Lateral Capacity of WSDOT Bearing Anchor Bolt Details

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16. ABSTRACT

The objective of this research is to recommend a rational design method for bridge anchor bolt connections used by the Washington State Department of Transportation (WSDOT).

An experimental investigation of the behavior of 3/4 inch diameter and 1-1/2 inch diameter, A449 canister/grout anchor bolts under various loading conditions was conducted. Shear loads were applied to anchor bolts subjected to various eccentricities, and the results obtained were used to develop tension/shear interaction relationships.

Elliptical tension/shear interaction relationships provided the best fit to the test data. However, closed-form solutions using tri-linear tension/shear interactions could be conservatively used by bridge designers at WSDOT. To verify the tension/shear interaction relationships, tests were performed on half-scale multiple anchor bolt connections, consisting of two anchor bolts parallel to the applied shear load. The strengths indicated by the multiple anchor bolt tests were conservative when compared to the results obtained from the testing program.

It has also been determined that the AISC/LRFD (25) tension stress limits for bearing type connections could be used for designing WSDOT anchor bolt connections with A449 1-1/2 inch diameter bolts.

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"Lateral Capacity of WSDOT Bearing Anchor Bolt Details"

LATERAL CAPACITY OF WSDOT BEARING ANCHOR BOLT DETAILS

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SUMMARY

The objective of this research is to recommend a rational design method for bridge anchor bolt connections used by the Washington State Department of Transportation (WSDOT).

An experimental investigation of the behavior of 3/4 inch diameter and 1-1/2 inch diameter, A449 canister/grout anchor bolts under various loading conditions was conducted. Shear loads were applied to anchor bolts subjected to various eccentricities, and the results obtained were used to develop tension/shear interaction relationships.

Elliptical tension/shear interaction relationships provided the best fit to the test data. However, closed-form solutions using tri-linear tension/shear interactions could be conservatively used by bridge designers at WSDOT. To verify the tension/shear interaction relationships, tests were performed on half-scale multiple anchor bolt connections, consisting of two anchor bolts parallel to the applied shear load. The strengths indicated by the multiple anchor bolt tests were conservative when compared to the results obtained from the testing program.

It has also been determined that the AISC/LRFD (25) tension stress limits for bearing type connections could be used for designing WSDOT anchor bolt connections with A449 1-1/2 inch diameter bolts.

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The ultimate load tests in this study were conducted to determine the tension/shear interaction relationships for multiple anchor bolt connections. The results for the ultimate load tests of the half-scale (3/4") and full-scale (1 1/2") indicated that an elliptical tension/shear interaction relationship fits well with test data, agreeing with previous test results for anchor bolt connections (12,13). However, the use of tri-linear tension/shear interaction relationships allows for conservative closed-form solutions.

Test data of the half-scale multiple bolt connection were compared to the predicted strengths using a tri-linear tension/shear interaction, showing conservative results, with the exception of pure shear. At zero eccentricity (pure shear) the strength of the connection was predicted accurately. At four and one-half inch eccentricities, the predicted strength was approximately 50 percent above the actual strength of the connection. For a six inch eccentricity, the predicted strength was 85 to 100 percent greater than the actual strength of the connection obtained during testing.

Comparing the AISC/LRFD (25) tension stress limits to the full-scale anchor bolt test results, the AISC/LRFD limits were found to be conservative.

DESIGN RECOMMENDATIONS

The design recommendations resulting from this study are as follows:

(1) For 1-1/2 inch diameter, A449 canister/grout anchor bolts, tension stress limits for fasteners in bearing type connections given in AISC/LRFD (25) can be used.

(2) When designing anchor bolt connections in earthquake regions such as western Washington State, the coefficient of friction should not be considered in design, due to the negative gravitational forces that may occur.

INTRODUCTION

It is common practice in highway bridge construction to use anchor bolts for connecting the superstructure to the substructure. In bridges influenced by lateral loads such as those produced in an earthquake, the deck superstructure is where inertia forces are mainly generated. However, the design of the deck superstructure of a bridge is generally governed by dead and live load effects, not by seismic loadings. Inertia forces may result in shear and uplift (tension) at the anchor bolt connection which may lead to catastrophic failures.

Despite the significance of anchor bolts loaded in a combination of shear and tension, current Washington State Department of Transportation (WSDOT) specifications (8) do not contain a design method for combined loading conditions. WSDOT's present design practice considers the lateral loads as pure shear applied to the anchor bolts. However, eccentric loading will clearly result in a combination of shear and tension in anchor bolts. A relationship describing the interaction of tension and shear on anchor bolts, based on experimental data, is necessary for the rational design development and application of a method for bridge anchor bolts.

The main objective of this research is to develop a rational design method for the canister/grout anchor bolt systems used by WSDOT (see Fig. 1), subject to combined shear and tension loadings. Sub-objectives of this research are: (1) the evaluation of existing theoretical and experimental data relevant to the research, (2) the experimental evaluation of single and multiple anchor bolts under combined shear and tension loading conditions and (3) the development of a design methodology based on the interaction curves obtained from the experimental data.

BACKGROUND

The increasing use of anchor bolt connections in bridge and building construction has led to a need for better understanding the behavior of anchor bolt details. Consequently, extensive research has been invested in the study of this type of structural details in the last decade. A review of this research is presented, including techniques of casting anchor bolts in concrete, followed by discussions of existing literature on the behavior of grouted anchor bolts, and the capacity of anchor bolt connections subjected to combined loading conditions.

Anchor bolts are cast in concrete using two major methods, namely the cast-in-place and retrofit installation techniques. Cast-in-place (CIP) installation of anchor bolts in concrete consists of two common types, canister/grout and CIP headed anchors while retrofit installation is achieved by several methods (i.e., expansion, undercut and adhesive or grouted anchors). Canister/grout installation of anchor bolts is currently used by WSDOT (8) in bridge structures. Although there has been little research conducted on this type of anchor bolt installation, preliminary conclusions based on tests conducted for the State of California Department of Transportation (Caltrans) by Swirsky, et al. (34) indicated better behavior when compared to CIP headed anchor bolts.

Cast-in-place headed anchors are widely used in anchor bolt installation in concrete. Extensive studies have been performed on CIP headed anchors regarding development length (7), pullout (13), tension (18,20), shear (18,21,36), reversed cyclic loading (22) and combined tension/shear behavior (13,26).

Retrofit installation has also been studied extensively. Collins, Klingner and Polyzois (10) performed a study comparing retrofit anchor bolts to CIP headed anchor bolts. The anchor bolts were subjected to static, fatigue and impact tensile loads. All anchor bolts in the study exhibited ductile behavior up to a maximum impact load corresponding to the anchor steel yield load.

Canister/grout anchor bolts are similar to retrofit anchor bolts in that the cavity around the bolt is filled. The loading characteristics of three types of grout and two sizes of retrofit anchor bolts were investigated by Conrad (11). Combined shear and tension on grouted bases have also been studied by Adihardjo and Soltis (5). The tests indicated the interdependence of grout and anchor bolt in determining the capacity of a grouted base detail subjected to combined shear and tension.

Anchor bolt connections in bridges may be subject to a wide variety of loading conditions (e.g., tension, shear, and axial load effects). There has been extensive research on anchor bolts subject to either tension (12,16,17,18,20) or pure shear (12,18,21,22,27,28,36) loadings. However, research on anchor bolts subject to combined tension and shear was far limited. Furthermore, most of the studies on anchor bolts under combined tension and shear have concentrated on single anchor bolts while little attention has been given to multiple anchors.

