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BEHAVIOR OF LARGE ANCHOR LOADED IN TENSION IN UNREINFORCED/REINFORCED CONCRETE

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ABSTRACT

This paper presents the research results of pullout tensile tests for extra-large sized cast-in-place anchor bolts. These tests were performed to evaluate the tensile performance of large sized anchor not addressed by ACI 318 and ACI 349 Appendix D, that is greater than 2 in anchor diameter or 25 in embedment depth. These tests are also intended to verify if the anchor bolts used in nuclear power plant are within safe margin. The effects of regular (conventional) and special reinforcement on the strength of large sized anchor bolt are also evaluated. From these test results the applicability of design formulae for small sized anchor to the large sized anchors are reviewed and compared with predicted value based on the existing formulae. Suggestions are made how to incorporate the effects of deep embedment or large diameter on the tensile capacity of cast-in-place anchor bolts.

Keywords: anchor; anchor bolt; concretes; embedments; cast-in-place; tension tests

1. INTRODUCTION

The current design of Nuclear Power Plants in Korea utilizes large anchor bolts exceeding 2 in. in diameter with a yield strength of 140 ksi and an ultimate strength of 155 ksi, and with embedments exceeding 25 in. While the tensile behavior of smaller anchors has been studied extensively the world over, large anchors have not been adequately addressed. Large anchors were tested in tension to develop design criteria for large anchors not addressed by ACI 318-02 (ACI 318 2002) Appendix D and ACI 349-01 Appendix B (ACI 349 2001), and to evaluate the effectiveness of capacity-prediction methods developed for smaller anchors.

To evaluate the effects of anchor diameters and embedment depths, various types of anchor specimens were tested, ranging from 2.75 to 4.25in. in diameter, and with embedment depths ranging from 25 to 45 in. The material used to fabricate the test specimens is of the same type and quality previously used for the Korean NPP NSSS anchor bolts. The anchor bolts are SA540 Gr. B23 with F_y =140ksi and F_u =155 ksi conforming to ASME SA540.

2. PREDICTED METHODS FOR TENSILE CAPACITY

Many empirical formulas have been proposed to calculate the tensile breakout capacity and more recently the Concrete Capacity Method (Fuchs et al. 1995) has been proposed as a derivative of the so-called Kappa Method (CEB 1991). As a results of work in ACI Committees 318 and 349 (Subcommittee 3), it has been proposed to modify the CC Method slightly, changing the exponent of the method from 1.5 to 1.67 at an effective embedment depth of 250mm (9.84in).

The tensile breakout capacities of large anchors were computed by the CC-Method using embedment-depth exponents of 1.5 (Eq. 1) and 1.67 (Eq. 2) in the formula proposed with mean values of test results for small sized anchors. Capacities were also computed using the 45-Degree Cone Method (Eq. 3) recommended by ACI 349-97 and the CC-Methods (Eq. 4) as per ACI 349-01, considering 5% fractile values. For ACI 349-01 the value k=16 is valid for cracked concrete, but the tests were performed in uncracked concrete. Therefore a k-value k=20 (k=16 x1.25) was used for calculations of predicted capacity.

The formulas used for computation of tensile capacities are delineated below (NUREG/CR-5563, 1999). The purpose of this computation is to test the applicability of the formulas to large anchors.

$$N_n = 40\sqrt{f'_c}\,h_{ef}^{\;1.5} \qquad \text{(CC Method with exponent of 1.5 at } h_{ef}\text{)} \qquad (1)$$

$$N_n = 26.7\sqrt{f'_c}\,h_{ef}^{\;1.67} \qquad \text{(CC Method with exponent of 1.67 at } h_{ef}\text{)} \qquad (2)$$

$$N_n = 4\sqrt{f'_c}\,\pi h_{ef}^2 (1 + d_h / h_{ef}) \qquad \text{(45-Degree Cone Method ACI 349-97)} \qquad (3)$$

$$N_n = 20\sqrt{f'_c}\,h_{ef}^{\;5/3} \qquad \text{(CC-Method - uncracked ACI 349-01)} \qquad (4)$$

Where,
f'_c = Concrete Compressive Strength (psi),
h_{ef} = Effective Embedment Depth (inch),
d_h = Diameter of Anchor Head (inch)

3. DESCRIPTION OF EXPERIMENTAL PROGRAM

3.1 Test Setup

The schematic layout of the test frame assembly, consisting of a loading frame, loading plate, jack assembly, load cell and other items, and a photograph of the actual assembly are shown in Figure 1.

