. Anchor reinforcement is used in such conditions to improve the strength and ductility of the embed plate. Current design guidelines in estimating the capacity of anchorages with anchor reinforcement under eccentric shear loading are very conservative. No experimental results are available under such conditions which leads to conservative design models. A test program to investigate the performance of embed plates with anchor reinforcement was developed for 2×3 embed plates subjected to eccentric shear loading with varying amounts of anchor reinforcement provided in the form of close hoops.

For reference tests, a concrete edge breakout was observed as expected with an asymmetric breakout shape. For a low amount of anchor reinforcement area with 10 mm (#3) rebars, a relative increase of 1.34 times reference tests was observed with improved ductility. The governing failure changed from concrete cone breakout to yielding of rebars. For a medium amount of reinforcement with 12 mm (#4) rebars, a relative increase of 1.64 times the reference test was observed with a similar failure pattern as series 2. For a high amount of anchor reinforcement with 16 mm (#5) rebars, the same observations as series 2 and series 3 were made with a relative increase of 1.74 times the reference value. For all cases with anchor reinforcement, a secondary breakout was observed after the yielding of the anchor reinforcement.

A direct comparison was made, at the mean level, with ACI 349 predicted capacities. In all cases, concrete edge breakout strength was governing the overall capacity when calculated using ACI 349. This demonstrates an underutilization of the anchorage capacity of embed plates when used with anchor reinforcement and a call for improvement in the current design guidelines. Based on the tests performed, a new improved model will be developed and published in an upcoming publication.

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