Anchor bolt failure mechanisms in combined tension and shear are characterized by yielding and fracture in the threaded portion of the anchor bolt due to tension, kinking and bending. For bolts, studs and bars under combined tension and shear loads, Cannon, Godfrey and Moreadith (9) suggested that the area of the steel required be the sum of that needed for tension and shear.

Cook and Klingner (13) performed studies on CIP headed anchors, adhesive anchors and undercut anchor bolts in order to formulate tension/shear interaction relationships. Tests were conducted to ultimate loads on 18 two-anchor bolt patterns with two anchor bolts perpendicular to the load with an assumed rigid baseplate. The anchor bolts were subjected to various combinations of moment and shear by applying an eccentric load at various eccentricities. It was concluded that an elliptical tension/shear interaction relationship was appropriate for anchor bolts in steel-to-concrete connections, and that a linear tension/shear interaction relationship was conservative.

Cook and Klingner (13) also performed tests on multiple-anchor bolt steel-to-concrete connections. The tests consisted of 13 four-anchor and 12 six-anchor bolt specimens, all with a rigid baseplate. Again the test specimens were subjected to various combinations of moment and shear loads. The results of the ultimate load tests indicated that a design procedure based on limit design theory is appropriate for multiple-anchor bolt connections cast in concrete.

Adihardjo and Soltis (5) conducted tests on anchor bolts to determine the effects of a grout pad on anchor bolt behavior. Their test program indicated the interdependence of grout and anchor bolt strengths in determining the capacity of a grouted base detail subjected to combined shear and tension. Also concluded was the fact that existing interaction equations based on bolts embedded directly in concrete were not applicable to the grouted base conditions when the shear component becomes predominant. The Prestressed Concrete Institute (29) also had developed interaction formulas based either on the concrete capacity or the stud capacity for combined tension and shear loading on headed studs.

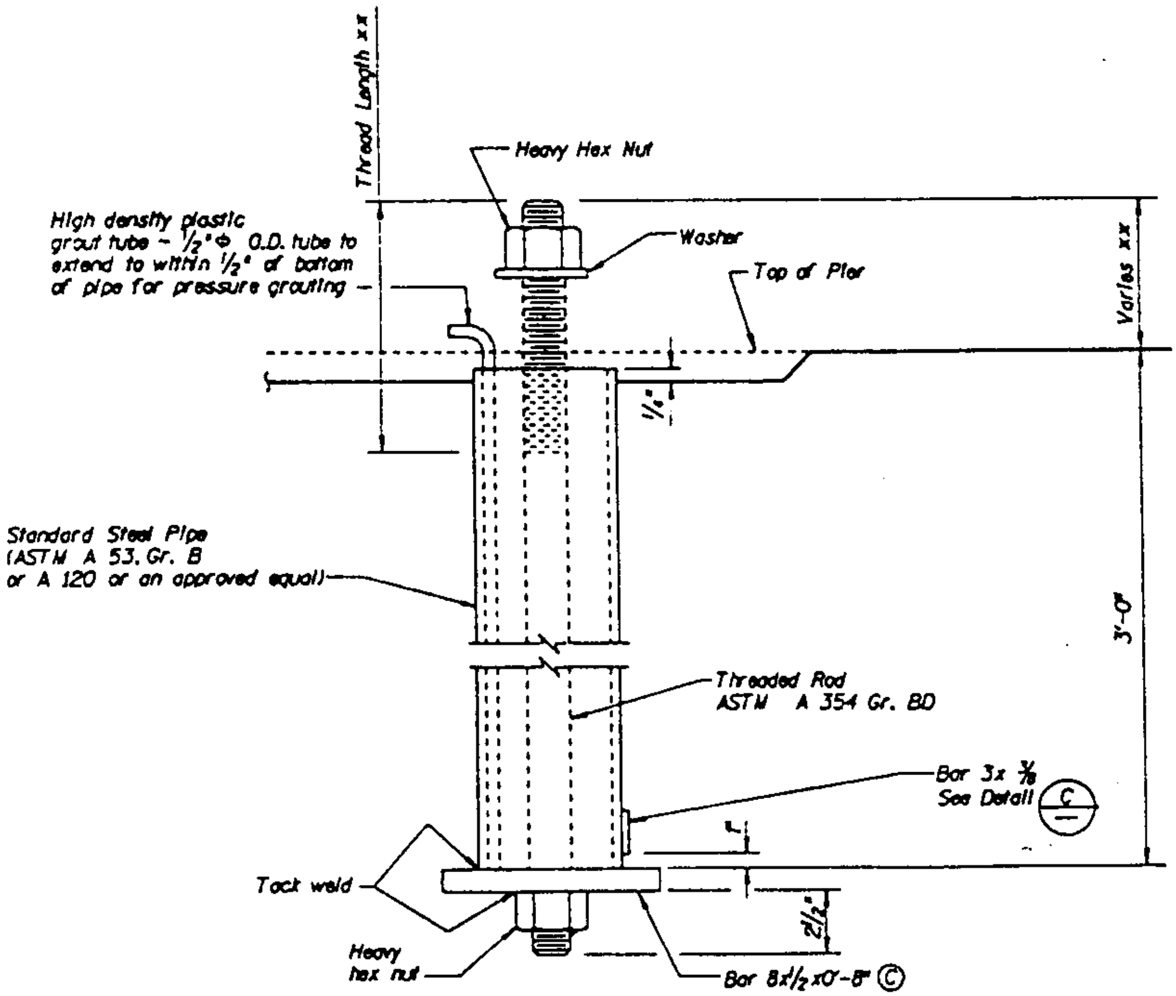


Figure 1 Typical anchor bolt detail currently used by Washington State Department of Transportation (reproduced from (8)).

EXPERIMENTAL PROGRAM

The objective of the experimental program is to determine tension/shear interaction relationships for WSDOT canister/grout anchor bolt connections embedded in concrete. The anchor bolts were cast in concrete as specified by WSDOT Bridge Design Manual [8] using a canister/grout method. Tests were conducted on 10 full-scale (1 1/2 inch diameter), 13 half-scale (3/4 inch diameter) single anchor bolt connections, and 6 half-scale multiple anchor bolt connections (two bolts parallel to the applied shear load).

MATERIAL PROPERTIES

The full-scale (or half-scale) anchor bolts used were 38 (or 19) inches long with 4 (or 2) inches of threads on one end and 6 (or 3) inches of threads on the other. All anchor bolts were fabricated by a commercial bolt manufacturer to meet the requirements of ASTM A449. Heavy hex nuts were used, meeting ASTM A563 Grade C requirements. Hardened steel washers, meeting AASHTO M293 requirements, were used.

The canisters were made of standard black pipe, meeting the requirements of ASTM A53 Grade B/A120. The total length of the full-scale (or half-scale) canisters was 26 (or 13) inches. At one end of the canisters, flat bar stock meeting the requirements of AASHTO M183 was tack welded. For the full-scale (or half-scale) tests, the flat bar was 5 (or 2 1/2) inches square and 1/2 (or 1/4) inch thick. A heavy hex nut meeting the requirements of ASTM A563 Grade C was tack welded to the bottom of the flat bar.

The grout used was a non-shrink, cementitious grout representative of the grout used in canister/grout applications for bridge construction by WSDOT. The grout was used to fill the cavity between the anchor bolt and the canister and to build the grout pad under the base plate. The strength of the grout was obtained using 2 inch by 4 inch cylinders, resulting in an average compressive strength of 5,585 psi, measured at 14 days (33).

The concrete used was a Washington State Department of Transportation Class AX mix, typically used for bridge construction by WSDOT. Four inch by eight inch cylinders were tested after 28 days of curing, and concrete strengths of at least 4000 psi were obtained (33).

