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Effect of Reinforcing Details on the Shear Resistance of Anchor Bolts Under Reversed Cyclic Loading

by R. E. Klingner, J. A. Mendonca, and J. B. Malik

Following an extensive literature study, a three-phase experimental investigation was conducted using single A307 anchor bolts embedded in normal weight concrete and loaded quasi-statically in monotonic and reversed cyclic shear. All bolts were embedded sufficiently to develop the minimum specified tensile strength of the anchor steel. Based on the results of this and other investigations, expressions are presented for calculating the minimum critical edge distance at which an anchor embedded in plain concrete can develop its full strength in shear. Using 180 deg hairpins, reinforcing details were developed allowing an anchor to develop its full shear strength even when placed at less than that minimum critical edge distance. These details were found to be satisfactory under reversed cyclic as well as monotonic shear. Procedures are presented for the design of anchor bolts and studs under monotonic and reversed cyclic shear loads.

Keywords: anchor bolts; cyclic loads; earthquake resistant structures; embedment; failure; loads (forces); plain concrete; reinforced concrete; reinforcing steels; **shear strength; shear tests;** structural design; studs; tensile strength.

Anchor bolts embedded in concrete are a common element in many types of construction. Such anchor bolts must often transmit combinations of tensile force, shear force, and bending moment to the concrete. Anchor bolts are now also used in more critical applications, often involving resistance to reversed cyclic loads. 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 three phases, whose specific objectives are given below:

Phase I

To determine the effect of edge distance on the shear resistance of ¾-in. A307 bolts embedded in normal weight unreinforced concrete.

Phase II

To develop reinforcing details allowing an anchor bolt to develop its full strength in shear even when placed at an edge distance small enough to cause reduced capacity in the bolts tested in Phase I.

Phase III

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.

LITERATURE REVIEW

For small embedment lengths, an anchor bolt loaded in shear will fail by pulling out of the concrete, leaving a cone-shaped hole.[1,2] This failure mode is very similar

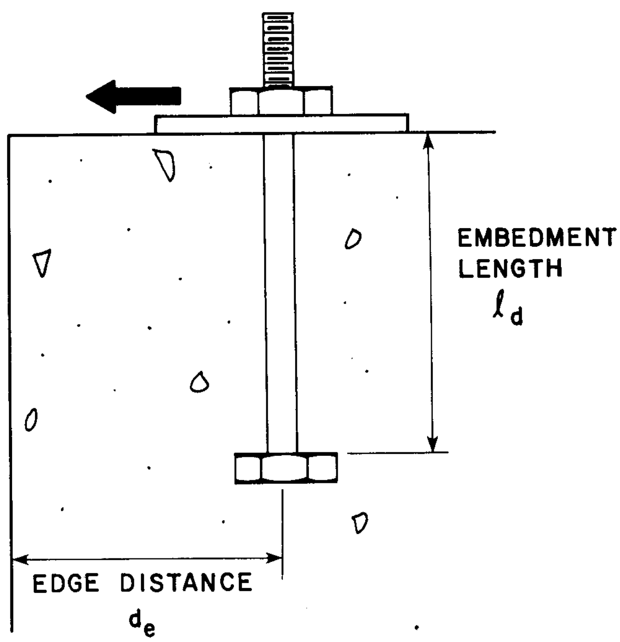

Fig. 1 — Typical anchor bolt loaded in shear

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*ACI member **R. E. Klingner** is an assistant professor of civil engineering at The University of Texas at Austin, Austin, Tex. He is currently the secretary of ACI-ASCE Committee 442, Response of Buildings to Lateral Forces.*

***J. A. Mendonca** is working with a consulting engineering firm in Salvador, Bahia, Brazil. He completed all coursework requirements for a MS in civil engineering at The University of Texas at Austin in June 1980.*

***J. B. Malik** is an engineer with Datum Engineering, Inc., Dallas, Tex. He obtained his MS in civil engineering at The University of Texas at Austin in June 1980.*

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.[3-8] While a discussion of those is beyond the scope of this paper, the authors studied all of them and recommend those of References 7 and 8 as being rational and in satisfactory agreement with available test data. All shear tests discussed herein had sufficient embedment to develop the minimum specified tensile strength of the anchor steel, and therefore to prevent this type of failure.

As summarized in Table 1, seven different design procedures are available in the U.S. for designing anchor bolts and welded studs loaded in shear.[1,3,4,5,7,8,9] While space limitations prevent a complete discussion of these, basic points can be presented. Failure can occur either by shearing failure of the anchor itself or by failure of the concrete. Some procedures [1,3,4,5] calculate the steel shearing resistance as the anchor's cross-sectional area times the ultimate tensile strength of the steel. Another procedure[8] calculates the steel shearing resistance using the shear-friction concept, whose theoretical basis depends on having a loading plate-concrete interface rough enough to mobilize tensile clamping force in the anchor under small shearing displacement. This shear-friction mechanism was not observed in the authors' tests, in which the loading plate either rotated away from the concrete or slid easily on it.

