REINFORCING DETAILS FOR ANCHOR BOLTS SUBJECTED TO REVERSED CYCLIC SHEAR

R. E. Klingner (I)
J. A. Mendonca (II)
J. B. Malik (III)
<u>Presenting Author</u>: R. E. Klingner

SUMMARY

An experimental investigation was conducted using single ASTM A307 anchor bolts embedded in normal weight concrete and loaded quasi-statically in monotonic and reversed cyclic shear. Using 180 deg hairpins, reinforcing details were developed allowing an anchor to develop its full shear strength even when placed at small distances. These details were found to be satisfactory under reversed cyclic as well as monotonic shear.

INTRODUCTION

Anchor bolts embedded in concrete are a common element in many types of construction. Such anchor bolts must often transmit combinations of tension and shear to the concrete. Anchor bolts are used in critical applications involving resistance to reversed cyclic loads due to seismic shaking. To develop satisfactory design procedures for such conditions, it is necessary to study anchor bolt behavior under cyclic loads.

OBJECTIVES AND SCOPE

The general objective of the investigation was to propose guidelines for the design of anchor bolts loaded quasi-statically under monotonic or reversed cyclic shear. The investigation was divided into severalphases. Due to space limitations, only two of those phases will be discussed here. The objectives of those phases were:

1) to develop reinforcing details allowing an anchor bolt to develop its full strength in shear even when placed at edge distances small enough to cause reduced capacity; and

2) to investigate the effects of reversed cyclic shear loads on the ultimate shear resistance of short anchor bolts, and to recommend design procedures for such bolts.

(I) Associate Professor of Civil Engineering, University of Texas at Austin, USA

(II) Consulting Engineer, Salvador, BRAZIL

(III) CBM Engineers, Houston, Texas, USA

BACKGROUND

For small embedment lengths, an anchor bolt loaded in shear will fail by pulling out of the concrete, leaving a cone-shaped hole (Figs. 1, 2). This failure mode is very similar to that observed for bolts in tension. To develop its full resistance in shear, an anchor bolt or stud must be embedded sufficiently to preclude this type of tensile pullout failure. Several procedures are available for calculating that minimum embedment (Refs. 1, 2). While a discussion of those is beyond the scope of this paper, the authors studied all of them and recommend those of References 3 and 4 as being rational and in satisfactory agreement with available test data. All shear tests discussed here had sufficient embedment to develop the minimum specified tensile strength of the anchor steel, and therefore to prevent this type of failure.

EXPERIMENTAL INVESTIGATION

Test Specimens and Materials

Tests were conducted on a total of 56 anchor bolts embedded vertically in concrete blocks measuring 8 x 3 x 2 ft, shown in Fig. 3. So that the reinforcement would not affect bolt shear resistance, the blocks were reinforced below the level of the bolts. The characteristics of each block are shown in Table 1, along with the mechanical characteristics of the materials used.

All anchor bolts conformed to ASTM A307 and were hexagonal-headed, with a nominal diameter of 3/4 in. They were 12 in. long and embedded to a depth of 8 in. To minimize material property variations, all bolts were obtained from the same lot. The bolts had an actual shank diameter of 0.72 in. As determined by tests the bolts had an ultimate tensile strength of 27.1 kips and an ultimate shear strength of 20.1 kips. Using the nominal gross area, these correspond to ultimate tensile and shear strengths of 62.3 ksi and 45.5 ksi, respectively. The minimum specified tensile strength was 60 ksi. As discussed below, some bolts were placed in concrete reinforced with hairpins made from #5 deformed bars, Grade 60.

Test Setup and Instrumentation

The test setup, shown in Fig. 4, was chosen specifically to eliminate direct reaction against the block, which might have interfered with the test results by creating a compressive stress field in the concrete between the bolt and the reaction point.

Test data consisted of applied loads, bolt deflections, and hairpin strains (for those bolts having hairpins). As shown in Fig. 5, the deflection of the bolt relative to the block was measured using a linear potentiometer. The strain gages attached to one or both hairpin legs (Fig. 6) were used primarily to detect possible yielding of the hairpins, although nominal hairpin forces were also computed.

SUMMARY OF EXPERIMENTAL RESULTS: MONOTONIC SHEAR TESTS

All test results are summarized in Table 1. The preliminary tests of Block 1 showed that different loading plate sizes and surface preparation did not significantly affect bolt strength, and all subsequent tests were carried out using 6-x-6 in. loading plates and normal surface preparation. Tests on 16 bolts in Block 2 confirmed that under monotonic shear loading, bolts at large edge distances (12 in.) failed in shear at applied loads not more than 10 percent above the ultimate bolt shear resistance as previously determined by pure shear tests. Bolts at small edge distances failed suddenly by cracking of concrete in a semiconical failure surface (Fig. 2), at loads considerably below the ultimate bolt shear resistance.