The base plates were designed to behave in a rigid manner. For the full-scale (half-scale) single connection, a 3 (1-1/2) inch thick base plate was used with steel meeting ASTM A36 requirements. In order to insure rigid behavior and to allow for eccentric loads, one inch gussets were welded over the full length of the base plate. To prevent deformation of the base plates in the anchor bolt hole, an oversize hole was bored in order to press in a sleeve made of 4140 hardened steel, welded into place to prevent any vertical slip.

FABRICATION OF THE SPECIMENS

The forms for the footings were constructed using high density, plastic-coated, 3/4 inch thick plywood. The plywood was fixed to 2 inch by 4 inch (2x4) wood stiffeners by wood screws. A PVC pipe was anchored to the strong back floor in the testing frame. Horizontal reinforcing hoops were tied to the PVC pipe using tie wires, which were also used to tie the transverse and longitudinal reinforcement to the horizontal hoops. Fig. 2(a) shows photographs of reinforcement and canister pipes for a full-scale specimen, while Fig. 2(b) shows a full-scale specimen concrete block before testing.

Two anchor bolts were suspended by a 2x4 wood board across the top of each form, allowing the testing of either two single anchor bolt specimens or one multiple anchor bolt specimens in each footing. A rectangular piece of plywood with 45 degree sides was placed between the canister and the 2x4 to reproduce the grout recess area as specified in WSDOT Bridge Design Manual (8).

A total of three concrete pours were conducted, each following the same procedure. Two days after each pour, the forms were removed and the footings air-cured until test time. After ten to fourteen days of curing, grout forms were built around the recessed area with 45 degree sides. Steel shims were placed on the concrete in the recessed area as specified in the WSDOT Bridge Design Manual (8). Grout was then poured into the cavity between the anchor bolt and canister and flush with the top of the steel shims. Fig. 3 shows the placement of the steel shims in a full-scale specimen before grouting. The forms were removed the following day with the grout air-cured until test time.

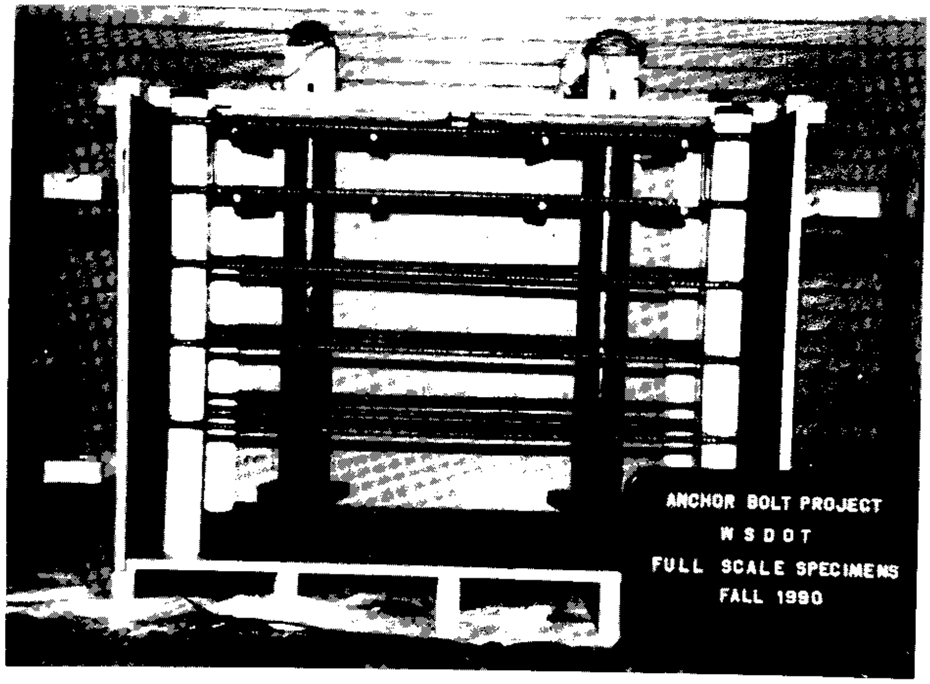
INSTRUMENTATION

The behavior of the specimens was monitored during the tests using linear variable displacement transformers (LVDT) and load cells. Commercially-manufactured load cell washers placed between the anchor nut and base plate (see Fig. 4) were used to measure the tensile forces in the anchor bolts. The applied shear load was measured by a commercially-manufactured 300 kip load cell, placed in the horizontal push arm. LVDT's were used to measure the horizontal and vertical displacement of the anchor bolts as shown in Fig. 5.

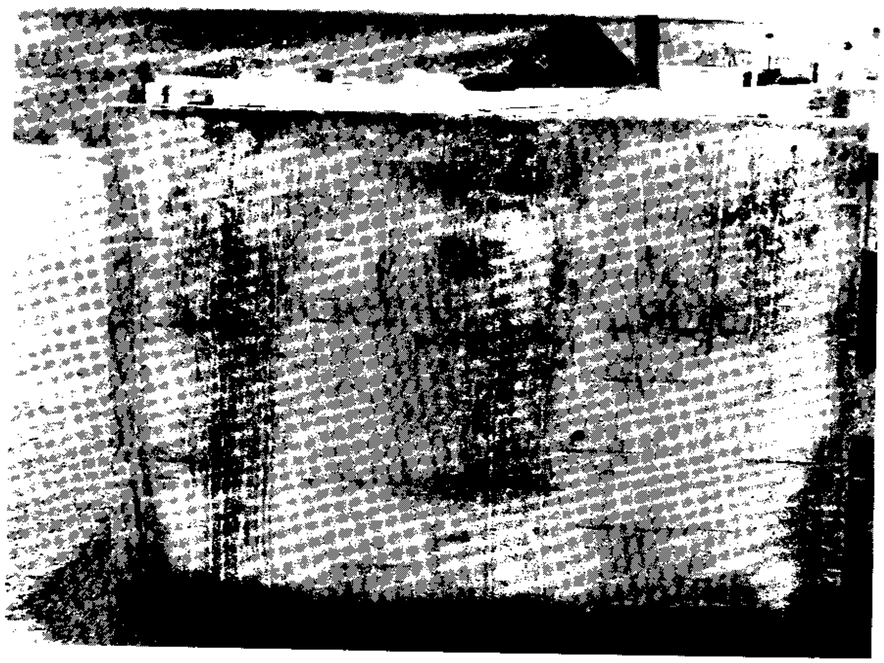
TEST SETUP AND TESTING PROCEDURES

The same test setup and procedures were followed for the half-scale and full-scale single anchor bolt connections and half-scale multiple anchor bolt connections. The anchor bolt footing was first anchored to a laboratory strong floor within an H-frame, then a horizontal stay was then placed opposite to the applied shear load between the footing and wide-flange column of the H-frame to avoid any lateral movement. The test setup for a half-scale multiple anchor bolt connection is shown in Fig. 6.

The horizontal shear load was applied to the base plate by a 200 kip hydraulic ram powered by an electric pump. The horizontal applied shear load was monitored by a load cell mounted to the horizontal push arm between the hydraulic ram and the base plate. The tensile forces in the anchor bolts were monitored by compression load cell washers placed between the base plate and anchor nut. The horizontal displacement of the anchor bolts were monitored by an LVDT fixed to the footing and magnetically connected to the base plate, while the vertical displacement of the base plate was monitored by an LVDT fixed to the base plate. The eccentric shear force was applied in five second increments by a manually triggered pump. The eccentric shear load, the anchor tension and the bolt displacement were recorded on a personal computer immediately after each applied shear load increment.



a) reinforcement



b) uncracked specimen

Figure 2 Photographs of reinforcement (a), and an uncracked specimen (b), for a full-scale single anchor bolt connection.