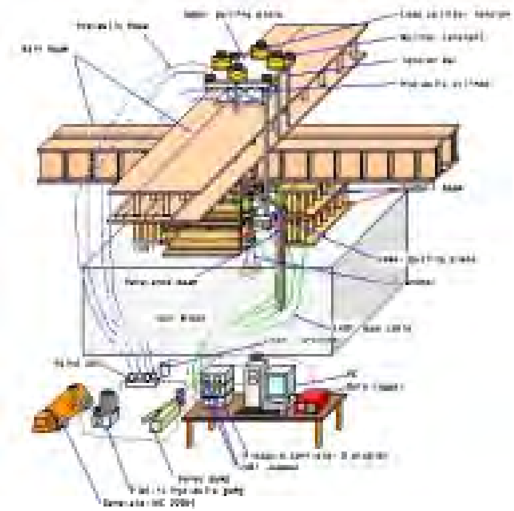


Figure 1 Tension Test Setup (Schematic and Photo)

3.2 Test Specimens

Five test specimens for tension tests of large anchors were chosen to evaluate the effects of embedment depth, anchor diameter and supplementary reinforcement patterns on the tensile capacity of the anchors. Four replicates of each specimen were tested. The schematic sketches of test specimens are shown in Figure 2 and detailed figures showing the location of various gauges are shown in Figure 3. The size of the concrete test blocks was determined so as to avoid failure of the adjacent anchor in the same block by utilizing the support beam of the loading frame. The distances between the axis of each anchor and the inner face of the support reaction are 62.2 in., 81.9 in., 100.8 in., 62.2 in. and 62.2 in., for each Test Specimens T1, T2, T3, T4 and T5, respectively. Wooden and steel frames were constructed to suspend the cast-in-place anchors in the correct position and at the correct embedment depth, as illustrated in Figure 2.

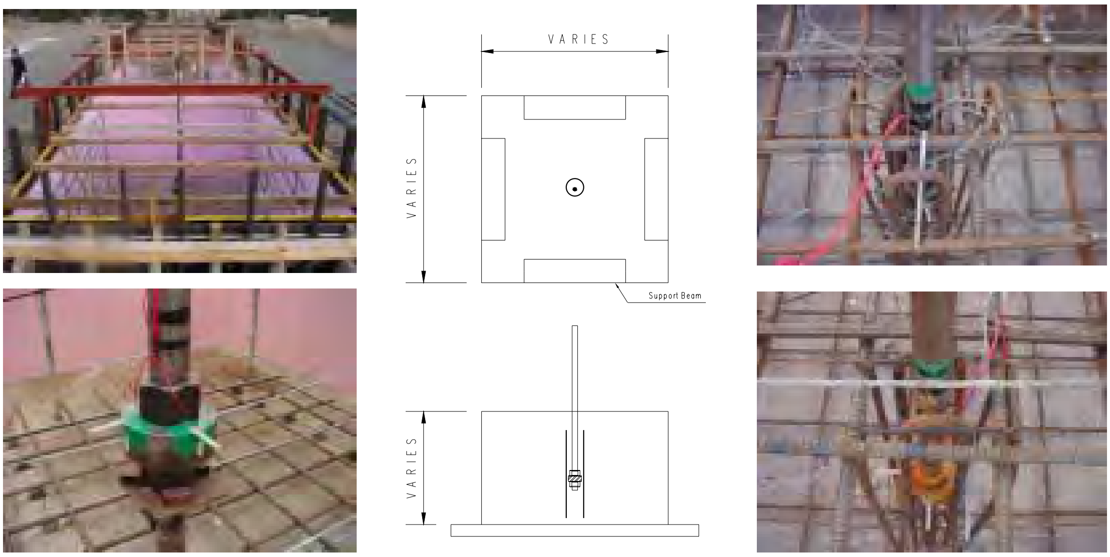


Figure 2 Schematic Sketch and Photo of Tension Test Specimens (T1, T2, T3, T4 and T5)

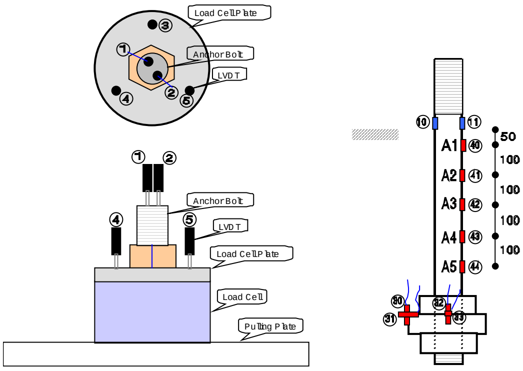


Table 1 Summary of Tension Test Specimens

Specimen No.	Reinforce-ment	Anchor Diameter D_b (in)	Embed. Depth h_{ef} (in)
T1	None	2.75	25
T2	None	3.75	35
T3	None	4.25	45
T4	Supp. #1	2.75	25
T5	Supp. #2	2.75	25

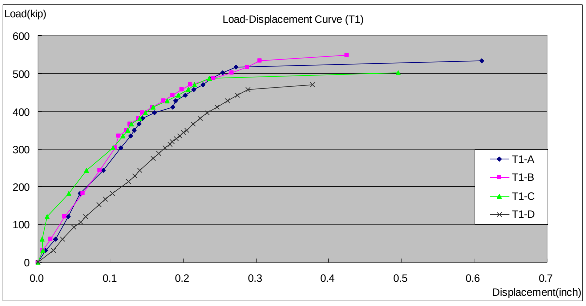
Figure 3 Location of LVDT and Strain Gauges (T1)