Procedures for predicting shear capacity as governed by steel and concrete failure, and for estimating the minimum critical edge distance necessary for a bolt to reach its full capacity, are discussed in detail in References 1, 2, and 4.

As shown in Table 1, the next tests (Block 3) consisted of monotonic shear tests on 16 bolts, with 2- and 4-in. edge distances. Some bolts were placed in plain concrete, while others were placed in concrete reinforced with 180 deg hairpins. As shown in Fig. 6, four types of hairpins were used. Hairpin Type 1, used with bolts at 4-in. edge distances, was placed away from the bolt and close to both the top and side surfaces of the blocks. Hairpin Types 2 and 3 were placed directly against the bolt and close to the top surface of the concrete. These two hairpin types were identical except for the edge distances of the bolts they reinforced. Type 2 hairpins were used with bolts at 4-in. edge distances, while Type 3 hairpins were used with bolts at 2-in. edge distances. Type 4 hairpins were placed directly against the bolt, but relatively far from the top surface of the concrete.

All hairpins were #5 bars, Grade 60. This hairpin diameter was selected to avoid yield under the maximum bolt shear capacity. To prevent anchorage failure, a leg length of 23 in. was provided in accordance with the development length recommendations of ACI Committee 408 (Ref. 5). All hairpins had an inside bend diameter of 3.75 in., corresponding to six bar diameters as specified in Reference 6.

Bolts under Monotonic Shear Load in Reinforced Concrete

Typical load deflection curves are shown in Fig. 7. All bolts behaved similarly until spalling occurred in front of the hairpin. After spalling, all bolts reinforced with hairpin types other than Type 4 reached maximum loads as high as those reached by bolts in plain concrete at large edge distances. Ultimate bolt resistance is believed to be limited by kinking of the bolt between the loading plate and the hairpin, and the hairpins must therefore be designed to resist forces as large as the ultimate tensile strength of the bolt. Due to the large distance between the Type 4 hairpins and the top of the block, that type of hairpin resulted in much more flexible behavior than the other types, and Type 4 hairpins were not studied further. It was concluded that hairpin Types 1 through 3 would provide satisfactory concrete reinforcement for bolts located at less than the critical edge distance and loaded in monotonic shear.

SUMMARY OF EXPERIMENTAL RESULTS: CYCLIC SHEAR TESTS

The last test series comprised reversed, quasi-static, cyclic load tests on bolts in plain as well as reinforced concrete. As shown in Table 1, Block 3 contained four bolts in plain concrete at 10 in. edge distances. Block 4 contained 16 bolts at edge distances of 2 and 4 in., all in concrete reinforced with hairpin Types 1, 2, or 3. As shown in Fig. 6(b), all such cyclically loaded bolts were provided with two hairpins, one for resisting shear in each direction.

All bolts in this test series either were located at sufficiently large edge distances or provided with sufficient reinforcement to prevent concrete failure. Two types of cyclic loading program were used. Type 1 consisted of a series of reversed cycles to monotonically increasing maximum loads, and Type 2 consisted of one reversed cycle to high maximum loads, followed by a series of reversed cycles to monotonically increasing maximum loads. The shear behavior of bolts was basically independent of the loading program. To save space, figures are presented for the Type 1 loading program only, and the relatively minor differences in response are discussed briefly.

Bolts under Reversed Cyclic Load in Plain Concrete

Figure 8 shows typical load-deflection results for bolts at large edge distances in plain concrete. Bolts subjected to the Type 1 loading program had an almost constant secant stiffness of about 100 kips/in. during the first three cycles to 5 kips. The stiffness decreased to about 20 kips/in. after cycling to 10 kips, and the hysteretic curves began to exhibit pinching near the origin. During the cycles to 15 kips, the stiffness reduced further, and bolt failure occurred in the steel due to shear, as determined by examination of the failure surface. Bolts subjected to the Type 2 loading program behavior similarly, except that the sharp reduction in stiffness occurred after the large loading pulse, the final failure took place during one of the three subsequent cycles to 10 kips.