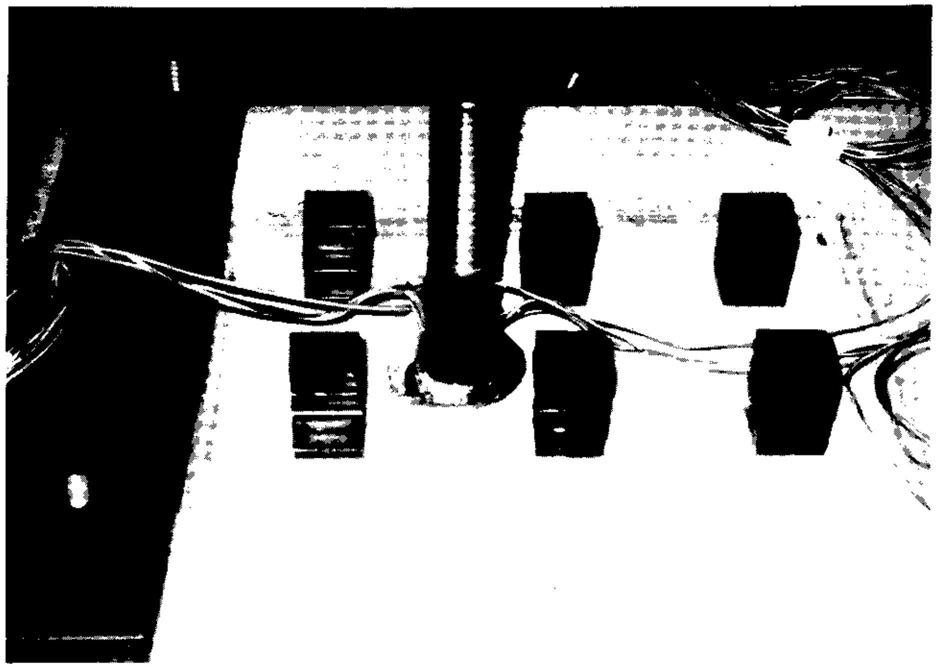


Figure 3 Photograph of steel shims in place on a full-scale single anchor bolt specimen before grouting.

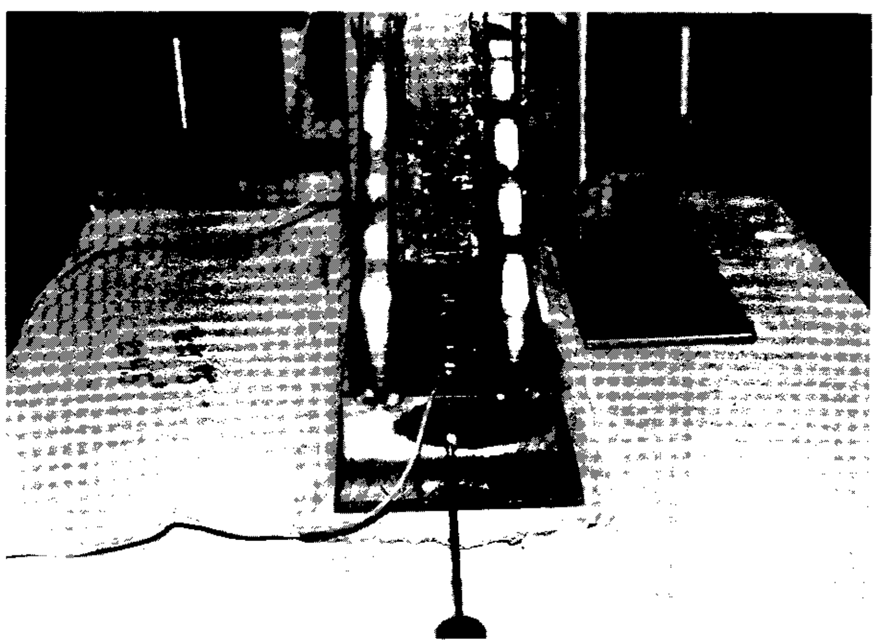


Figure 4 Photograph of a load cell washer positioned on a half-scale multiple anchor bolt connection.

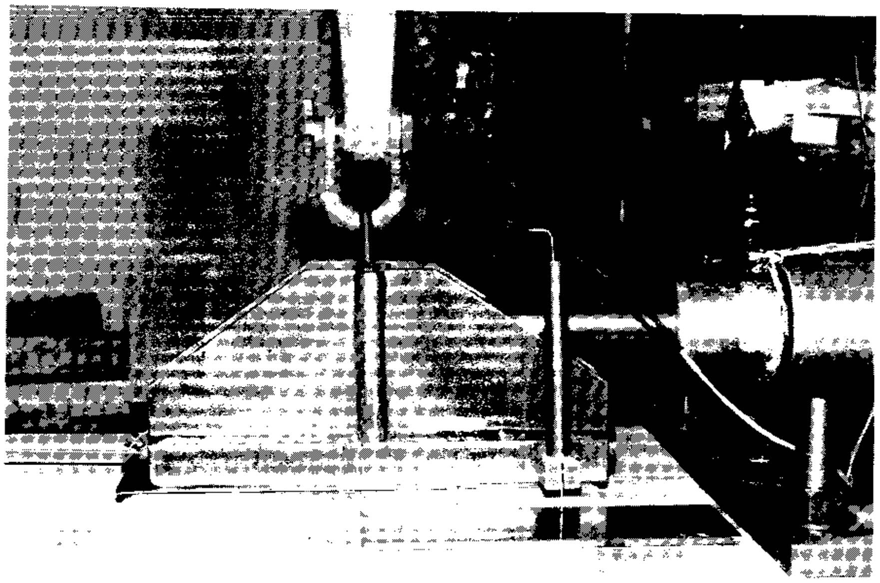


Figure 5 Photograph of the LVDTs in place on half-scale specimens for a multiple bolt connection.

Figure 6 Photograph of test setup for a half-scale multiple anchor bolt connection.

DISCUSSION OF TEST RESULTS

Results and tension/shear interaction curves obtained from testing thirteen single half-scale and ten full-scale anchor bolts are presented and compared. Results from testing six multiple half-scale anchor bolt connections are compared to those of the half-scale single anchor bolts.

FRICTION TESTS

During preliminary full-scale testing, the base plate and the steel shims both caused severe crushing of the grout pad. In order to control both friction and crushing of the grout pad, a thin steel greased plate was placed between the base plate and the grout pad. Friction tests were performed in order to determine the coefficient of friction between the base plate and the thin steel greased plate by applying a known external compressive load to the base plate and then pushing on it until slip occurred. Based on these tests, a coefficient of friction μ of 0.18 was determined. More detailed results of three friction tests may be found in Ref. (33). Using the coefficient of friction above μ, the sum of the anchor bolt tension forces T, and the applied shear load V, the shear load carried by the anchor bolts V_B is determined as:

$$V_B = V - \mu T \tag{1}$$

where: $\mu T \leq V$

ULTIMATE LOAD TESTS

All tests were conducted to their ultimate load where failure of the anchor bolts occurred by yielding and fracture. In all but the preliminary test, the strength of the connection was controlled by the strength of the anchor bolts. The applied shear load and anchor bolt tensile loads were measured directly by load cells. Load-vs-displacement anchor bolt diagrams are presented in Figs. 9 and 10. The dashed line in these diagrams represents the tension versus uplift displacement of the anchor bolt, while the solid line represents the shear versus horizontal displacement of the anchor bolt. It is noted from observing ultimate load test results that the anchor bolts underwent significant inelastic deformation before failure. Typical anchor bolt deformation for a full-scale test is shown in Fig. 7, which also shows that shear transfer occurred primarily by bearing. Crushing and radial cracks in the grout pad were observed opposite to the applied load after each ultimate load test (shown in Fig. 8). The crushing and cracking of the grout pad around the anchor bolt was caused by the anchor bolt bearing on the grout pad.

