Load Testing of a Precast Concrete Double-Tee Flange Connector

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The performance of precast concrete connections during seismic events is of particular concern to structural engineers. This article presents the results of tests conducted on the JVI Vector Connector, a commercially available shear connector designed for use between precast concrete elements such as double-tee flanges. Twenty-nine tests were conducted, simulating forces likely to occur under service conditions: monotonic and cyclic horizontal shear force, monotonic and cyclic horizontal shear force with applied tension, monotonic tension-only force, and monotonic vertical shear force. Load and deformation behavior was recorded for each test to support the evaluation of connector strength and ductility. Results showed good consistency between identical tests and with the results of a similar investigation carried out by Oliva. Discussion of test results and recommendations for future flange connector research are presented.

Flange connectors are used to join adjacent precast, prestressed concrete double-tee members, which are commonly used in parking structure construction. These flange connectors not only facilitate the equalizing of camber between adjacent precast units, but also provide for transfer of gravity loads and lateral forces between units. For transfer of lateral forces, particularly in regions of high seismicity, cast-in-place concrete topping reinforced with welded wire fabric is typically used for diaphragms. In recent years, questions have been raised as to the respective contributions of the flange connections and the cast-in-place toppings in diaphragm behavior.

While the flange connectors have been extensively used for many years in precast, prestressed concrete construction, only a very limited amount of testing has been conducted to provide an adequate basis for their design. Furthermore, the performance of precast double-tee diaphragms in the 1994 Northridge Earthquake in California and other recent seismic events has raised fundamental questions regarding typical diaphragm design. These concerns have led to considerable debate among structural engineers and have prompted further research.

This paper presents the results of 29 tests conducted on A36 steel and stainless steel flange connectors embedded in precast concrete slabs and exposed to varying loads, simulating conditions experienced by flange connectors in service. The results provide an estimation of the strength and ductility of the JVI Vector Connector under different loading conditions. Details of the test program are presented, and the structural behavior of the connectors under the various types of applied loads is described.

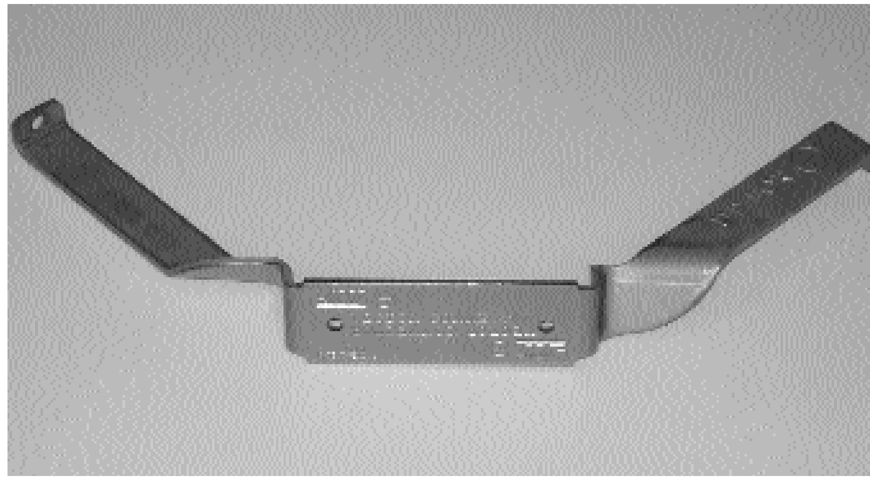

Fig. 1. The JVI Vector Connector was designed as a shear connection between adjacent precast concrete elements, such as double-tee flanges.

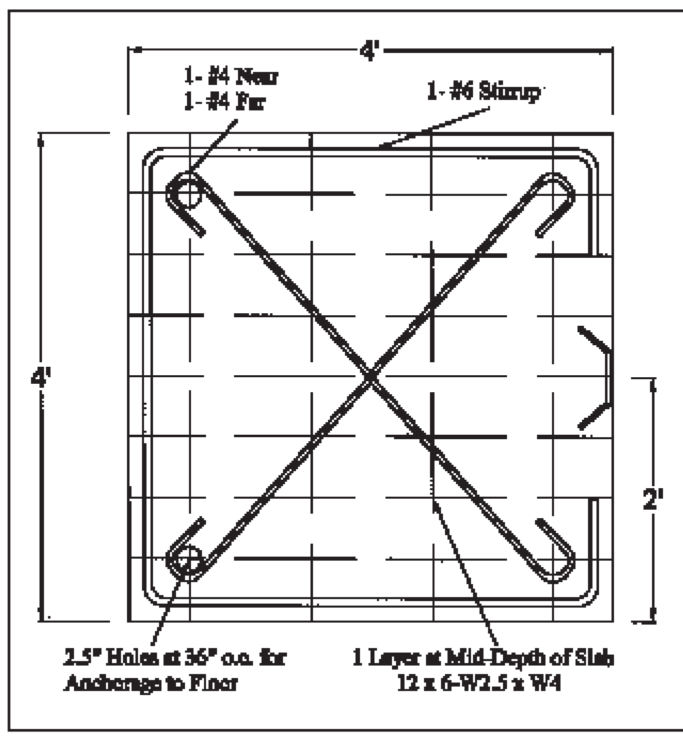

Fig. 2. Detail of test specimen.