Bolts under Reversed Cyclic Load in Reinforced Concrete

Figure 9 shows that during the cycles to 5 kips, bolts with Type 1 hairpins subjected to Loading Program 1 exhibited stiffness characteristics similar to those described above for bolts at large edge distances. During the 10-kips cycles, however, concrete in front of the top hairpin spalled off, causing an abrupt decrease in stiffness from 100 to only about 12 kips/in. Deflection increased during each successive cycle to the 10-kip level. In one of these anchorages, the concrete between the bolt and to hairpin crushed under repeated 10-kip load cycles, resulting in still more deflection. Final failure in the bolts was due to shear, and typically took place during the 15-kip cycles at maximum deflections exceeding 2 in. Under the Type 2 loading program, the spalling and decrease in stiffness was observed during the large pulse, and final bolt failure occurred in the steel during one of the three subsequent 10-kip cycles.

Comparison of Figs. 9 and 10 shows that during the first cycles to 5 kips, bolts in concrete reinforced with hairpin Types 2 and 3 and subjected to the Type 1 loading program behaved similarly to bolts with Type 1 hairpin.

During the cycles to 10 kips, the concrete in front of the to hairpin spalled off, and the bolt experienced large deflections. Bolt failure typically took place due to combined tension and shear during one of these 10-kip cycles, at a deflection of about 2 in. Bolts subjected to the Type 2 loading program failed at slightly smaller deflections.

Based on these test results, it was concluded that anchor bolts at edge distances greater than critical in plain concrete would perform satisfactorily under reversed cyclic shear loads, with ultimate failure occurring in the bolts themselves as a result of low-cycle fatigue. Similarly, it was concluded that anchor bolts located closer than the critical edge distance would perform satisfactorily if placed in concrete reinforced by 180 deg hairpins corresponding to hairpin Types 2 and 3, i.e., hairpin reinforcement placed directly against the bolt and as close as possible to the top of the block (Fig. 6(b)). Although Type 1 hairpins gave significantly increased spalling resistance and good load-deflection performance in some tests, one reversed cyclic load test referred to earlier resulted in failure due to large deflections when the concrete spalled away from the bolt and the hairpin. Since this mode of failure would always be a possibility under reversed cyclic loads, Type 1 hairpins are not recommended for such applications. Under reversed cyclic loads, bolts placed in both reinforced and unreinforced concrete typically failed at loads about 50 percent less than those resisted by monotonically loaded bolts, owing to the effects of low-cycle fatigue.

SUMMARY AND CONCLUSIONS

Following an extensive literature study (Refs. 1, 2), an experimental investigation was conducted using single A307 anchor bolts embedded in normal weight concrete and loaded quasi-statically in monotonic and reversed cyclic shear.

Using 180 deg hairpins, reinforcing details were developed using an anchor bolt to develop its full shear strength even when placed at less than the minimum critical edge distance as determined above. These details were found to be satisfactory under reversed cyclic as well as monotonic shear.

For good ultimate load performance under monotonic load, the hairpin should be placed directly against the anchor and as close as possible to the level at which the shear is applied. This same type of reinforcement, augmented by an additional hairpin at the base of the anchor, was found to give the best performance under reversed cyclic loads. These preferred hairpin details correspond to hairpin Types 2 and 3 as discussed here, and are shown in Fig. 6(b).

ACKNOWLEDGMENTS

The research project described herein was funded by the National Science Foundation under Grant No. ENG78-05435--Research Initiation. The research was performed at the Phil M. Ferguson Structural Engineering Laboratory, Balcones Research Center at The University of Texas at Austin.

SI EQUIVALENTS

1 in. = 25.4 mm	1 ft = 0.3048 m
1 kip = 4.448 kN	1 psi = 6.895 kPa

REFERENCES

1. Klingner, R. E., and Mendonca, J. A., "Tensile Capacity of Short Anchor Bolts and Welded Studs: A Literature Review,: ACI Journal, Proceedings V. 79, No. 4, July-August 1982, pp. 270-279.

2. Klingner, R. E., and Mendonca, J. A., "Shear Capacity of Short Anchor Bolts and Welded Studs: A Literature Review," ACI Journal, Proceedings V. 79, No. 5, September-October 1982, pp. 339-349.

3. ACI Committee 349, "Proposed Addition to: Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-76)," and "Addition to Commentary on Code Requirements for Nuclear Safety Related Structures (ACI 349-76)," ACI Journal, Proceedings V. 75, No. 8, August 1978, pp. 329-347.

4. Klingner, R. E.; Mendonca, J. A.; and Malik, J. B., "Effect of Reinforcing Details on the Shear Resistance of Short Anchor Bolts Under Reversed Cyclic Loading," ACI Journal, Proceedings V. 79, No. 1, January-February 1982, pp. 3-12.