Table 1 shows values of anchor bolt shear loads V_B, anchor bolt tensile forces T, horizontal displacements δ_h, and vertical displacements δ_v, corresponding to the ultimate applied shear loads applied to the full-scale single anchor bolt connections. Figs. 9(a) and 9(b) show typical load-vs- displacement curves for the tests in which failure was dominated by anchor bolt shear and tension, respectively.

Table 2 shows values of anchor bolt shear loads V_B, anchor bolt tensile forces T, horizontal displacements δ_h, and vertical displacements δ_v, corresponding the ultimate applied shear loads applied to the half-scale single anchor bolt connections. Figs. 10(a) and 10(b) show typical load-vs-displacement diagrams for tests in which failure was dominated by anchor bolt shear and tension, respectively.

Table 3 shows values of anchor bolt shear loads V_B, anchor bolts tensile forces T_1, and T_2, horizontal displacements δ_h, and vertical displacements δ_v, corresponding to the ultimate applied shear loads applied to the half-scale multiple anchor bolt connections. The anchor bolt tensile forces shown in the shaded areas in Table 3 correspond to the anchor bolt (s) which failed. Table 3 shows that at zero eccentricity both anchor bolts failed simultaneously. For tests performed at eccentricities of 4.5 and 6 inches, only the anchor bolt in the high tension zone failed.

TENSION/SHEAR INTERACTION RELATIONSHIPS

The purpose of the single anchor bolt tests was to determine the interaction relationship for anchor bolts under combined tension and shear loading. The results of the single anchor bolt tests were used to construct tension/shear interaction curves for the two sizes of anchor bolts studies in this experimental program. The anchor bolt tensile forces were measured directly, while the anchor shear forces were dependent on the coefficient of friction,as described by Eq. 1.

The combined anchor bolt tension and shear forces calculated for the half-scale and full-scale anchor bolt tests are given in Figs. 11(a) and 11(b), respectively. These figures also show the test data compared to an elliptical tension/shear interaction relationship, providing a reasonably conservative fit to the test data. However, to develop a closed-form solution, tri-linear interaction relationships are presented in Figs. 12(a) and 12(b), which provide an even more conservative fit to the test data.

The half-scale single anchor bolt tri-linear interaction relationship shown in Figure 12(a) has limits of:

$$V_{max} \leq 20 \text{ kips} \tag{2}$$

and

$$T_{max} = \begin{cases} -3.0\ V + 72.5 \text{ when } \leq 35 \text{ kips} \\ \text{or} \\ -0.8\ V + 45.0 \text{ when } \geq 35 \text{ kips} \end{cases} \tag{3}$$

where V_{max} is the maximum applied shear load/bolt, T_{max} the maximum applied tensile load bolt, and V being the applied shear load/bolt. Similarly, the full-scale single anchor bolt tri-linear interaction relationship shown in Figure 12(b) has limits of:

$$V_{max} \leq 85 \text{ kips} \tag{4}$$

and

$$T_{max} = \begin{cases} -2.8\ V + 298 \text{ when } \leq 130 \text{ kips} \\ \text{or} \\ -0.7\ V + 170 \text{ when } \geq 130 \text{ kips} \end{cases} \tag{5}$$

As shown in Figures 12(a) and 12(b), a direct comparison between the interaction relationships of the half-scale and full-scale anchor bolts cannot be made. This is due to the different strength characteristics in the material properties of ASTM A449 3/4 and 1-1/2 inch diameter bolts.

COMPARISON BETWEEN TEST RESULTS AND AISC/LRFD FORMULAS

AISC/LRFD formulas for A449 bolts in bearing-type connections are only available for 1-1/2 inch diameter and greater. For 1-1/2 inch diameter A449 anchor bolts, tension stress limits for fasteners in bearing-type connections are given in AISC/LRFD (25) as:

Threads included in the shear plane:

$$0.73F_u - 1.8f_v \leq 0.56F_u \tag{6}$$

Threads excluded in the shear plane:

$$0.73F_u - 1.4f_v \leq 0.56F_u \tag{7}$$

where F_u (ksi) and f_v (ksi) are the specified minimum tensile strength and the computed shear stress of the bolt, respectively, and the above stress limits are based on LRFD resistance reduction factors (i.e., 0.75 in tension, and 0.65 in shear). Using F_u equal to 105 ksi and a full-scale bolt cross sectional area of 1.405 sq. in., the tensile stress limit in Eq. 6 transforms into a tensile force limit as follows:

$$107.69 - 1.8\ V \leq 82.61 \tag{8}$$

The solid line in Fig. 13 shows the comparison of the full-scale test results and the design curve given in AISC/LRFD (25). By removing the resistance factors included in the tension stress limit formula (Eq. 8), following a procedure in Ref. (32), an unfactored AISC/LRFD tensile force limit is derived:

$$137.20 - 1.76\ V \leq 110.64 \tag{9}$$

The dashed line in Fig. 13 represents the failure curve given in Eq. 9 compared to the full-scale test results, which again shows the conservativeness of the LRFD interaction formulation with respect to the test data obtained.

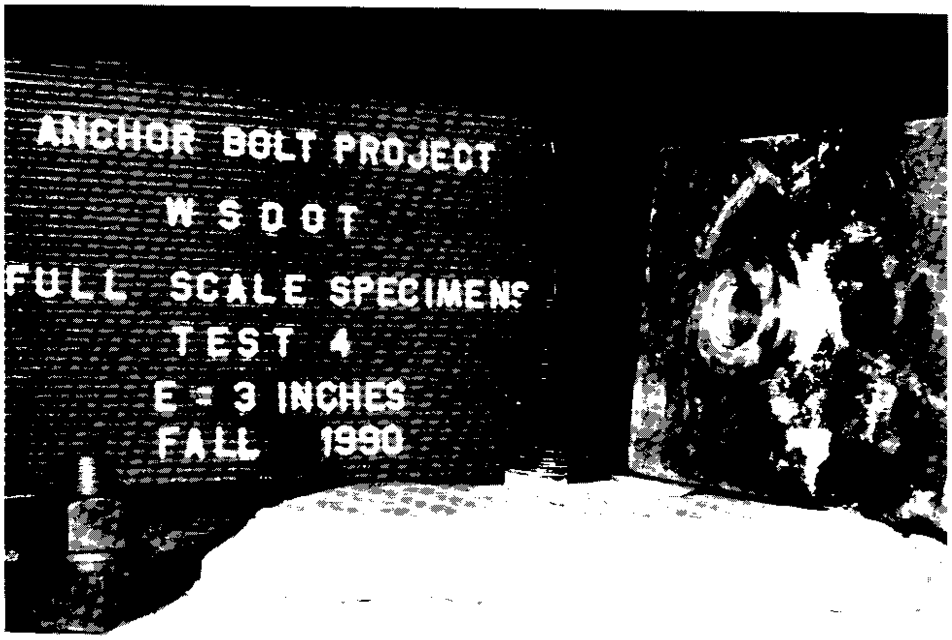


Figure 7 Photograph showing a typical anchor bolt deformation on a full-scale single connection.