PREVIOUS RESEARCH

As previously mentioned, very little research has been conducted to estimate the strength and ductility of precast concrete flange connectors. One of the first experimental testing programs on flange connectors was carried out by Pincheira et al.[1] Following common practice at the time, the flange connectors comprised two 12 in. (305 mm) long No. 3 (10M) steel reinforcing bars welded at a 45 degree angle to a 6.0 x $1\frac{1}{2}$ x $\frac{1}{4}$ in. (152 x 38 x 6.4 mm) steel plate. These flange connectors were the precursors to connectors currently produced commercially.

The failure of and severe damage to several parking structures during the 1994 Northridge Earthquake led to concerns regarding the effectiveness of current diaphragm design in general. Specific concerns also arose at this time as to the properties and performance of flange connectors used in conjunction with topped (cast-in-place) slabs. These issues were discussed by Fleischman et al.,[2] and this work has further focused attention on the subject of diaphragm design.

The most comprehensive testing conducted on flange connectors, prior to Fleischman's study, was completed by Oliva.[3] These tests, conducted on the first generation of the JVI Vector Connector (manufactured by JVI Inc. of Lincolnwood, Illinois), served as the basis for the testing on the modified version of the connector conducted for this report (see Fig. 1). As will be discussed later, several different combinations of load were applied to determine the overall behavior of the connector.

Recently, the necessity for topped slabs to provide diaphragm action was questioned. Cleland and Ghosh[4] summarize the thoughts of many structural engineers in questioning whether flange connectors alone can be sufficient for the required diaphragm action. These issues raised awareness within the design community and led to an increase in flange connector research, including the tests described herein.

TEST PROGRAM

Specimen Detail

To simulate the actual conditions of a connector in a double-tee flange in the laboratory, the flange connectors used for this study were embedded in 48 x 48 x 4.0 in. (1220 x 1220 x 102 mm) concrete slabs. Each slab speci-

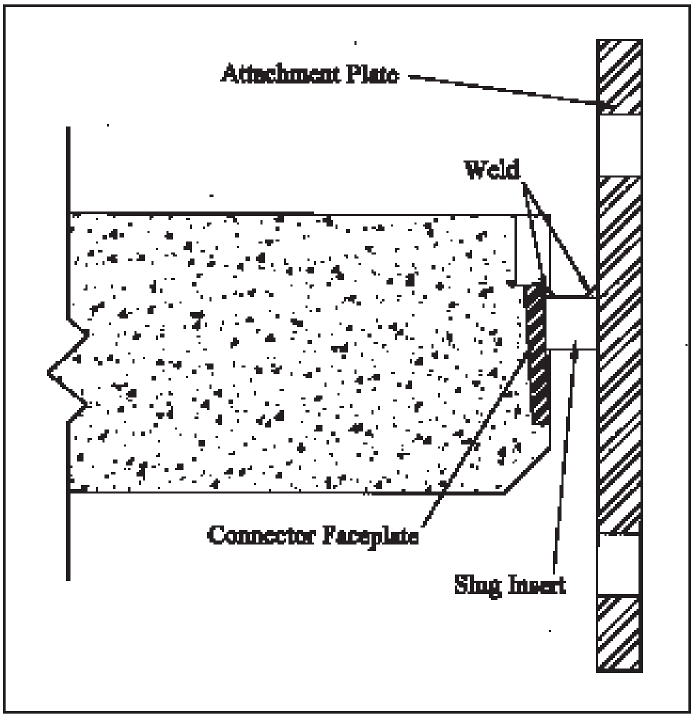

Fig. 3. Detail of connector/plate assembly.

men had a flange connector placed at the midpoint of the slab. A detail of slab dimensions and reinforcement are shown in Fig. 2. Note that two 2.5 in. (64 mm) diameter holes set 36 in. (915 mm) apart were cast into the slabs for anchorage to the floor of the testing facility. For ease of testing, each specimen had a rectangular slug welded to the flange connector. A $^5/_8$ in. (16 mm) thick steel plate was welded to the slug. This plate was used to apply the specified forces to the connection and to speed the testing process in later stages.

A custom fixture was fabricated to aid in the welding process and to ensure consistency between test specimens. AWS-certified welders produced all the required welds in the plant of the specimen producers, PBM Concrete of Rochelle, Illinois. A detail of the plate/connector configuration is provided in Fig. 3. The following details were used for this configuration:

1. Specimens used $^3/_4$ x $^3/_4$ x 4.0 in. (19 x 19 x 102 mm) rectangular slugs (made of A36 steel or A304 stainless steel).

2. The top of the slugs were positioned $^1/_4$ in. (6.4 mm) below the top of the connector.

3. The slug-to-connector weld was made with a $^1/_4$ in. leg by 4.0 in. long (6.4 x 102 mm) fillet weld using 7018 electrodes with the A36 steel and 309 electrodes with the stainless steel.