The case of concrete failure is handled in several ways. First, consider an anchor located far from a free edge. Three procedures[7,8,9] assume that concrete failure is precluded if sufficient tensile embedment is provided as discussed at the beginning of this review, and if sufficient edge distance is provided as discussed later in this section. Based on the results of research conducted by the authors and others, this approach seems satisfactory and rational. Other procedures[3,4] calculate concrete resistance based on the shear-friction hypothesis. As discussed above, the validity of this approach depends on actually being able to mobilize a tensile clamping force across the concrete-loading plate interface. Also, this mechanism seems more closely related to tensile yielding of the bolt rather than failure of the concrete. The remaining procedures[1,5] calculate concrete resistance using empirical formulas based on the experimental results of Reference 1. While these formulas may give satisfactory agreement with the results of those tests, the authors believe that the approach of References 7, 8, and 9 is more rational, insofar as it is based on experimentally observed failure modes.

For anchors close to a free edge, three procedures[7,8,9] idealize the concrete failure surface as a semicone

Table 1 — Summary of procedures for calculating nominal shear resistance*

Reference	Steel failure	Concrete failure far from edge/close to edge	Comments
1	$V_s = A_s f_{ut}$	$V_c = 1.106\ A_s f_c'^{0.3} E_c^{0.44}$	f_c', E_c in ksi units $E_c = 57\ \sqrt{f_c'}$, psi
		$V_c' = V_c \left(\frac{d_e - 1}{8D} \right)$	Short edge distance
3	$V_s = A_s f_{ut}$	$V_c = A_s (0.9\ f_{ut})$	
		$V_c = (2500\ d_e - 3500)$	Short edge distance
4	$V_s = A_s f_{ut}$	$V_c = A_s (0.9\ f_{ut})$	
		$V_c = 3250\ (d_e - 1)\ \frac{\sqrt{f_c'}}{\sqrt{5000}}$	Short edge distance
5	$V_s = A_s f_{ut}$	$V_c = 6.66 \times 10^{-3}\ A_s f_c'^{0.3}\ E_c^{0.44}$	f_c', E_c in psi units $E_c = 57{,}000\ \sqrt{f_c'}$
		$V_c' = V_c \left(\frac{d_e - 1}{8D} \right)$	Short edge distance
7	$V_s = \frac{A_s f_y}{C}$	$V_c = 2\pi\ d_e^2\ \sqrt{f_c'}$	$C = 1.0$ to 1.5
8	$V_s = A_s (0.9\ f_y)$	$V_c = 2\pi\ d_e^2\ \sqrt{f_c'}$	Short edge distance only
9		$V_c = 2\pi \left[\frac{d_e + D/2}{\tan\alpha} \right]^2 \sqrt{f_c'}$ $\alpha = (d_e + D/2)\ 4 + 25 \text{ deg} \leq 45 \text{ deg}$	Short edge distance only

*Refer to Fig. 1 and Nomenclature for definition of terms.

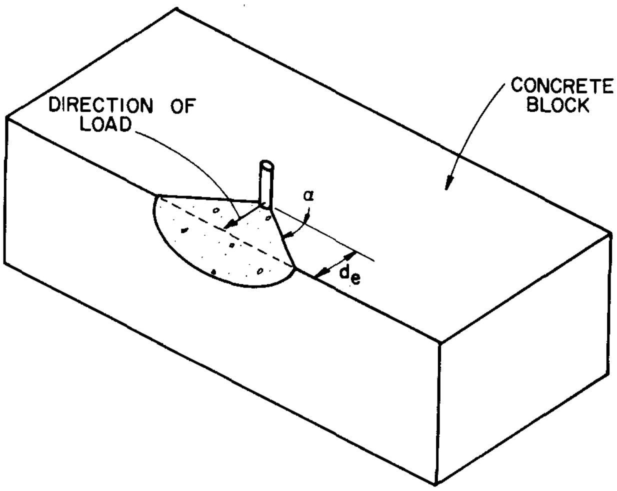

Fig. 2 — Idealized semiconical concrete failure surface

whose height is approximately equal to the edge distance and whose sides have an inclination α with respect to the face of the block (Fig. 2). One procedure[9] uses a variable α, while the other two take α equal to 45 deg. The concrete failure load is computed as the shear force resultant of stresses equal in magnitude to the concrete tensile strength (conservatively taken as $4\sqrt{f_c'}$), and acting perpendicular to this semiconical failure surface. The authors believe this procedure to be rational and in satisfactory agreement with test data. The other procedures[1,3,4,5] calculate the concrete shear resistance using either empirical formulas[3,4] or an empirical reduction of the concrete strengths for an anchor located far from an edge[1,5]. While these formulas may give satisfactory agreement with the test results on which they were based, the authors believe that the approach of References 7, 8, and 9 is more rational.