4. TEST RESULTS

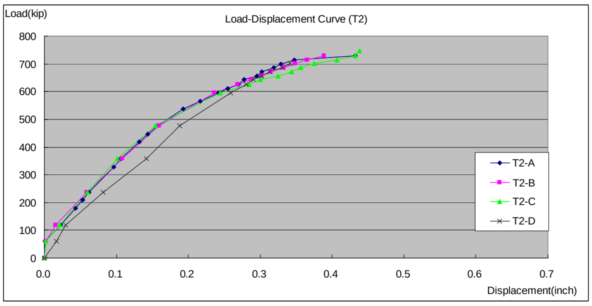
4.1 Load-displacement Curves

The load-displacement curves for specimens T1, T2, T3, T4 and T5 are shown in Figures 4(a) through 4(e). These load-displacement curves are plotted using the data measured at the top of each anchor. The relationship of load and displacement for each sample varies based on the age and the concrete strength at the time of test.

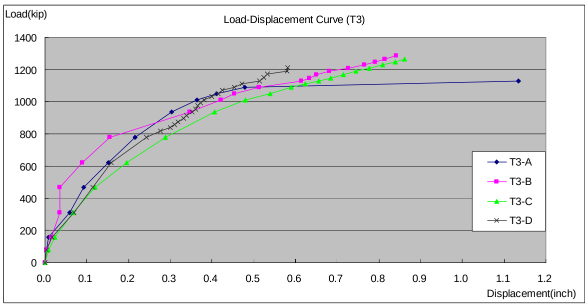
The projected lengths (from top of concrete face to top of anchor bolt) of anchor specimens T1, T2, T3, T4 and T5 are 41.7, 48.6, 53.1, 41.7 and 41.7 in. respectively. The measured displacements shown in Figure 8 include the portion of displacements for the projected length and thus most of the measured displacement came from the projected length. The actual displacements at the top of the concrete surface, which are accumulated along the embedded portion of anchor, will be very small before concrete breakout failure.



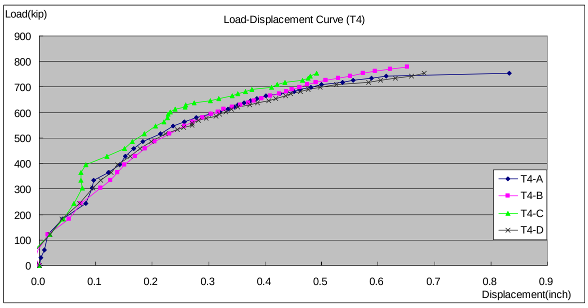
(a) T1-A to T1-D



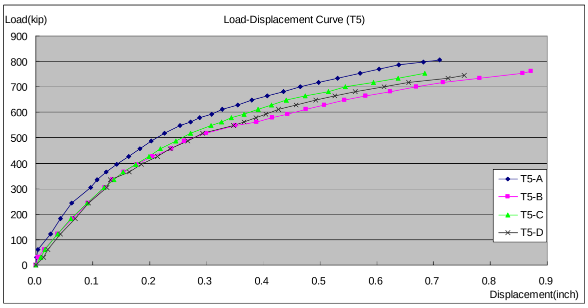
(b) T2-A to T2-D



(c) T3-A to T3-D



(d) T4-A to T4-D



T5-A to T5-D

Figure 4 Load-displacement curves for Test Specimens

4.2 Specimen Behavior

The loads corresponding to the specified tensile yield 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 tested 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.

Cracking patterns for un-reinforced tension test specimens are shown in Figure 5. 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 α=20° to α=30°; typical crack profiles are shown in Figure 5(b). The cracking patterns on the sides and top face are shown in Figure 5(a). With specimens T4 and T5 no cracking was observed at the concrete surface and no cores were taken to check whether a cone had begun to form.