Appropriate testing was conducted on all materials used for fabricating the test specimens. All concrete had a compressive strength of 5100 to 5400 psi (34.5 to 37.2 MPa) at time of test-

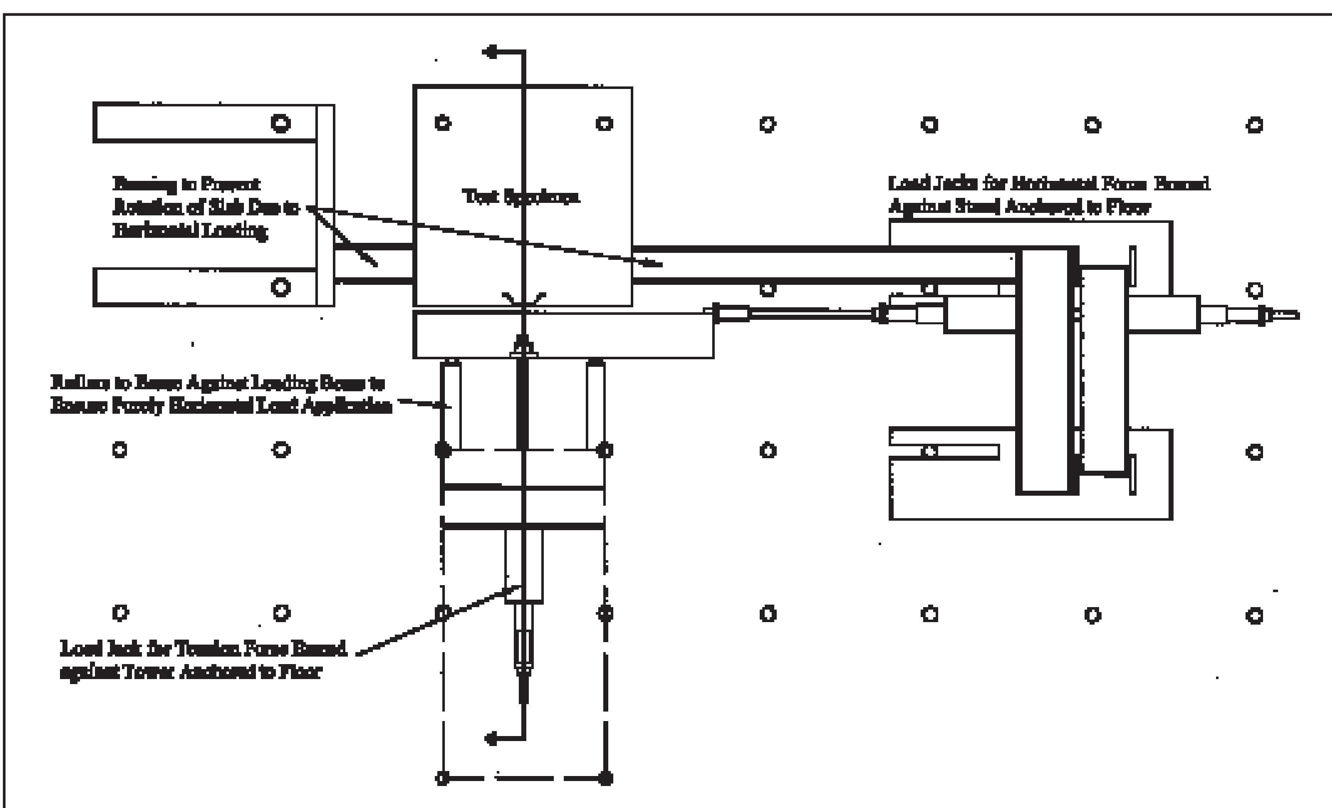

Fig. 4. Plan of horizontal shear test setup.