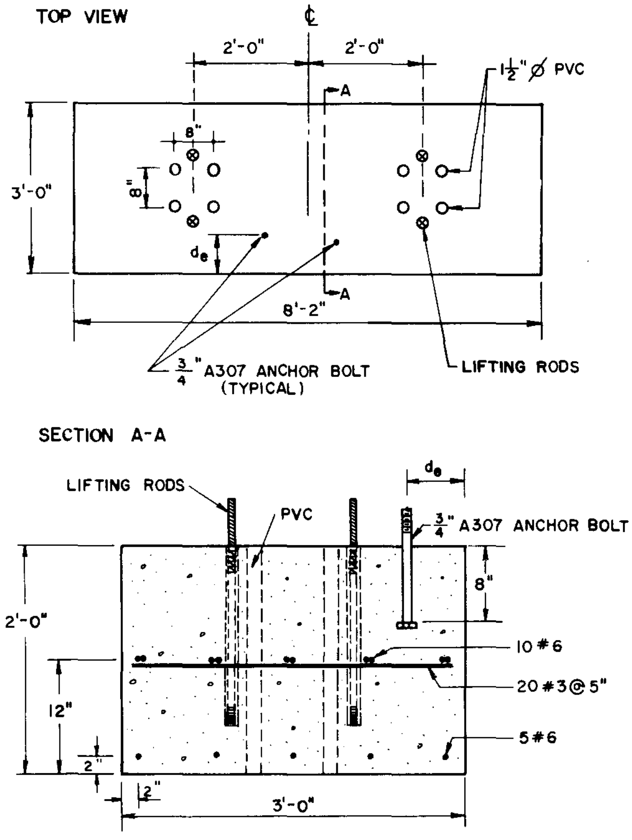

Fig. 3 — Cross section of typical specimen

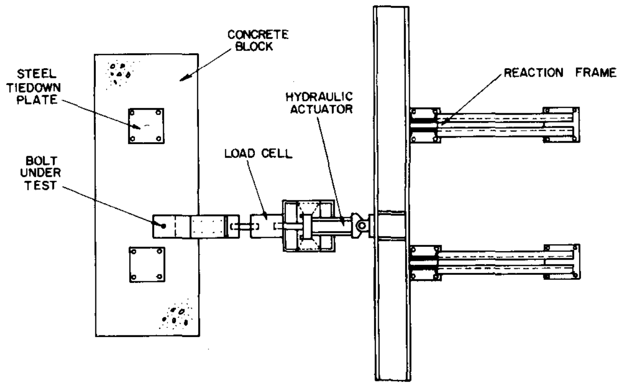

Fig. 4 — Test setup

EXPERIMENTAL INVESTIGATION

Test specimens

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 2 and will be described in detail in the sections dealing with each phase of this investigation.

Materials

All blocks were cast using commercial ready-mixed concrete with Type 1 cement and 5/8-in. aggregate. After stripping, the blocks were air cured until testing. Standard 6 x 12 in. test cylinders were cured next to the blocks and were tested in compression at 7 days, 28 days, and before and after each block was tested. Splitting tensile tests were also performed on the cylinders after each block was tested. Table 2 gives average concrete strengths at the time each block was tested.

All anchor bolts conformed to ASTM A307 and were hexagonal-headed, with a nominal diameter of ¾ 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 61.3 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,