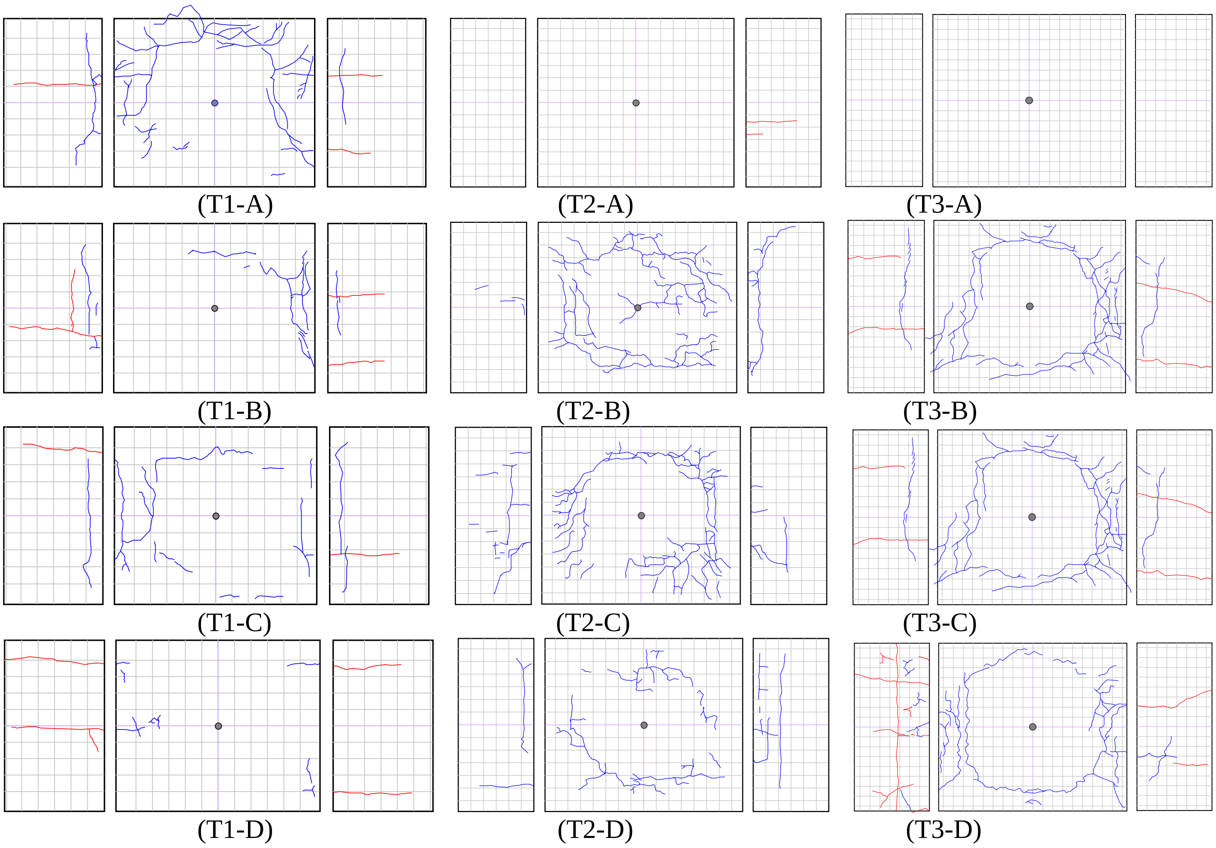


Figure 5(a) Cracking Pattern for Four Replicates (A, B, C & D) of Specimen T1, T2 and T3

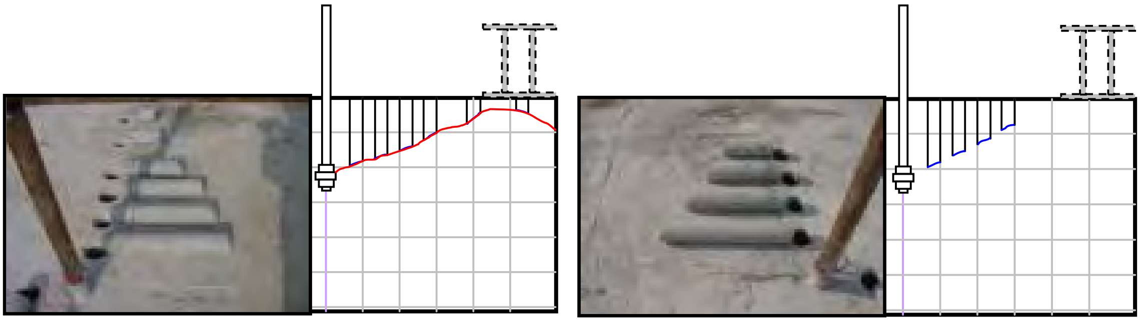


Figure 5(b) Typical Internal Crack Profile in Specimen (T1)

4.3 Predicted and Tested Tensile Breakout Capacities

Tension test results for unreinforced tensile test specimens T1, T2 and T3 are presented in graphical and tabular form as shown in Table 2(a) and Figure 3(a). The predicted capacities, and the mean and 5% fractile of the tested capacities are summarized by effective embedment depths. Four replicates of each specimen were tested. Since each replicate was tested at a different age, the tensile test results were normalized to a compressive strength of 5500 psi to facilitate comparison. Test results for the reinforced tensile test specimens T4 and T5 are similarly summarized in Table 2(b) and Figure 3(b).

The formula for computing tensile capacities of anchor bolts in concrete proposed by ACI 349-01 are based on 5% fractile, while those for the CC Method are based on the mean of test results. Therefore, it is reasonable to compare the mean and 5% fractile values of test results to the predictions by CC-methods and ACI349-01 respectively. Comparisons of the test results to capacities predicted by formulas are included in this report with distinguishing the technical basis of each formula.