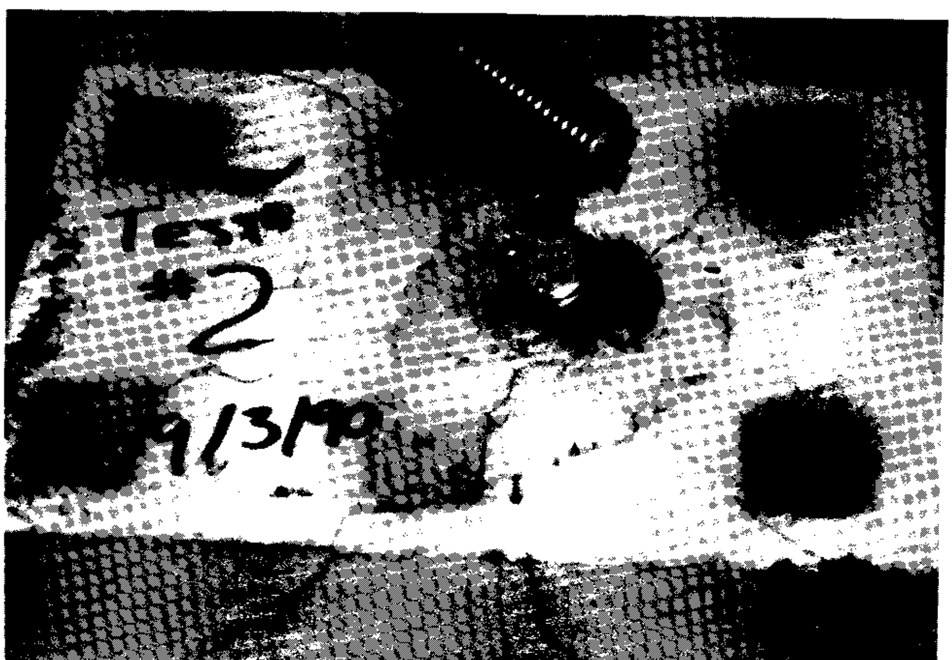
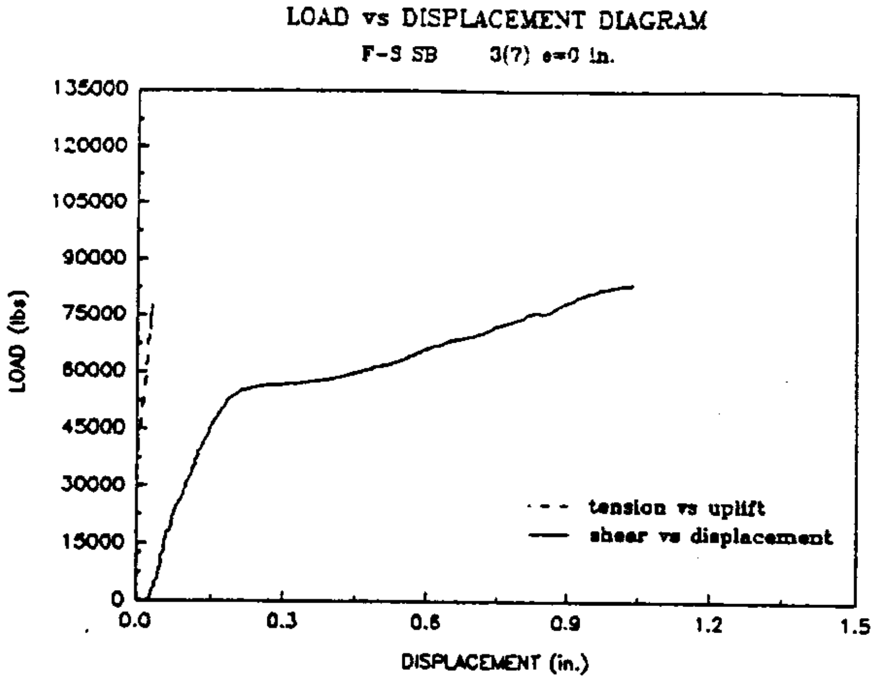
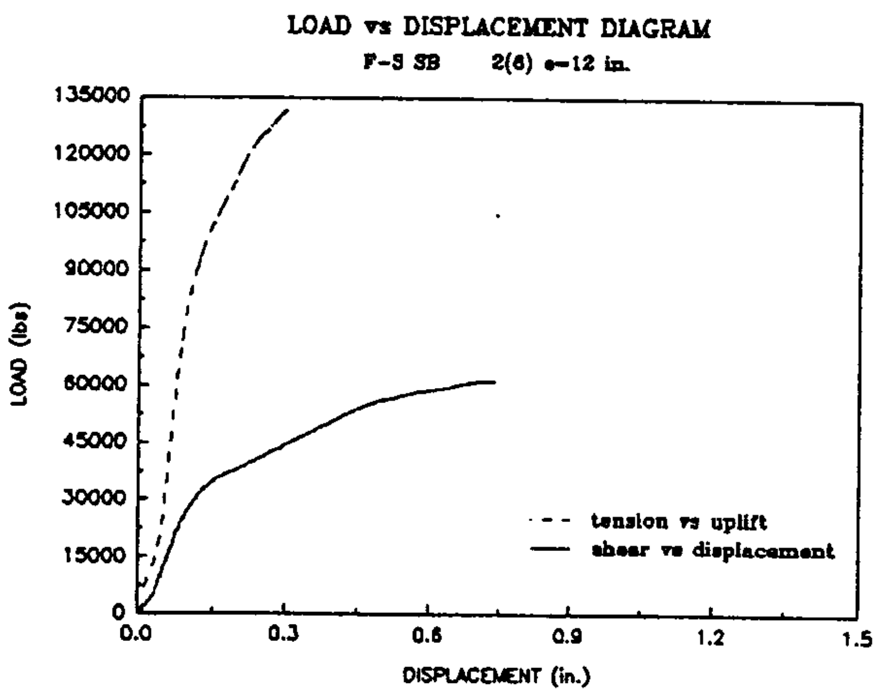


Figure 8 Photograph showing typical crushing and radial cracking on a full-scale connection.