Table 2 — 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	

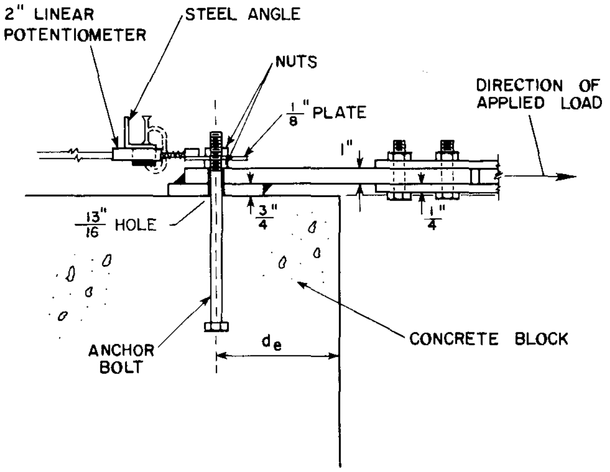

Fig. 5 — Instrumentation for measuring bolt deflections

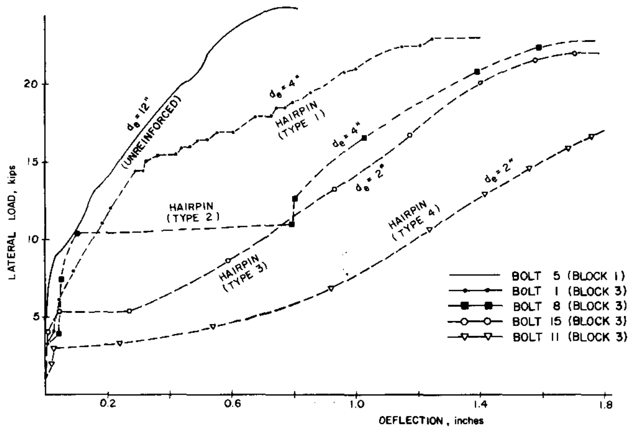

Fig. 6 — Typical load-deflection curves, monotonic shear

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, one end of which was clamped to a steel angle attached to the block, and the other end of which rested against the nuts used to tighten the anchor bolts. The strain gages attached to one or both hairpin legs (Fig. 8) were used primarily to detect possible yielding of the hairpins, although nominal hairpin forces were also computed.

PHASE I

Effect of surface condition and loading plate size

Phase I comprised a total of 24 tests, eight of which were conducted on Block 1, and the remaining 16 of which were conducted on Block 2. As shown in Table 2, Block 1 contained eight bolts, all tested in monotonic shear and placed at an edge distance of 12 in., judged sufficient to insure failure by yielding of the bolt rather than failure of the concrete. All bolts were tightened snug tight with a hand wrench prior to testing. The bolts were tested using loading plates measuring 6 x 6 in. and also 12 x 12 in. Three different surface conditions were used between the concrete and the loading plate: normal (steel-troweled, no curing compound); a thin, sand-cement mortar coating; and a Teflon sheet. Fig. 6 shows a typical load-deflection result for this block, obtained using a 6 x 6 in. loading plate with normal surface preparation. All bolts failed by shear in the steel at the interface between the block and the loading plate. For any given surface preparation, the failure load was about the same for the 6-in. and 12-in. plates. In each test, some local crushing of the concrete occurred prior to failure, in front of the bolt in the direction of loading. Because the loading plate covered the affected region, the load level at which this crushing began was not observed. The failure loads for the mortar and Teflon surfaces were about 5 and 10 percent less, respectively, than those for normal surface preparation. The effects of loading plate size and surface preparation were therefore concluded to be predictable and comparatively minor, and all subsequent tests were carried out using 6 x 6 in. loading plates and normal surface preparation.

Bolts under monotonic shear load in plain concrete

As shown in Table 2, the rest of Phase I comprised tests on 16 bolts placed at edge distances of 2, 4, 6, and 8 in. and tested in monotonic shear. These four edge distances were selected as being small enough to cause concrete rather than bolt failure under shear loads.

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. Several of the bolt resistances were adversely affected by concrete damage from previous tests on adjacent bolts, and two of the 16 bolts in Block 2 were not tested for that reason.

Effect of edge distance on bolt shear resistance

The experimental results of Blocks 1 and 2 are plotted in Fig. 7. Bolts located at edge distances less than some critical edge distance d_{cr} failed due to concrete cracking, while bolts located at edge distances greater than that critical value failed due to bolt shear. As noted in Table 2, seven of the 14 tests in which concrete failure was observed had some degree of preexisting damage. While not sufficient to change the failure mode, this damage probably did reduce the concrete failure resistance, so the affected tests are therefore differentiated in Fig. 7.

Strength as governed by concrete failure

The experimental results of Phase II were compared with the various expressions predicting strength as gov-