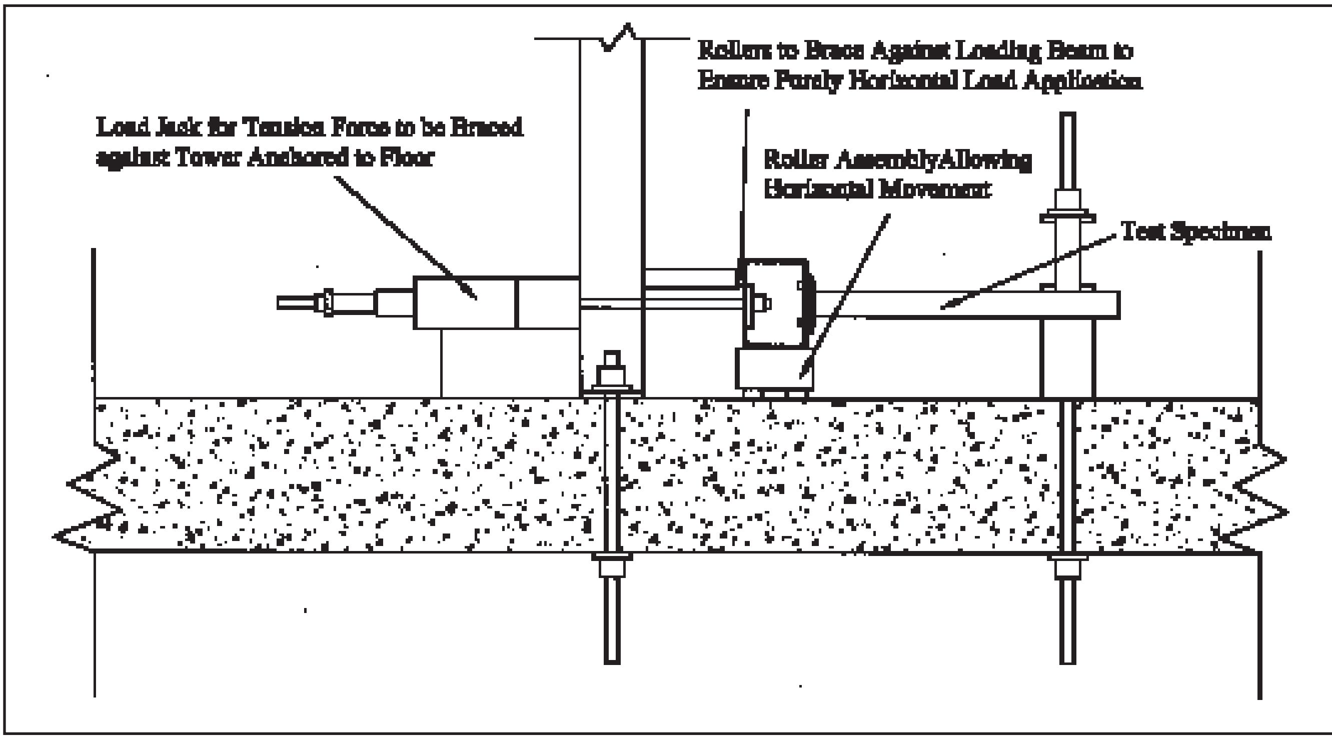

Fig. 5. Section of horizontal shear test setup.

ing. For the flange connector steel, tension test coupons were fabricated out of flange connectors from the same batch as those cast in the test specimens. The A36 steel coupons exhibited yield stresses of 60 to 62 ksi (413.7 to 427.5 MPa) and ultimate stresses of 71 to 74 ksi (489.5 to 510.2 MPa). Stainless steel coupons exhibited yield stresses of 50 to 52 ksi (344.8 to 358.5 MPa) and ultimate stresses of 96 to 98 ksi (662.0 to 675.7 MPa).

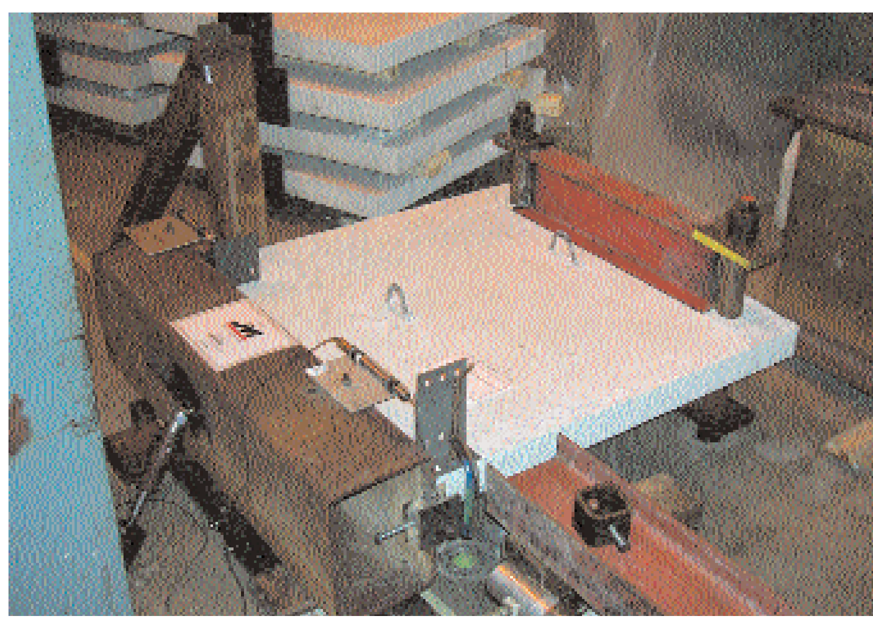

Fig. 6. Horizontal shear test setup. LVDTs were placed on both sides of the concrete slab to record relative slip between the slab and the loading beam.

Test Setup and Procedure

For each of the various types of tests conducted, test procedures were kept similar to allow for a reliable comparison of data between tests. The following description of each of the test procedures gives details of the load application and instrumentation setup; these and other test variables were kept the same to allow for consistency between tests. For all of the test setups employed, loading was applied gradually so that data readings could be taken at regular intervals and so that the specimens could be inspected at regular intervals during the testing.

Monotonic horizontal shear – Line drawings and a photo of the test setup employed for horizontal shear tests (including those with an initial tension force applied to the connection) are given in Figs. 4 through 6. Each specimen was held in place using a 3.0 ft (0.9 m) spreader beam that was anchored to the floor of the laboratory. The slabs were also braced on each side to prevent slab rotation due to applied horizontal shear loads. As previously mentioned, each specimen had a plate welded to the connection allowing the specimen to be bolted to a custom-fabricated tube, which was used to apply the necessary forces to the connection.