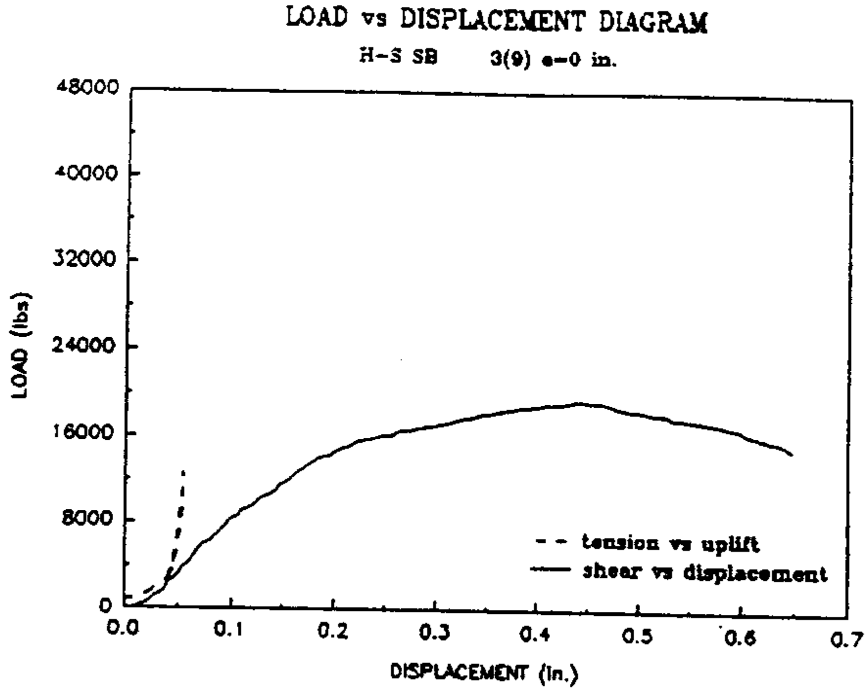


(a) failure dominated by shear

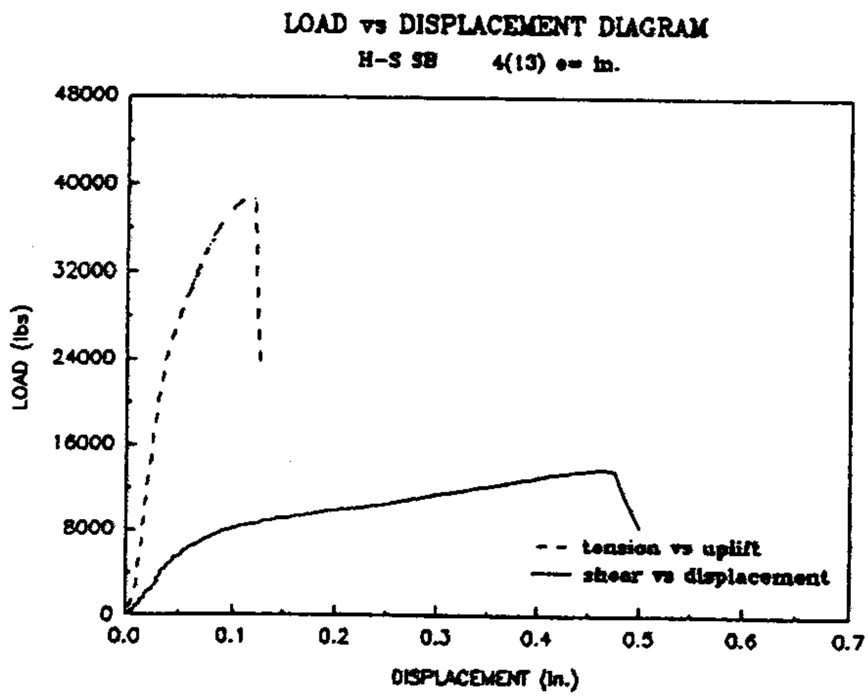


(b) failure dominated by tension

Figure 9 Typical load-vs-displacement for full-scale single anchor bolt: (a) failure dominated by shear, (b) failure dominated by tension.

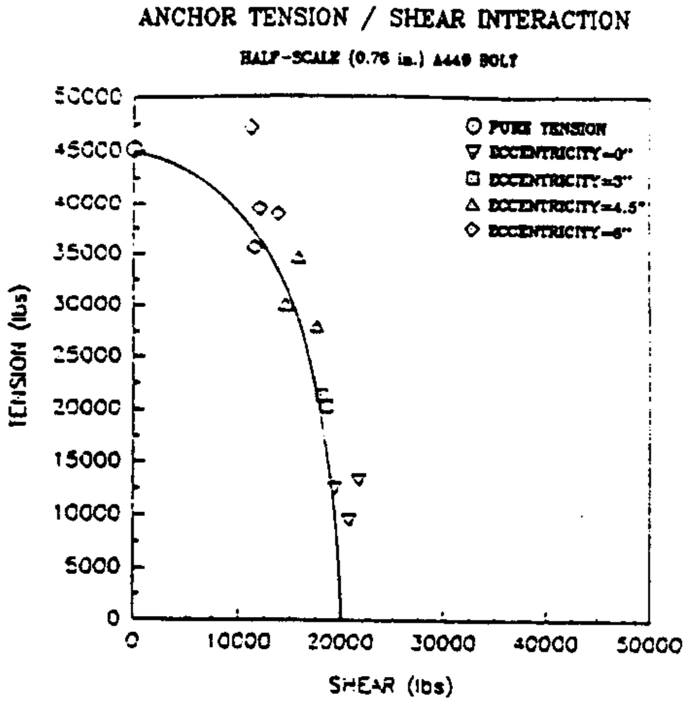


(a) failure dominated by shear

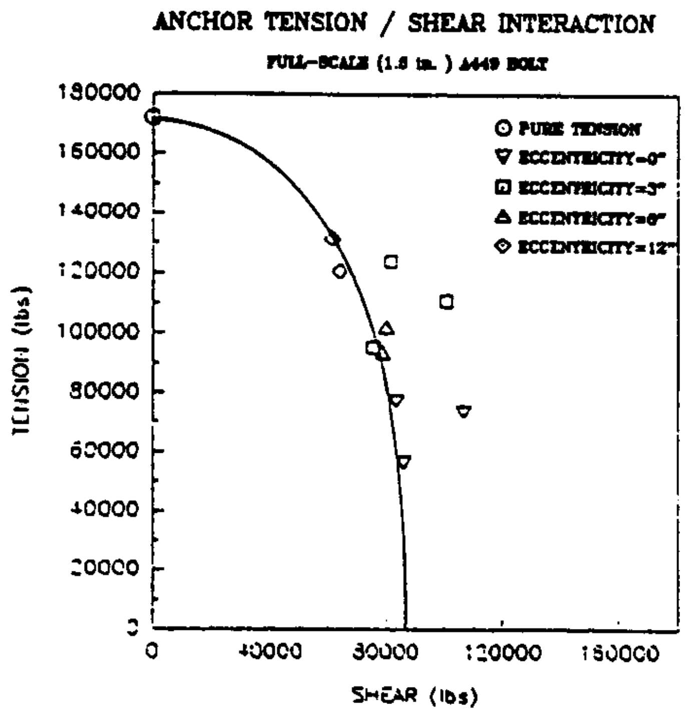


(b) failure dominated by tension

Figure 10 Typical load-vs-displacement for half-scale single anchor bolt: (a) failure dominated by shear, (b) failure dominated by tension.