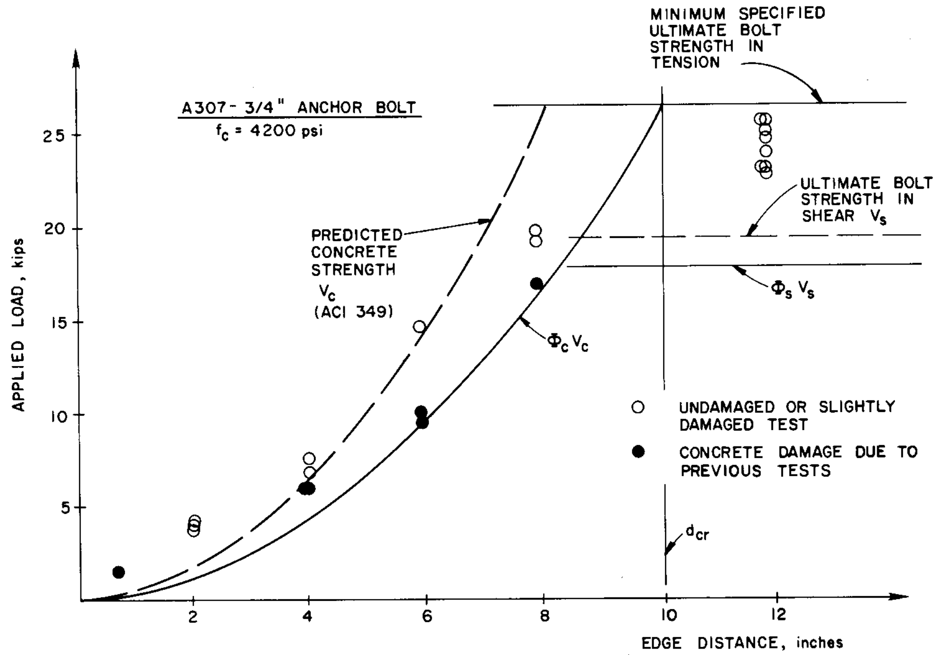

Fig. 7 — Monotonic shear strength of unreinforced bolts

erned by concrete failure. The most acceptable agreement was given by the equation of ACI Committee 349,[8] which is also plotted in Fig. 7. While this equation is unconservative with respect to two of the authors' tests with 6 in. edge distance, those tests involved some previous damage. While it is also unconservative for both tests with 8 in. edge distance and no previous damage, the design strength of anchors with such large edge distance will usually be governed by steel capacity. The ACI 349[8] equation for concrete resistance is

$$V_c = 2\pi\, d_e^2 \sqrt{f_c'} \qquad (1)$$

where d_e = edge distance in in., and
f_c' = concrete compressive strength, psi

To use this expression as part of a strength design procedure, an understrength factor of $\phi_c = 0.65$ was chosen. This value, while more conservative than the value of 0.85 currently used for diagonal tension, is believed justified by the high degree of scatter observed in the concrete failure data, and also by the fact that concrete failure around anchor bolts in plain concrete results in immediate, brittle anchorage failure.

Strength as governed by steel failure

Fig. 7 also shows the experimental results for the eight specimens whose strengths were governed by steel failure. All strengths lie above the ultimate bolt strength in simple shear and below the specified minimum ultimate tensile strength. Consistent with this observation, the authors suggest that if threads are excluded from the shear plane, bolt shear resistance as governed by steel failure should be taken as the nominal cross-sectional area of the anchor times the ultimate shear strength of the anchor.

In the absence of actual test data, the ultimate shear strength is usually unavailable to a designer. Based on the results of this and other investigations,[10] the authors suggest that the ultimate shear strength be taken as 0.75 times the minimum specified ultimate tensile strength of the anchor. For design purposes, the shear strength of anchorages as governed by steel failure can then be computed as

$$\phi_s V_s = \phi_s A_s (0.75\, f_{ut}) \qquad (2)$$

where ϕ_s = 0.90, and
f_{ut} = specified minimum ultimate tensile strength of anchor

These expressions are also plotted in Fig. 7.

Calculation of critical edge distance

As previously discussed, the critical edge distance d_{cr} is the smallest edge distance at which a bolt in plain concrete will fail by shear of the bolt rather than failure of the concrete. This critical edge distance can conservatively be estimated by solving for that edge distance at which the calculated concrete resistance [Eq. (1)], reduced by a ϕ-factor of 0.65, will just equal the maximum possible steel resistance.

As noted previously, the maximum possible steel resistance exceeds the nominal bolt area times the specified minimum ultimate shear strength, but does not exceed the nominal bolt area times the specified minimum ultimate tensile strength. The authors therefore recommend that the maximum possible steel shear resistance be computed as

$$V_{s\,max} = A_s f_{ut} \qquad (3)$$

Then the critical edge distance can conservatively be computed by

$$d_c \geq D \sqrt{\frac{f_{ut}}{8\phi_c \sqrt{f_c'}}} \tag{4}$$

As shown in Fig. 7, this critical edge distance is about 10 in. for the anchors tested in this investigation.

PHASE II

As shown in Table 2, Phase II comprised monotonic shear tests on 16 bolts in Block 3, placed at short edge distances of 2 and 4 in. These bolts were placed in concrete reinforced with 180 deg hairpins so their shear resistances would equal those of bolts in plain concrete at large edge distances. As shown in Fig. 8, four types of hairpins were used. Hairpin Type 1, used with bolts are 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.[12] All hairpins had an inside bend diameter of 3.75 in., corresponding to six bar diameters as specified in Reference 11.