The loading tube rested on rollers, allowing free movement when horizontal loads were applied. Rollers were also placed against the face of

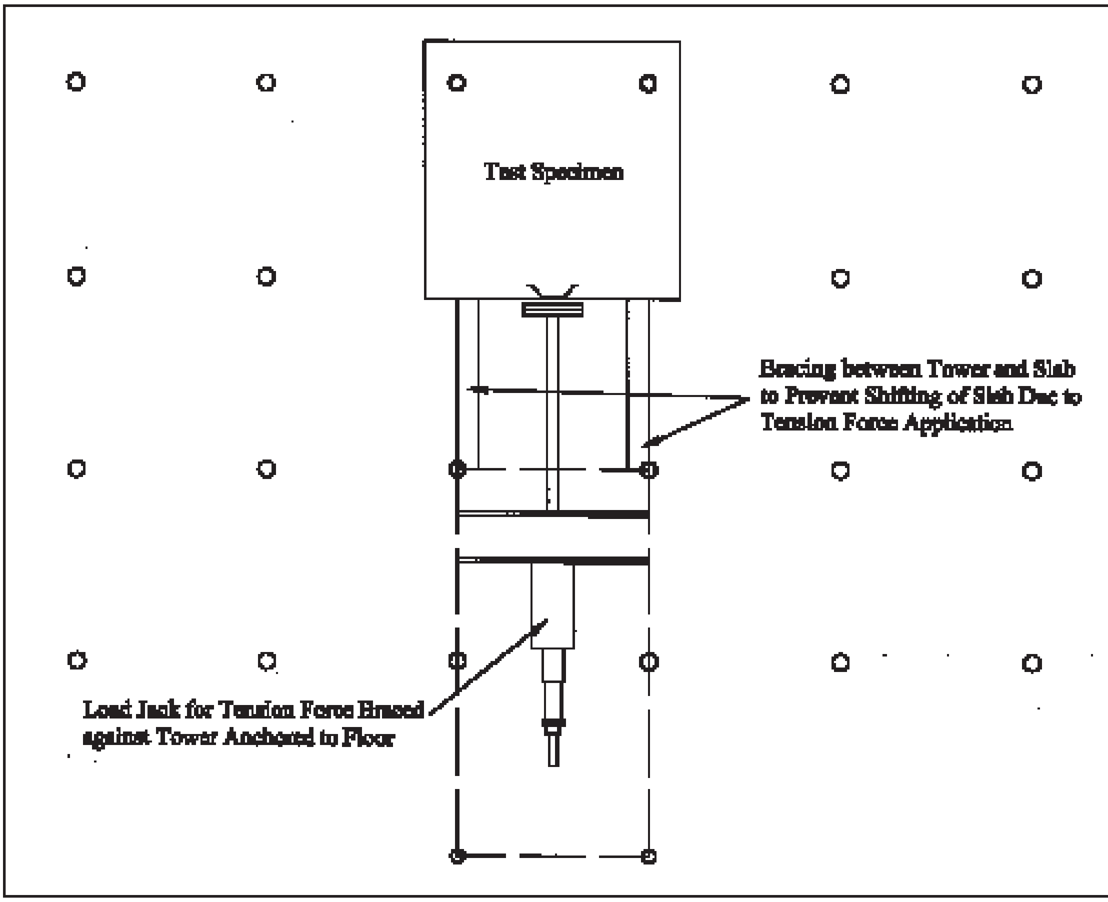

Fig. 7. Plan of tension test setup.

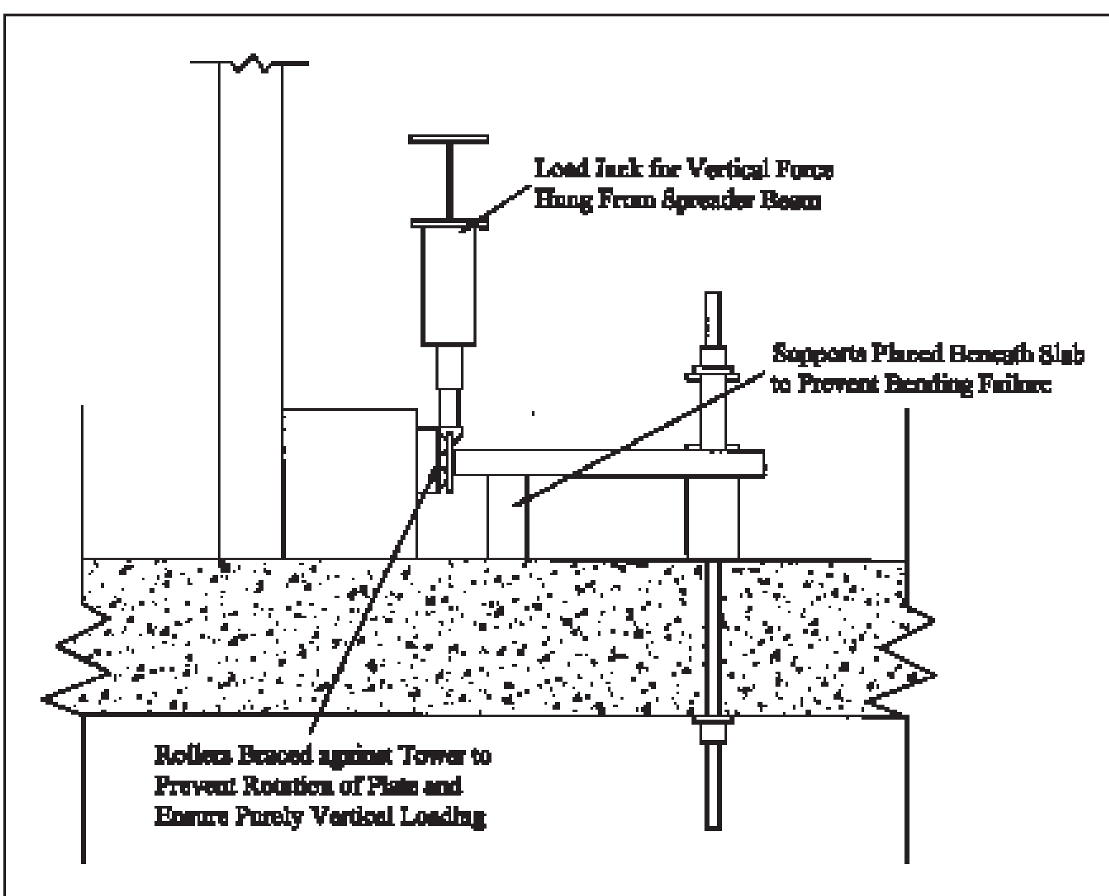

Fig. 8. Section of vertical shear test setup.