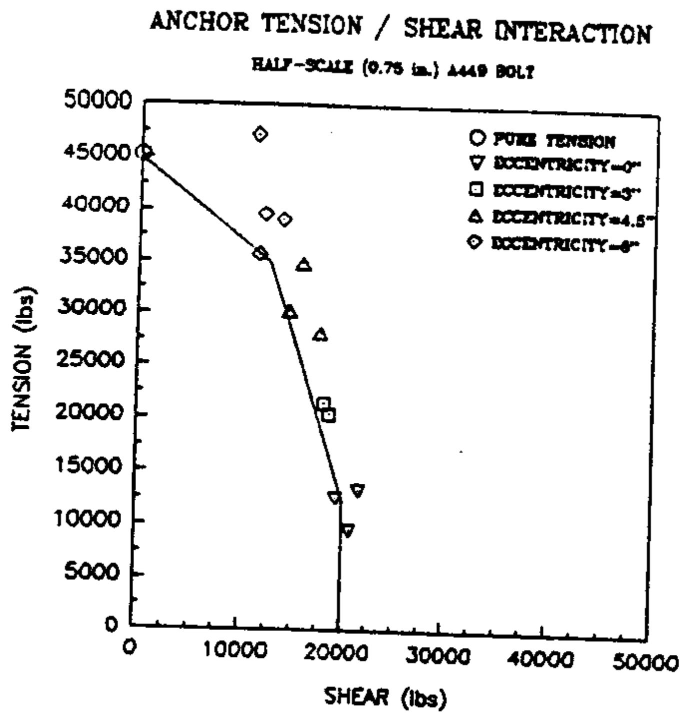


(a) half-scale

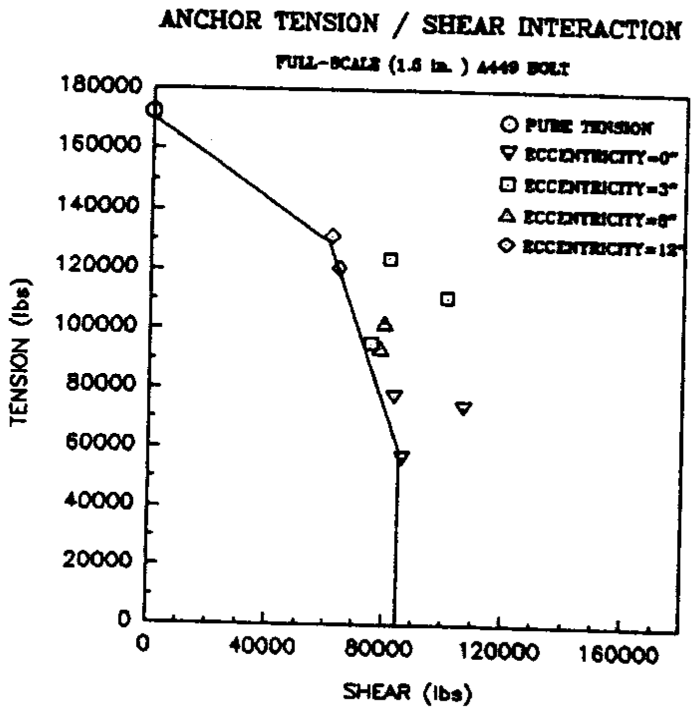


(b) full-scale

Figure 11 Elliptical tension/shear interaction relationship for single anchor bolts: (a) half-scale, (b) full-scale.



(a) half-scale



(b) full-scale

Figure 12 Tri-linear tension/shear interaction for single anchor bolts: (a) half-scale, (b) full-scale.

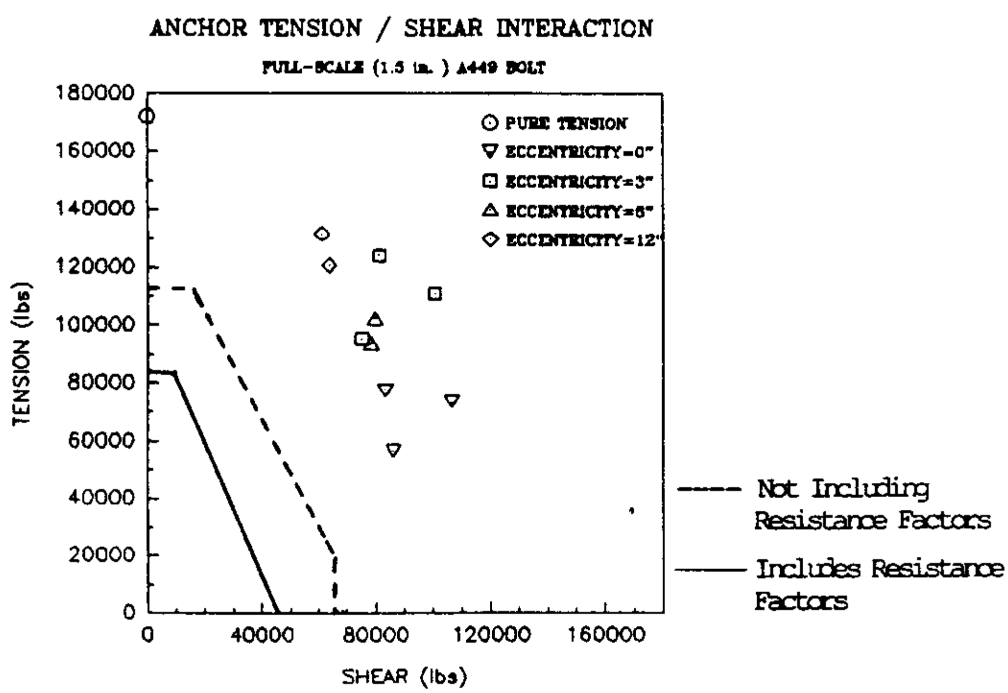


Figure 13 Comparison between full-scale single anchor bolt test results and AISC/LRFD tensile force limits.

Table 1 Full-Scale Single Anchor Bolt Results

FULL-SCALE SHEAR, TENSION AND DISPLACEMENT AT ULTIMATE LOAD					
ECCENTRICITY e (inch)	TEST NUMBER	SHEAR V_B (lb)	TENSION T (lb)	DISPLACEMENT	
				δ_h (inch)	δ_v (inch)
PURE TENSION	1(0) *	0	172160	-	-
0	1(1)	109305	73930	1.268	0.028
0	2(2)	86129	56849	0.907	0.024
0	3(7)	83374	77655	1.038	0.031
3	1(8)	81244	124218	0.856	0.106
3	2(9)	75250	95405	0.694	0.099
3	3(10)	100669	111066	1.135	0.159
6	1(3)	79866	102447	0.813	0.167
6	2(4)	78476	93932	0.953	0.187
12	1(5)	63926	120860	**	**
12	2(6)	61454	131632	0.739	0.299

* Number outside parentheses is the test order for a given eccentric load, while the number inside parentheses is the test order of the specimen

** Displacement measurement not available

Table 2 Half-Scale Single Anchor Bolt Results

HALF-SCALE SHEAR, TENSION AND DISPLACEMENT AT ULTIMATE LOAD					
ECCENTRICITY e (inch)	TEST NUMBER	SHEAR V_B (lb)	TENSION T (lb)	DISPLACEMENTS	
				δ_h (inch)	δ_v (inch)
PURE TENSION	1(0) *	0	45200	-	-
0	1(1)	21683	13340	**	**
0	2(4)	20803	9582	0.635	0.053
0	3(9)	19398	12633	0.647	0.056
3	1(2)	18114	21453	0.578	0.088
3	2(5)	18656	20484	0.591	0.079
4.5	1(7)	17645	28138	0.631	0.101
4.5	2(8)	15787	34838	0.678	0.098
4.5	3(12)	14510	30222	0.534	0.092
6	1(3)	11165	47156	0.568	0.144
6	2(10)	11511	35693	0.538	0.111
6	3(11)	12017	39540	0.541	0.120
6	4(13)	13801	39013	0.500	0.127

* Number outside parentheses is the test order for a given eccentric load, while the number inside parentheses is the test order of the specimen

** Displacement measurement not available

Table 3 Half-Scale Multiple Anchor Bolt Results

SHEAR, TENSION AND DISPLACEMENTS AT ULTIMATE LOADS						
ECCENTRICITY e (inch)	TEST NUMBER	SHEAR $V_{B1}+V_{B2}$ (lb)	TENSION		DISPLACEMENTS	
			T_1 (lb)	T_2 (lb)	δ_{hl} (inch)	δ_v (inch)
0	1(1) *	41336	14798	15552	0.008	0.510
0	2(6)	40039	10972	12011	0.006	0.632
4.5	1(4)	35765	27586	20241	0.042	0.468
4.5	2(5)	36583	21304	1858	0.066	0.485
6	1(2)	40391	34173	975	0.088	0.503
6	2(3)	36282	23303	2300	0.057	0.486

* Number outside parentheses is the test order for a given eccentric load, while the number inside parentheses is the test order of the specimen

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