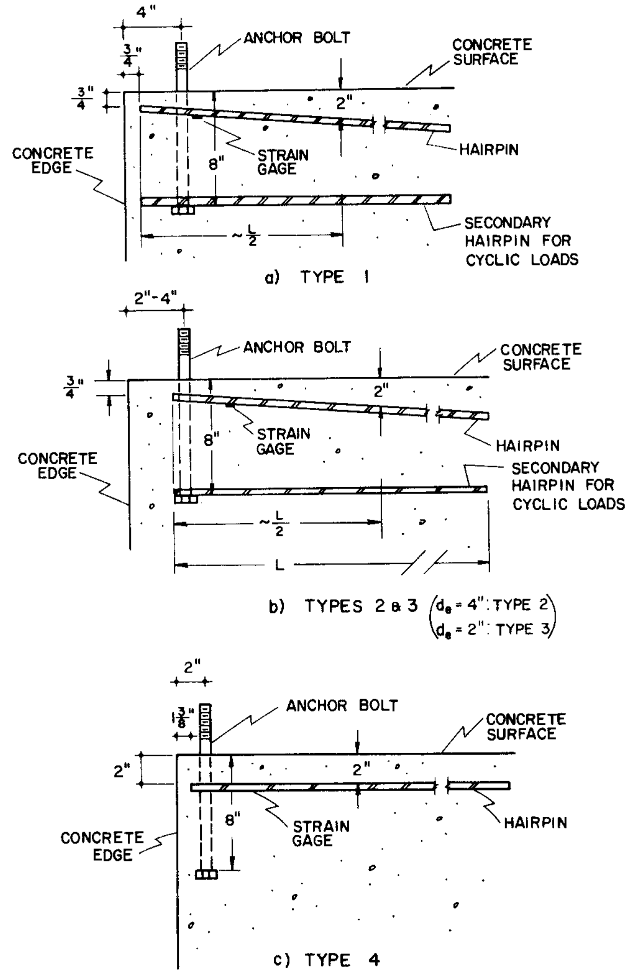

Fig. 8 — Hairpin reinforcement

Bolts under monotonic shear load in reinforced concrete

Fig. 6 shows typical load-deflection results for bolts at small edge distances in reinforced concrete. All bolts behaved similarly until spalling occurred in front of the hairpin. In all bases, the spalling load could be conservatively estimated using the expression for $\phi_c V_c$ developed during Phase I. Based on observations of three tests only, this spalling load was about 40 percent higher for Type 1 hairpins than for the other types, probably due to the more uniform distribution of tensile stress in the Type 1 hairpin, and also to the favorable state of biaxial compression in the concrete between that type of hairpin and the bolt. With the exception of hairpin Type 1, the presence of hairpin reinforcement had little effect on spalling loads. While hairpin Types 2-4 take some of the load normally carried by the concrete prior to spalling, they also create stress concentrations that increase the tendency for spalling.

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. Bolts in concrete reinforced with Type 4 hairpins showed large deflections at each load level compared to bolts reinforced with other types of hairpins. This relatively low stiffness was due to the large distance between the Type 4 hairpins and the top of the block and is consistent with the kinking mechanism proposed above. The low stiffness led to unsatisfactory performance, in that the bolts failed to reach their capacity even at large deflections. As a result of these tests, 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.

While hairpin reinforcement was effective in maintaining ultimate shear capacity, Fig. 6 shows that bolts placed at small edge distances in reinforced concrete reached their ultimate strengths at deflections about twice those corresponding to bolts at large edge distances in plain concrete. Although deflection under ultimate loads is generally a less critical design criterion than are service load deflections, it is possible that in