5. ACI Committee 408, "Suggested Development, Splice, and Standard Hook Provisions for Deformed Bars in Tension," Concrete International: Design & Construction, V. 1, No. 7, July 1979, pp. 44-46.

6. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-77)," American Concrete Institute, Detroit, 1977, 102 pp., and "Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-77)," American Concrete Institute, Detroit, 1977, 132 pp.

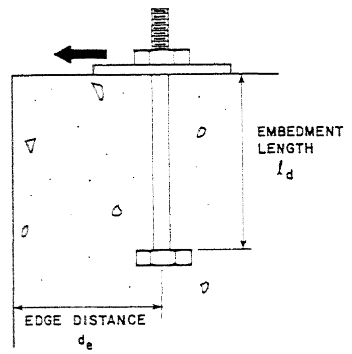

Fig. 1 Typical anchor bolt loaded in shear

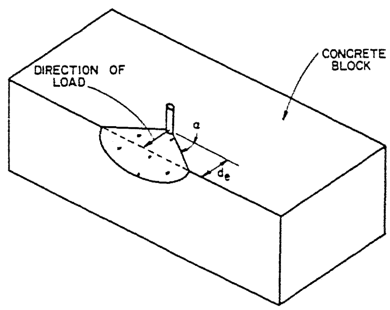

Fig. 2 Idealized semiconical concrete failure surface

TABLE 1 SUMMARY OF TEST RESULTS

Bolt	Edge distance, in.	Hairpin reinforcement type	Ultimate load, kips	Loading	Comments
			Block 1 — f_c = 4262 psi		
1	12	None	23.8	Monotonic	6x6 plate, mortar; steel failure
2			24.5		6x6 plate, mortar; steel failure
3			22.8		12x12 plate, mortar; steel failure
4			25.5		12x12 plate, mortar; steel failure
5			25.0		6x6 plate, normal; steel failure
6			25.5		12x12 plate, normal; steel failure
7			23.0		12x12 plate, Teflon; steel failure
8			23.0		6x6 plate, Teflon; steel failure
			Block 2 — f_c = 4200 psi, f_t = 380 psi		
1	2	None	3.85	Monotonic	Concrete failure
2	2		1.50		Edge damage — concrete failure
3	2		4.00		Concrete failure
4	4		6.75		Concrete failure
5	4		6.00		Edge damage — concrete failure
6	4		6.00		Preexisting crack — concrete failure
7	6		10.00		Preexisting crack — concrete failure
8	4		7.50		Concrete failure
9	6		9.30		Preexisting crack — concrete failure
10	8		19.00		Slight crack — concrete failure
11	2		4.1		Concrete failure
12	8		16.70		Edge damage — concrete failure
13	2		—		Test not conducted (damage)
14	8		19.50		Concrete failure
15	6		14.50		Concrete failure
16	4		—		Test not conducted (damage)
			Block 3 — f_c = 6200 psi, f_t = 490 psi		
1	4	1	23.0	Monotonic	
2	4	1	23.0		
3	4	1	22.5		
4	2	3	22.8		
5	2	4	—		Test stopped, large deflections
6	2	3	22.0		
7	4	2	23.0		
8	4	2	23.0		
9	4	2	22.0		
10	10	None		Cyclic, Type 1	
11	2	4	18.5	Monotonic	
12	10	None		Cyclic, Type 1	
13	2	4	15.5	Monotonic	
14	10	None		Cyclic, Type 2	
15	2	3	22.0	Monotonic	
16	10	None		Cyclic, Type 2	
			Block 4 — f_c = 4200 psi, f_t = 410 psi		
1	2	3		Cyclic, Type 1	
2	2	3		Cyclic, Type 1	
3	4	2		Cyclic, Type 1	
4	2	3		Cyclic, Type 1	
5	4	1		Cyclic, Type 1	
6	2	3		Cyclic, Type 2	
7	4	2		Cyclic, Type 2	
8	4	1		Cyclic, Type 2	
9	4	2		Cyclic, Type 1	
10	2	3		Cyclic, Type 2	
11	4	1		Cyclic, Type 2	
12	2	3		Cyclic, Type 2	
13	4	2		Cyclic, Type 2	
14	2	3		Cyclic, Type 2	
15	4	1		Cyclic, Type 1	
16	2	3		Cyclic, Type 2	