the tube at each end, ensuring that there was no rotation of the tube due to the applied horizontal loads. Horizontal loads were applied to the loading tube by two hydraulic jacks braced against a heavy frame anchored to the floor. Horizontal forces were applied by one of the two jacks at a time to create the intended push/pull force sequence. For all variations of horizontal shear tests, linear variable differential transducers (LVDTs) were placed on both sides of the slab (see Fig. 6) to read relative slip between the slab and the loading beam.

Monotonic horizontal shear with tension – The procedure for these tests was the same as that for monotonic horizontal shear without tension. However, for these tests the connection was initially loaded with a tension force to cause 0.1 in. (2.54 mm) of joint opening. This 0.1 in. (2.54 mm) displacement was read from a dial gauge mounted on the slab and placed against the plate near the connection. This tension load was then held while the appropriate horizontal forces were applied.

A tension force was applied to the connection by a rod attached to the tube with force provided by another hydraulic jack. The jack for applying the tension force was braced against a tower anchored to the floor. The connection between the tube and the tension rod was designed to allow for the horizontal displacement that would occur when horizontal loads were applied. The application of the tension force was perpendicular to the center of the connector face. Tension force was applied using a hydraulic pump, allowing for the desired amount of displacement to occur.

Cyclic horizontal shear – The test program utilized for cyclic tests was based on the program developed and used by Oliva.[1] Load application for this test setup was based on the results of the monotonic horizontal shear tests. The yield force and corresponding displacement for each type of steel were estimated and used as a basis for cyclic loading. For cyclic testing, each specimen was first cycled back and forth to a load of 75 percent of the yield force. Subsequent load cycles were applied in groups of three at displacement levels of 2, 3, 5, 8, and 12 times the yield displacement until connection failure.

Table 1. Summary of test results on the JVI Vector Connector.

		Ultimate load (lb)	
Test type	Specimen number	A36 steel	Stainless steel
Monotonic horizontal shear without tension	1	18,613	19,523
	2	20,723	21,055
Cyclic horizontal shear without tension	3	21,433	23,208
	4	20,203	22,306
Monotonic horizontal shear with tension	7	13,183	18,187
	8	13,120	15,748
Cyclic horizontal shear with tension	9	14,802	17,070
	10	15,573	21,508
Monotonic tension	5	6,293	11,807
	6	7,513	11,360
	15x	N/A	13,855
Vertical shear, 4 in. slab	11	N/A	6,000
	12	5,876	6,000
	13	5,500	5,989
	14	N/A	5,400
Vertical shear, 6 in. slab	1	N/A	6,000
	2	N/A	6,600

Note: 1 lb = 4.448 N; 1 in. = 25.4 mm.

Table 2. Estimated yield forces and displacements.

Steel type	Estimated yield force (lb)	Estimated yield displacement (in.)
A36 steel	17,300	0.045
Stainless steel	15,000	0.175

Note: 1 lb = 4.448 N; 1 in. = 25.4 mm.

Cyclic horizontal shear with tension – The test program used for these tests was similar to that for the cyclic horizontal shear tests; however, an initial tension was again introduced to the connection as in the monotonic test.

Monotonic tension force – A line drawing of the test setup employed for pure tension tests can be seen in Fig. 7. The setup for these tests was very similar to that for tension force application in the horizontal shear tests. Load was applied by a rod welded to a custom plate, which was bolted to the plate welded to the connection. Special attention was given to ensure that load was applied at the center of the connection. For pure tension tests, bracing was placed between the tower and the slab to prevent slipping of the slab due to the applied loads. Tension loads for these tests were also applied gradually, allowing for data gathering of load and displacement. For tension tests also, two LVDTs, one on each side of the connection, were employed. The LVDTs rested on the slab and measured the opening of the connection relative to the slab.

Monotonic vertical shear – A line drawing of the test setup for vertical shear tests can be seen in Fig. 8. For vertical shear, the slabs were again anchored to the floor as in the other tests. For vertical shear testing, supports were placed below the slab closer to the connection to prevent bending failures of the slab upon load application. To apply the load, a hydraulic jack was hung from a spreader beam over the connection. Special attention was given prior to each test to ensure that the load was applied vertically at the center of the plate. A custom fixture was fabricated to apply the load to the plate. Rollers were also placed against the plate to prevent rotation and to ensure that a purely vertical force was being applied. Note that LVDTs measured the vertical displacement of the plate relative to the slab.