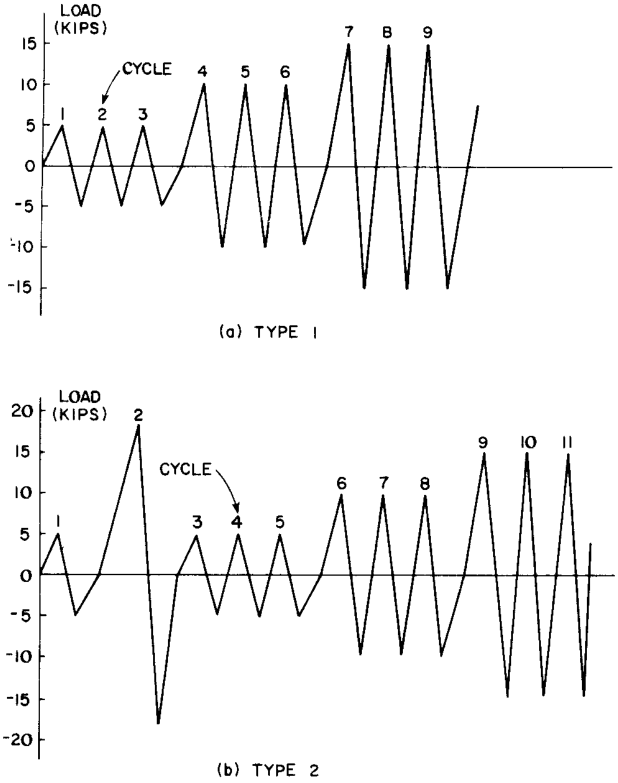

Fig. 9 — Cyclic loading programs

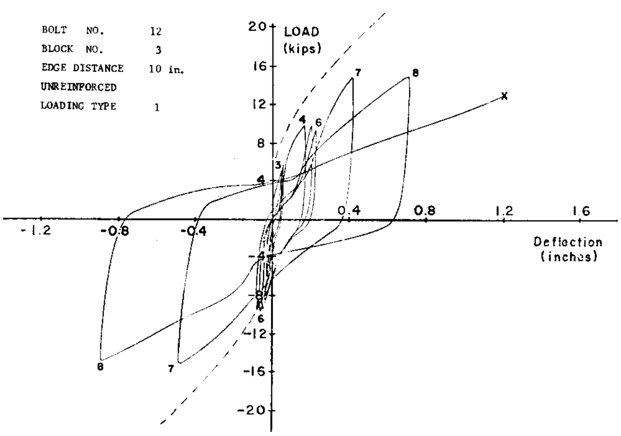

Fig. 10 — Typical load deflection plot, unreinforced bolts at large edge distances

some cases the useful capacity of a bolt could be limited by deflection considerations.

PHASE III

Phase III comprised reversed, quasi-static, cyclic load tests on bolts in plain as well as reinforced concrete. As shown in Table 2, Block 3 contained four bolts in plain concrete at 10 in. edge distances, greater than the critical edge distance as discussed previously. Block 4 contained 16 bolts at edge distances of 2 and 4 in., all in concrete reinforced with hairpin Types 1, 2, or 3. Because of its relatively poor performance in monotonic load tests, hairpin Type 4 was not used in the cyclic load tests. As shown in Fig. 8, all such cyclically loaded bolts were provided with two hairpins, one for resisting shear in each direction. In the case of a bolt subjected to shear loads directed away from the free edge, the lower hairpin was expected to develop tension as the bolt rotated about a fulcrum near its upper end. As anticipated, tensile forces in the lower hairpins were observed to be small. However, the lower hairpin sometimes prevented outward or upward movement of the bolt head following local bursting failure. The authors therefore believe that even though it may be lightly stressed, the lower hairpin is beneficial for bolts placed at edge distances small enough to lead to bursting [see Reference 8, B5.1.1(a)].

All bolts in Phase III, therefore, either were located at sufficiently large edge distances or provided with sufficient reinforcement to prevent concrete failure. As shown in Fig. 9, two types of cyclic loading program were used. 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

Fig. 10 shows typical load-deflection results for bolts at large 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 behaved 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.

While no spalling of concrete was noted during these tests, the concrete did crush around the top of the bolts. This localized crushing was similar to that observed in the monotonic tests but now occurred both in front of and behind the bolt. Fig. 10 shows that the unloading path for the bolts differs significantly from the loading path, indicating considerable plastic deformation followed by elastic unloading. The reduction in bolt shear capacity under reversed cyclic loads as contrasted with monotonic shear loads is believed due to the effects of low-cycle fatigue on the bolts themselves.

Bolts under reversed cyclic load in reinforced concrete

Fig. 11 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-kip 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 in-

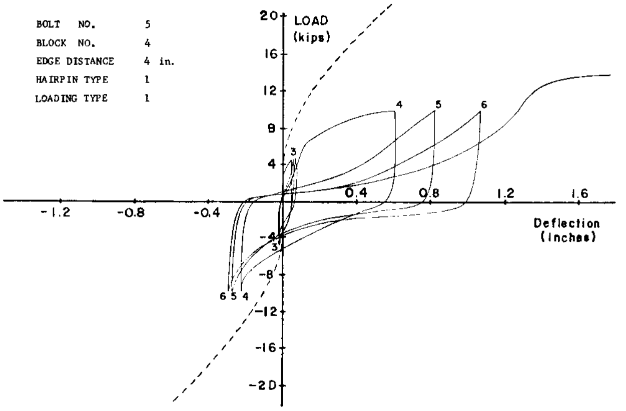

Fig. 11 — Load-deflection plot, reinforced bolt at small edge distance, Type 1 hairpin

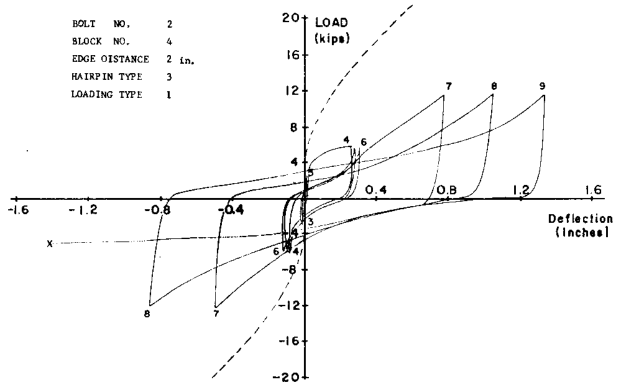

Fig. 12 — Load-deflection plot, reinforced bolt at small edge distance, Type 3 hairpin

creased during each successive cycle to the 10-kip level. In one of these anchorages, the concrete between the bolt and top 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 were observed during the large pulse, and final bolt failure occurred in the steel during one of the three subsequent 10-kip cycles.