Table 2(a) Tension Test Results and Predictions for Unreinforced Specimens (T1, T2, T3)

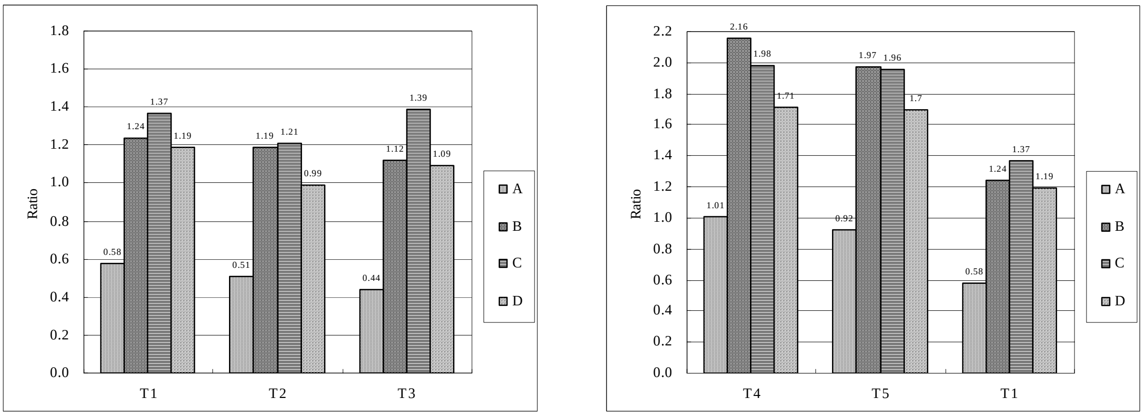
		Concrete Breakout Capacities (kip) by Embedment Depths		
		25 in. (T1)	35 in. (T2)	45 in. (T3)
Predictions	ACI349-97	676	1305	2138
	ACI349-01	317	555	844
	CCD-1.5hef	371	614	895
	CCD-1.67hef	428	750	1142
Test	Mean	509	744	1242
	COV (%)	5.8	2.8	6.1
	5% fractile	393	662	944
	5% fractile/ Mean	0.77	0.89	0.76

		Ratio of Observed to Predictions			
		25 in(T1)	35 in(T2)	45in(T3)	Mean
5% Fractile of Test Results	F5% /ACI349-97(A)	0.58	0.51	0.44	0.51
	F5%/ACI349-01(B)	1.24	1.19	1.12	1.18
Mean of Test Results	Mean/ CCD-1.5hef(C)	1.37	1.21	1.39	1.32
	Mean/ CCD-1.67hef(D)	1.19	0.99	1.09	1.09

Table 2(b) Tension Test Results and Predictions for Reinforced Specimens (T4, T5)

		Concrete Breakout Capacities (kip) by Embedment Depths		
		25 in. (T4)	25 in. (T5)	25 in. (T1)
Predictions	ACI349-97	676	676	676
	ACI349-01	317	317	317
	CCD-1.5hef	371	371	371
	CCD-1.67hef	428	428	428
Test	Mean	733	725	509
	COV (%)	1.7	3.5	5.8
	5% fractile	685	625	393
	5% fractile/Mean	0.93	0.86	0.77

		Ratio of Observed to Predictions			
		25 in. (T4)	25 in. (T5)	25 in. (T1)	T4/T1
5% Fractile of Test Results	F5% /ACI349-97(A)	1.01	0.92	0.58	1.74
	F5%/ACI349-01(B)	2.16	1.97	1.24	1.74
Mean of Test Results	Mean/CCD-1.5hef(C)	1.98	1.96	1.37	1.45
	Mean/CCD-1.67hef(D)	1.71	1.70	1.19	1.44



(a) Unreinforced Specimens (T1, T2 and T3) (b) Reinforced Specimens (T4 and T5)

Figure 6 Ratio of Test Results (5% fractile and mean) to Predicted Capacities for each method

5. ANALYSIS OF TEST RESULTS FOR UNREINFORCED SPECIMENS T1, T2 AND T3

The capacities computed by ACI349-97 are greater than (overestimate) the test values ($F_{5\%}$) for all embedded depths. The 5% fractile values ($F_{5\%}$) of test results are on average 120% of the predicted value for uncracked concrete per ACI 349-01. The ratio of the mean test value and the prediction with the least COV is 1.10. The slope of the trend line, however, shows the ratio of observed to predicted capacity varies with effective embedment having a little systematic error. Therefore it is difficult to suggest that a value of 29.4 should replace the current value of 26.7 in the CC-formula to predict the tensile capacity of large anchors and that a value of 22 might be used instead of 20 for uncracked concrete in ACI 349-01

The higher ratio $F_{u,test} / F_{u\,,\,predicted}$ when comparing the 5% fractiles with each other instead of the average values is due to the rather lower scatter of test results. On average the coefficient of variation (COV) is around 5%. This results in an average ratio $N_{u,5\%}$ /$N_{u,m}$, of 0.81, while in ACI 349-01 a ratio 0.75 is assumed. In actual structures the concrete strength and thus the concrete cone resistance might vary more than in the test specimen. Therefore the ratio $N_{u,5\%}$ / $N_{u,m}$ assumed in ACI 349-01 should be recommended to be maintained.