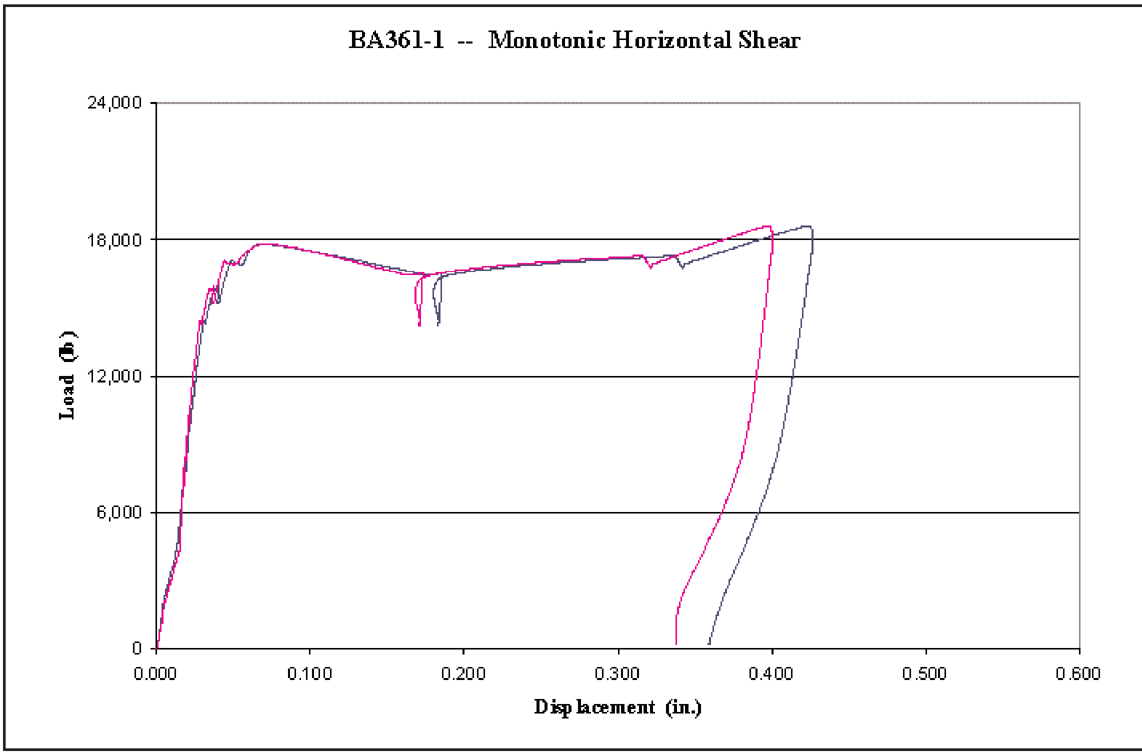

Fig. 9. Data load-displacement graph for horizontal shear without tension.

Fig. 10. Typical failure for horizontal shear without tension.

TEST RESULTS

Table 1 lists the results of all tests conducted as part of this study. The complete report is available through the JVI, Inc., website at www.jvi-inc.com. Brief descriptions of the types of failure that occurred for each type of test are discussed below.

Monotonic Horizontal Shear without Tension

Results of this series of tests were consistent between the two types of steel. The cracking loads and the ultimate loads were similar. The load-deformation behavior of the connection, however, varied significantly with steel type. The ductility of the stainless steel connectors was almost twice that of the A36 connectors. All specimens tested under this load type had very similar failures – i.e., cracking of the concrete at approximately 14 kips (62.3 kN), with pullout of the tension leg of the connector occurring at 19 to 20 kips (84.5 to 89.0 kN). Typical cracking for this type of test included spalling directly above the connection, crushing of the concrete due to the connector bearing on the side opposite of load application, and splitting of the slab at failure due to the pulling out of the tension leg of the connector.

Results of this series of tests were significant because, as previously mentioned, they would serve as the basis for the program employed for all cyclic testing. The values estimated from this series of tests are provided in Table 2. Fig. 9 shows a sample load-displacement graph and Fig. 10 depicts a typical failure.

Cyclic Horizontal Shear without Tension

As noted previously, the program for all cyclic testing was based on the yield force and displacement obtained from the tests for monotonic horizon-