Comparison of Fig. 11 and 12 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 exhibited behavior similar to that of bolts with Type 1 hairpin. During the cycles to 10 kips, the concrete in front of the top 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 the test results of Phase III, 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. 8 (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 top 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.

DESIGN PROCEDURES FOR SHORT ANCHOR BOLTS LOADED IN SHEAR

The objectives of these design procedures are to prevent spalling under service loads and to insure ductile failure of the anchor bolt, regardless of edge distance. These design procedures assume that the anchor bolt has been embedded sufficiently to avoid tensile pullout, as discussed in the beginning of the literature review.[7,8] For ¾ in. diameter A307 anchors, for example, the necessary embedment would typically be about 6 or 7 in.

Service level design

(a) Select bolt diameter so load factor times $P_{service} \leq \phi_s V_s$. In other words, the factored service-level shear should not exceed the ultimate bolt shear strength reduced by the steel understrength factor [Eq. (2)].

(b) To prevent spalling under service load, select d_e so that $P_{service} \leq \phi_c V_c$.
In other words, the service-level shear should not exceed the concrete cracking strength reduced by a concrete understrength factor. V_c is calculated by Eq. (1), and ϕ_c is taken as 0.65.

Ultimate strength design

As discussed in (a) above, the anchor bolt should be strong enough to resist the load (if known). If the maximum load cannot be estimated, the edge distance should be large enough so that the concrete cracking strength exceeds the maximum force that can be transmitted by the bolt.

(c) If possible, select d_e so that $\phi_c V_c \geq V_{s\,max}$. In other words, the cracking resistance of concrete reduced by an understrength factor should exceed the ultimate shear strength of the kinked bolt without any understrength factor. $\phi_c V_c$ is calculated as in (b) above, $V_{s\,max}$ is calculated using Eq. (3), and the resulting expression for critical edge distance is given by Eq. (4). If (c) cannot be satisfied, hairpin reinforcement must be provided. As shown in Fig. 8(b), the hairpin should be placed against the bolt shank, as close as possible to the plane on which the shear is applied (this corresponds to hairpin Type 2 or 3 as discussed previously). If the load is a cyclic load, two hairpins should be provided as shown in Fig. 8(b).

(d) Select the hairpin diameter so the minimum specified hairpin yield resistance reduced by a steel understrength factor exceeds the maximum possible tensile load resisted by a kinked bolt

$$\phi_s A_h f_{yh} \geqslant A_s f_{ut} \tag{5}$$

where A_h = cross-sectional area of two hairpin legs
f_{yh} = specified minimum yield strength of the hairpin steel.

(e) Provide sufficient development length for the hairpin reinforcement.[11,12]

SUMMARY AND CONCLUSIONS

Following an extensive literature study, a three-phase experimental investigation was conducted using single A307 anchor bolts embedded in normal weight concrete and loaded quasi-statically in monotonic and reversed cyclic shear.

1. Expressions were presented for the ultimate shear resistance of anchor bolts as governed by concrete and steel failure [Eq. (1) and (2), respectively]. These expressions correlate well with the observed results of this and other investigations and permit the calculation of the minimum critical edge distance at which a bolt in plain concrete can develop its full strength in shear [Eq. (4)].

2. Using 180 deg hairpins, reinforcing details were developed allowing 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.

3. 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 herein, and are shown in Fig. 8(b).

4. Procedures were presented for the design of anchor bolts under monotonic or reversed cyclic shear loads. Although this investigation comprised tests on A307 bolts, the results are believed to be applicable to bolts and welded studs in general.

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NOMENCLATURE

A_s = gross cross-sectional area of anchor shank (nominal diameter)
A_h = nominal cross-sectional area of two hairpin legs
D = nominal diameter of anchor shank
D_h = nominal diameter of anchor head
d_{cr} = critical edge distance [Eq. (4)]
d_e = edge distance (distance from center of anchor shank to nearest free edge, measured in direction of applied shear, shown in Fig. 1)
f_c' = concrete compressive strength, psi
f_{ut} = specified minimum ultimate tensile strength of anchor
f_{yh} = minimum specified yield strength of hairpin reinforcement
l_d = embedment length (distance from surface of concrete to start of anchor head, shown in Fig. 1)
V_c = shear strength of anchor as governed by concrete failure
V_s = shear strength of anchor as governed by steel failure
α = angle of inclination of semiconical failure surface (Fig. 2)
ϕ_c = capacity reduction factor for concrete in tension, taken as 0.65
ϕ_s = capacity reduction factor for steel in tension or shear, taken as 0.90

SI EQUIVALENTS

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

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