The test data for large anchors with 25-, 35-, and 45-in. embedment depths are compared with test values for anchor bolts with embedment depths of greater than 10in (250mm) available from other test sources. Ratios of mean observed values to the capacities predicted using the CC Method (exponents of 1.5 and 1.67 at h_{ef}) and the 45-degree cone method are plotted against the effective embedment (h_{ef}) in Figure 7.

The regression curve fits for the 5% fractile and mean values of test results for three embedment depths show the test results are closer to the CC formula with an embedment depth exponent of 1.67 than those for other methods as shown in Figure 7. The CC formula for tensile breakout capacity of conventional-sized anchor with an embedment-depth (h_{ef}) exponent of 1.67 results in the best prediction of the capacity of very large anchors, as shown in Table 2(a) and Figure 6.

Figure 8(a) shows that the 45-degree Cone Method has a much greater systematic error than those of the CC-Method. The exponent of 1.67 is associated with a little greater systematic error than the exponent of 1.5 for the large anchors of these limited tests while the ratio of predicted value with the exponent of 1.67and the tested capacity is closer to 1.0 than that with the exponent of 1.5 (Figure 8(b) and (c)). ig. 8(f) shows that the CC-Method changing the exponent of the method from 1.5 to 1.67 at an effective embedment depth of 250mm (10in.) predicts the average failure loads of anchors with $h_{ef} \geq 250$ mm (10in.) quite well. Only two test points at $h_{ef} = 250$ mm (10in.) fall below the assumed 5%-fractile equal to 75% of the average value.

Thus the exponent of 1,67 and k-value which can predict the capacities of large anchors with lesser margin of error need to be adjusted to better correspond with the test results. The slopes of the trend lines with these test data and the data from other sources including the test data having the embedments (200mm to 600mm) shows that the ratio of observed to predicted capacity with changing the exponent from 1.5 to 1.67 at embedment is a least systematic error.

At failure the related pressure under the head was on average 4.37, 3,63 and 5,31 for T1, T2 and T3. It was much smaller than the pressure allowed by ACI 349-01 (P = 10 f_c). Therefore, the Equation with $h_{ef}^{1.67}$ should only be used for anchors with a large head, where in uncracked concrete the pressure under the head at the characteristic failure load is $P_k \geq 3 \cdot f_c$. In cracked concrete 0.8 times the above given value should be used.

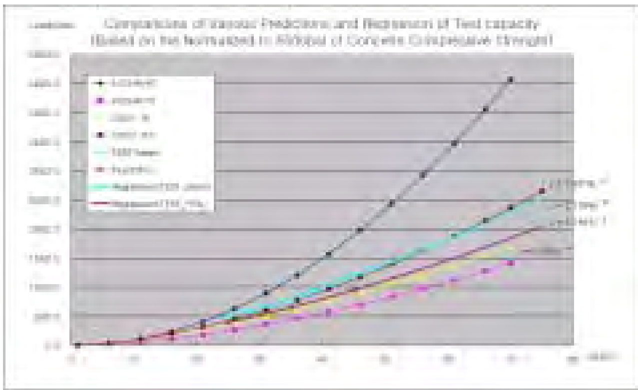
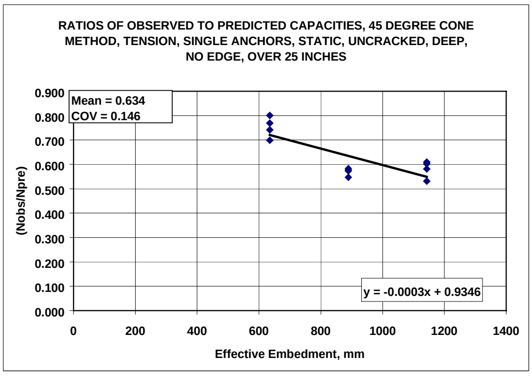
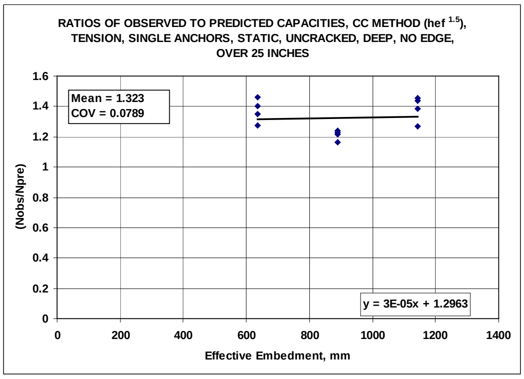


