Behavior of Large Anchor in Tension in Unreinforced/Reinforced Concrete

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1. Introduction

Major heavy equipment in the nuclear power plant such as reactor vessel and steam generator are installed utilizing large size anchor bolts which are greater than 2 in. in diameter and 25 in. in embedment depth into concrete. While the tensile behavior of smaller anchors has been studied extensively the world over, large anchors have not been adequately addressed. Tests of large anchors in tension were performed to develop design criteria and to evaluate the effectiveness of capacity prediction method developed for small anchors.

According to the test results, the concrete capacities can be predicted by the ACI 349-01, and they can be increased by the supplementary reinforcements designed by a strut-tie model.

2. Description of Experimental Program

2.1 Test Setup

Configuration of the test setup composed of loading frame, loading plate, jack assembly, load cell and other instrumentation facilities is shown in Figure 1.

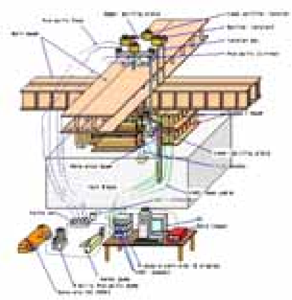

Figure 1. Schematic layout of tension test setup

2.2 Test Specimens

To evaluate the effects of anchor diameters and embedment depths, five types of anchor specimens were tested, ranging from 2.75 to 4.25in. in diameter and 25 to 45 in. in embedment depth. The anchor bolts conforming to ASME SA540 Gr. B23 with F_y =140ksi and F_u =155 ksi which have been used in the Korean nuclear power plant are used for the test.

Specimens No. T1, T2 and T3 have different anchor bolt diameters and embedment depth without reinforcing steel in the concrete. Specimens No. T4 and T5 have the same anchor details as T1 and different amount of reinforcing steel is provided to account for the effects of reinforcement on anchor behavior. To avoid propagation of concrete failure to the adjacent anchor resulting from the anchor being tested, support beam of the loading frame is located in between the neighboring anchors of each specimen. The distance between the anchor and inner face of the support beam is maintained farther than two times the embedment depth. Wooden and steel frames were constructed to suspend the cast-in-place anchors in the correct position, as shown in Figure 2.

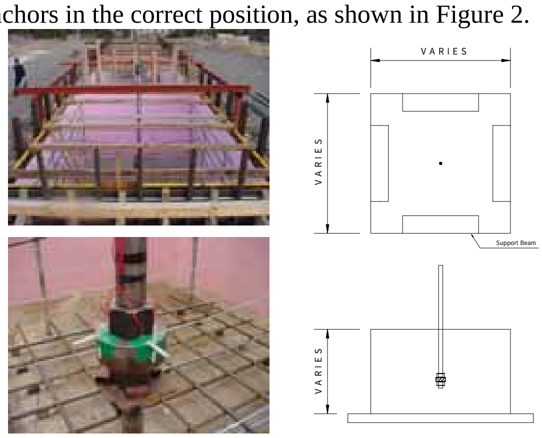

Figure 2. Schematic sketch and photo of tension test

2.3 Loading Procedure

The test load is increased progressively in cycles to a monotonically increasing maximum test load which is limited equal to 80% of the specified tensile strength of anchor bolt.

3. Test Results

3.1 Specimen Behavior

The loads corresponding to the specified tensile strength of anchors ASTM A540 of 155 ksi are 925.0, 1683.6.1 and 2192.4 kips for anchors 2.75, 3.75, and 4.25 in. in diameter, respectively. The tensile breakout capacity of unreinforced test specimens T1, T2, and T3 having 25-in., 35-in. and 45-in. embedments and reinforced test specimens T4 and T5 having 25 in are 513, 727, 1223, 759 and 766 kips, respectively not exceeding the ultimate tensile capacities of anchor bolt ASTM A540.