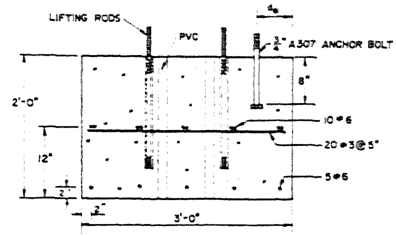

Fig. 3 Cross section of typical specimen

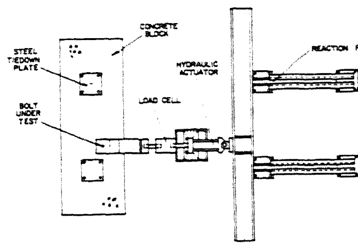

Fig. 4 Test setup

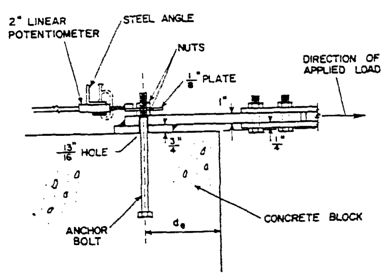

Fig. 5 Instrumentation for measuring bolt deflections

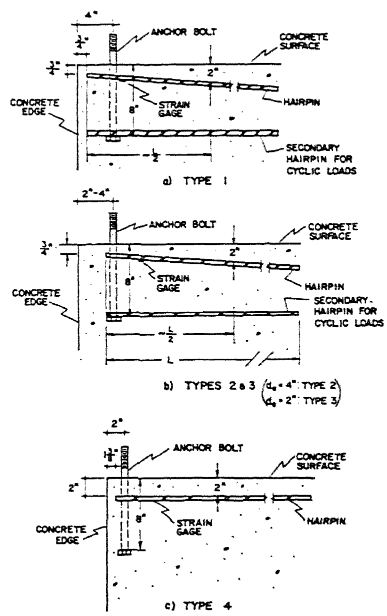

Fig. 6 Hairpin reinforcement

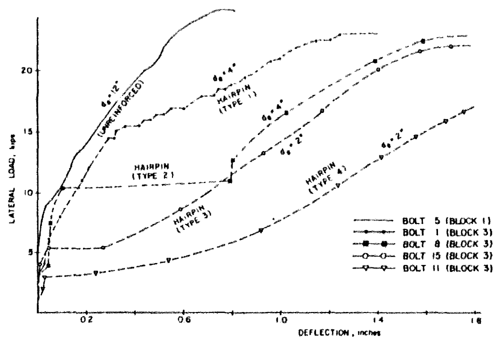

Fig. 7 Typical load-deflection curves, monotonic shear

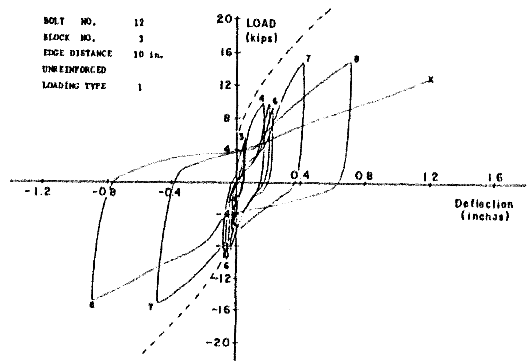

Fig. 8 Typical load-deflection plot, no reinforcement, large edge distance

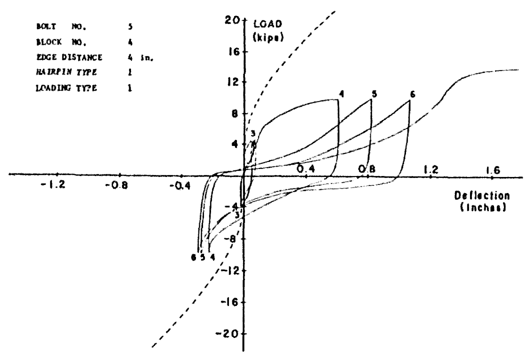

Fig. 9 Typical load-deflection plot, Type 1 hairpin, small edge distance

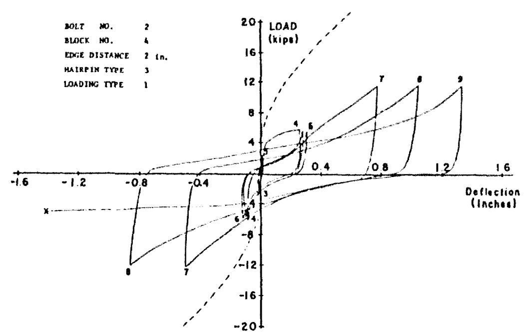

Fig. 10 Typical load-deflection plot, Type 3 hairpin, small edge distance