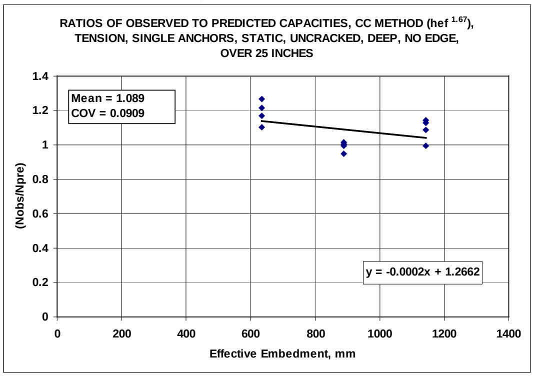
Figure 7 Curve Fittings for Test Results and Predictions



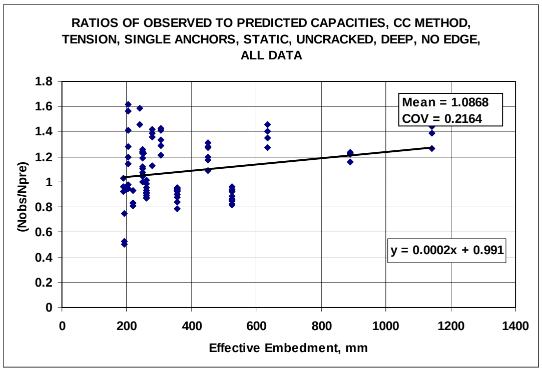
(c) 45-degree Cone Method



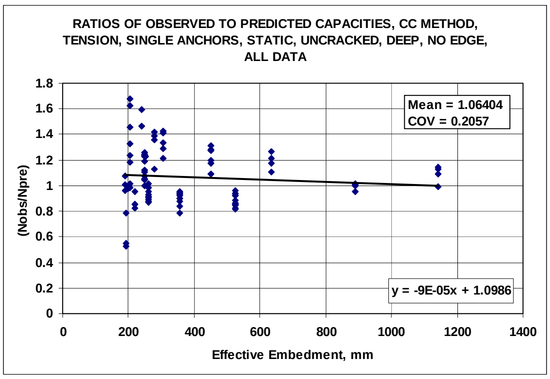
(a) CC-Method with 1.5 (test data only)



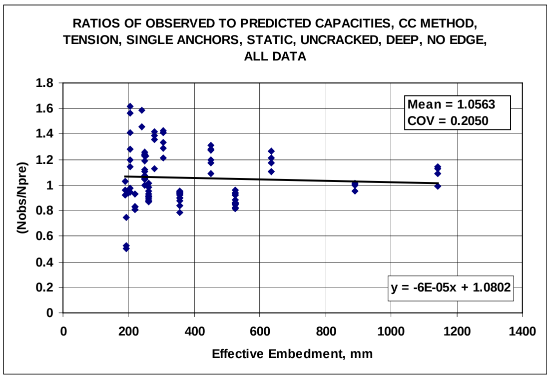
(b) CC-Method with 1.67 (test data only)



(d) CC-Method with 1.5(w/ other sources)



(e) CC-Method with 1.67(w/ other sources)



f) CC-Method changing from 1.5 to 1.67 at 250mm (10in.) (w/ other sources)

Figure 8 Ratios of Observed to Predicted Concrete Tensile Breakout Capacities

6. EFFECT OF SUPPLEMENTARY REINFORCEMENT

6.1 Reinforced Specimen T4

Test specimens T4, with supplementary reinforcement, are shown in Figure 2. While Test specimen T4-A was tested to concrete breakout failure, Test specimen T4-B, T4-C, T4-D were not tested up to apparent concrete break-out failure due to anchor bolt strains exceeding yield point. The mean tested failure load (759 kips) of those specimens, however, is close to the sum (805 kips) of the calculated reinforcement strength (378kips) and the unreinforced concrete strength (427kips) by the CC-Method (exponent =1.67). It can be inferred that the adopted reinforcement pattern effectively acted in the anchorage system for tension.

The tested breakout strength of the un-reinforced test specimen T1 with the same embedment depth of 25 in. was 513 kips; adding this to the calculated yield strength of the supplementary reinforcement (378kips), one obtains a predicted capacity for Specimen T4 of 891 kips. From this estimation, the effective increase in capacity due to supplementary reinforcement is roughly 246 kips, or about 65% of the calculated yield strength of the supplementary reinforcement.

The loading on T4-A was increased to the expected total yield force of the supplementary reinforcement, so that the load distribution to each of the two reinforcement groups could be estimated. The load resisted by the supplementary reinforcement in the nearer concentric circle (4.2 in. from the axis of the anchor) was 2.2 times the load resisted by the equal area of supplementary reinforcement on the far concentric circle (8.5 inch from the axis of the anchor).

According to the measured strains in the strain gauges attached to reinforcing bars, the reinforcing bars close to the anchor were more effective in increasing the tensile capacity, and their maximum stress occurred at the level close to that of the anchor head.