The general crack patterns comprised of one major longitudinal crack centered approximately on the sides of the block with a horizontal crack and some transverse cracks. On the top surface, the cracks formed a circular pattern around the anchor bolt. To physically identify the internal crack propagation defining the breakout failure cone, one sample of each specimen type was selected, and the concrete was cored on two orthogonal planes whose intersection coincided with the axis of the anchor. The cores defined a breakout cone whose angle with the concrete surface varied from $\alpha=20°$ to $\alpha=30°$.

3.2 Predicted and Tested Tensile Breakout Capacities

The test results are compared with a predicted tensile capacity. The predicted tensile capacities are calculated based on the CC-method with $h_{ef}^{1.5}$ and $h_{ef}^{1.67}$, ACI 349-97, and ACI 349-01 which are provided in Eq.(1), Eq(2), Eq(3) and Eq(4).

$$N_n = 40\sqrt{f'_c}h_{ef}^{1.5} \quad \text{(CC-Method with } h_{ef}^{1.5}\text{)} \quad \text{Eq.(1)}$$

$$N_n = 26.7\sqrt{f'_c}h_{ef}^{1.67} \quad \text{(CC-Method with } h_{ef}^{1.67}\text{)} \quad \text{Eq. (2)}$$

$$N_n = 4\sqrt{f'_c}\pi h_{ef}^2(1+d_h/h_{ef}) \quad \text{(ACI 349-97)} \quad \text{Eq. (3)}$$

$$N_n = 20\sqrt{f'_c}h_{ef}^{5/3} \quad \text{(ACI 349-01)} \quad \text{Eq. (4)}$$

The test results are converted to mean values and 5% fractile values. The tensile capacities by CC-method are compared with mean values while the tensile capacities by ACI 349 are compared with 5% fractile values. Comparisons of the test results to predicted capacities by formulas are shown in Table 1.

Table 1. Tension Test Results and Prediction for Unreinforced Specimens

		Ratio of Observed to Predictions			
		25 in (T1)	35 in (T2)	45in (T3)	Mean
5% Fractile of Test Results	F5%/ACI349-97	0.58	0.51	0.44	0.51
	F5%/ACI349-01	1.24	1.19	1.12	1.18
Mean of Test Results	Mean/ CC-Method w/$h_{ef}^{1.5}$	1.37	1.21	1.39	1.32
	Mean/ CC-Method w/$h_{ef}^{1.67}$	1.19	0.99	1.09	1.09

4. Summary

4.1 Large Anchors without Supplementary Reinforcement

Table 1 shows that ACI 349-97 significantly overestimates the tensile breakout capacity of large anchors. The ratio $N_{u,test}/N_{u,pred.}$ decreases with increasing embedment depth. Furthermore the slope of the breakout cone was much flatter than 45 degrees in ACI 349-97. Anchors at edge or group anchors would also be overestimated.

The CC-method with $h_{ef}^{1.5}$ is considered conservative for large anchors. The CC-method with $h_{ef}^{1.67}$ and ACI 349-01 predict the test result rather well.

4.2 Large Anchors with Supplementary Reinforcement

The supplementary reinforcement provided in T4 was not strong enough to resist the applied loads. Even in test T4-A, in which the supplementary reinforcement yielded, the reinforcement was capable of resisting only 60% of the applied peak load.

The results of tests for T4 showed that the peak load could be increased by about 50% comparing to the comparison test for T1 to which supplementary reinforcement is not provided. Therefore it is proposed to increase the concrete breakout resistance when supplementary reinforcement is present around each anchor of an anchor group. The supplementary reinforcement must be arranged as in T4 (4 U-shaped stirrups at a distance ≤ 4 in. or ≤ 0.15 h_{ef} from the anchor).

5. Conclusion

It is prudent to use ACI 349-01 using the equation with $h_{ef}^{1.67}$ in calculating the tensile breakout capacity of single large anchor bolt with a large head.

In a more general model, the supplementary reinforcement should be dimensioned to take up 100% of the applied load, thus neglecting the contribution of the concrete. The characteristic resistance of the supplementary reinforcement is given by the bond capacity of the supplementary reinforcement in the anticipated concrete cone, which should be assumed radiating from the head under an angle of 35 degrees.

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