6.2 Reinforced Specimen T5

All four replicates of test specimen T5, with supplementary reinforcement as shown in Figure 2 were not tested to breakout failure due to safety concerns related to the exceedance of specified tensile yield in the anchors. The mean tested capacity (766 kips) was much smaller than the sum (1,128 kips) of the calculated reinforcement strength and concrete strength per CC-method. From these test results, this layout of supplementary reinforcement contributes with a low level of effectiveness to the capacity of the anchor. This is corroborated by measured strains in the gauge attached to the reinforcing bar, which indicates little strain in the reinforcement. As noted above, however, the replicates of Specimen T5 were not fully loaded up to concrete breakout failure due to safety concerns.

As a consequence, the results of series T5 are judged to not be useful in verifying the effectiveness of supplementary reinforcement. By comparing results from Specimens T4 with those of Specimens T5, however, it is still possible to make inferences regarding the comparative effectiveness of different supplementary reinforcement patterns. At the same applied load, stresses in the supplementary reinforcement of Specimens T5 along the inner and outer circles, with twice the reinforcing bars of T4, are less than half those of the inner supplementary reinforcement in Specimens T5. The relative trends of stress distribution are similar for each reinforcement in both T4 and T5 and thus it can be inferred that the increase in tensile capacity is approximately proportional to the amount of supplementary reinforcements.

The load-displacement curves of Tests T4 show that the peak load was practically reached in the tests. In tests T5 the load could still be increased. In tests T4 the supplementary reinforcement was not strong enough to resist the concrete breakout load. In tests T5 the loading was stopped before the supplementary reinforcement could be fully activated. Therefore it is not possible to evaluate a general model from the test results. Test results show, however, that with supplementary reinforcement arranged as in Specimens T4 and dimensioned for about 80 to 100% of the expected ultimate or characteristic concrete breakout capacity, the observed breakout capacity was increased by about 50% over the unreinforced case. This result can reasonably be used in design.

7. SUMMARY AND CONCLUSIONS

7.1 Large anchors without supplementary reinforcement

Test results show that ACI 349-97 (Eq. (3)) significantly overestimates the tensile breakout capacity of large anchors. The ratio $N_{u,test} / N_{u,pred.}$ decreases with increasing embedment depth (Fig. 8(a)). Furthermore the slope of the cone was much flatter than 45° degrees. Therefore the overestimation of the failure loads would be even larger for fastenings at an edge or for group fastenings. For these reasons this method should not be used in the design.

The CC-method with $h_{ef}^{1.5}$ (Eq. (1)) is conservative for large anchors (Fig. 8(b)). This is probably due to the fact, that this method is based on linear fracture mechanics which is valid only for anchors with a small head. However, the tested anchors had a rather large head. The test results can best be predicted by the CCD-method with $h_{ef}^{1.67}$ (Eq. (2)). This method predicts the failure load of large anchors rather well (see Figs. 7, 8(e)). On average the measured failure loads are about 10% higher than the predicted values. However, if all available results are taken into account (see Fig. 8(f)), a change of Eq. (2) seems not to be justified.

It is proposed to calculate the characteristic resistance of single large anchor bolts with a large head as present in the tests according to ACI 349-01 using the equation with $h_{ef}^{1.67}$. However, according to the test results, the average cone angle is not 35° degrees (as assumed in the CCD-method) but only about 25° to 30° degrees. Therefore the characteristic spacing $s_{cr,N}$ and characteristic edge distance $c_{cr,N}$ are probably larger than $s_{cr,N} = 2c_{cr,N} = 3\ h_{ef}$. assumed in ACI 349-01. Therefore it seems prudent to calculate the resistance of anchorages at an edge or corner or of group anchorages according to ACI 349-01, but with $s_{cr,N} = 4.0\ h_{ef}$ instead of $s_{cr,N} = 3.0\ h_{ef}$ as given in ACI 349-01.

7.2 Large anchors with supplementary reinforcement

In Tests T4 the supplementary reinforcement was not strong enough to resist all the applied loads. Even in test T4-A, in which the supplementary reinforcement yielded, only about 60% of the applied peak load was resisted by the reinforcement. In tests T5 with a stronger reinforcement the tests had to be stopped because of tensile yielding of the anchors before the supplementary reinforcement had been fully mobilized. Therefore, the results of these tests cannot be used to develop a general design model for anchors with supplementary reinforcement.

Nevertheless, the results of Tests T4 showed that the peak load could be increased by about 50% compared to the comparison tests T1 without supplementary reinforcement. Therefore it is proposed to increase the concrete breakout resistance calculated according to Section 7.2 by a factor of 1.5 if supplementary reinforcement is present around each anchor of an anchor group. The supplementary reinforcement must be arranged as in tests T4 (4 U-shaped stirrups at a distance ≤ 4 in. or ≤ 0.15 h_{ef} from the anchor) and dimensioned for the characteristic resistance of the fastening calculated according to Section 7.2.

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 supplementary reinforcement should be designed with a strut-and-tie model. 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. The bond capacity should be calculated according to codes of practice (for example, ACI 318-02 or Eurocode 2 (2004). It is limited by the yield capacity of the bars. This model is described in detail in the CEN Specification for the Design of Fastenings (CEN 2004) or in the CEB Design Guide (CEB 1997).

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