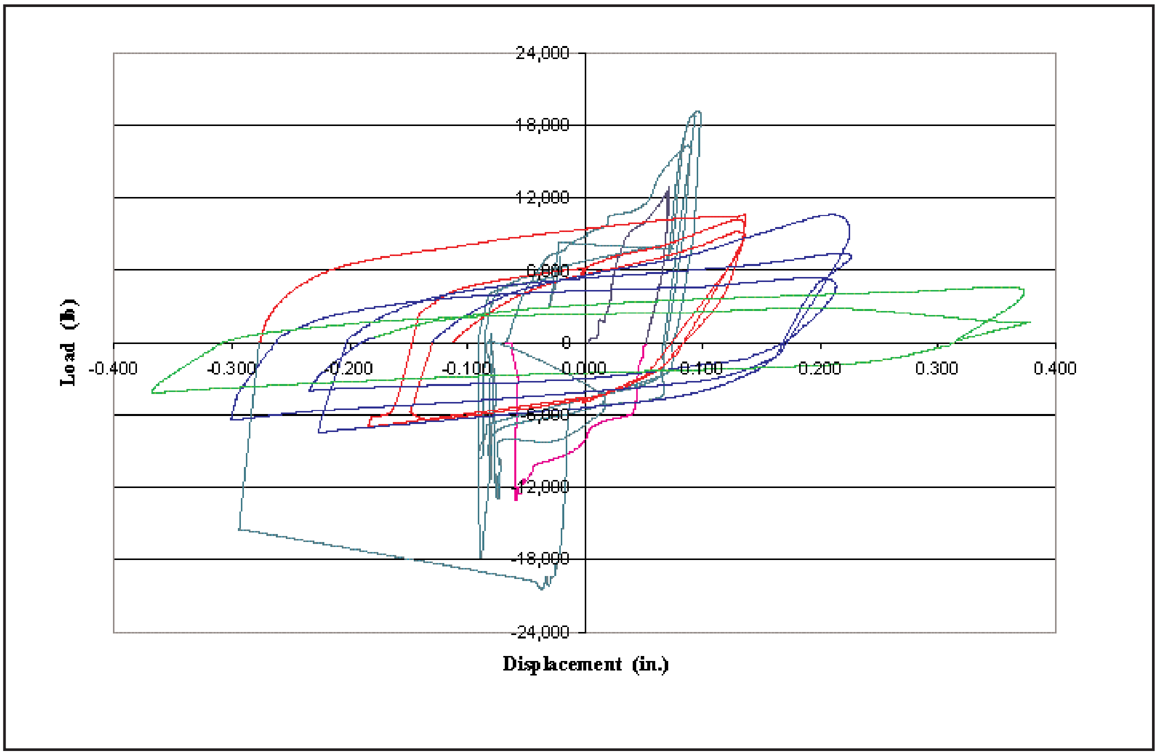

Fig. 11. Data graph: cyclic horizontal shear without tension.

tal shear without tension. Specimens were first cycled back and forth to a load of 75 percent of the estimated yield force. Specimens were then cycled in groups of three to displacements of 2, 3, 5, 8, and 12 times yield displacement until connection failure. The resulting cyclic programs for each type of steel were:

A36 steel

1. Cycle once to 13 kips (57.8 kN)
2. Three cycles at 0.090 in. (2.29 mm)
3. Three cycles at 0.135 in. (3.43 mm)
4. Three cycles at 0.225 in. (5.72 mm)
5. Three cycles at 0.360 in. (9.14 mm)
6. Three cycles at 0.540 in. (13.72 mm)

Stainless steel

1. Cycle once to 11.25 kips (50.0 kN)
2. Three cycles at 0.350 in. (8.89 mm)
3. Three cycles at 0.525 in. (13.34 mm)

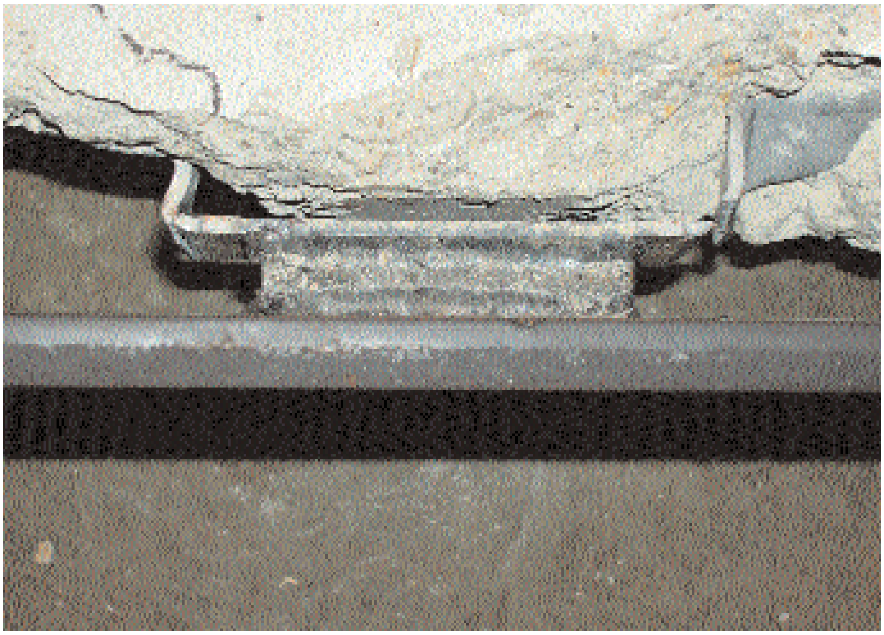

Fig. 12. Typical failure for cyclic horizontal shear without tension.

A36 steel – Tests for A36 connectors displayed very similar ultimate loads at approximately 21 kips (93.4 kN). Ductility, however, varied greatly between the two tests. Test BA361-3 failed on its first cycle up to 0.225 in. (5.72 mm), while Test BA361-4 failed on its second cycle up to a displacement of 0.360 in. (9.14 mm). In comparing test observations, Test BA361-4 experienced cracking at a much later stage, accounting for the extended life of the connection. Failure for these tests, as well as all other cyclic tests, was the result of face plate rupture.

Typically, concrete cracking occurred during the first displacement cycle, causing the concrete surrounding the face plate to break away. This left the connection free to displace back and forth with the cycles. As higher displacements were reached, the face plate began to tear on both sides. This tearing occurred at the first bend in the connector between the face plate and the leg. Typically, once tearing had initiated, the connector still functioned for at least two more cycles.

Stainless steel – Tests for stainless steel exhibited ultimate loads of approximately 22.5 kips (100.1 kN).

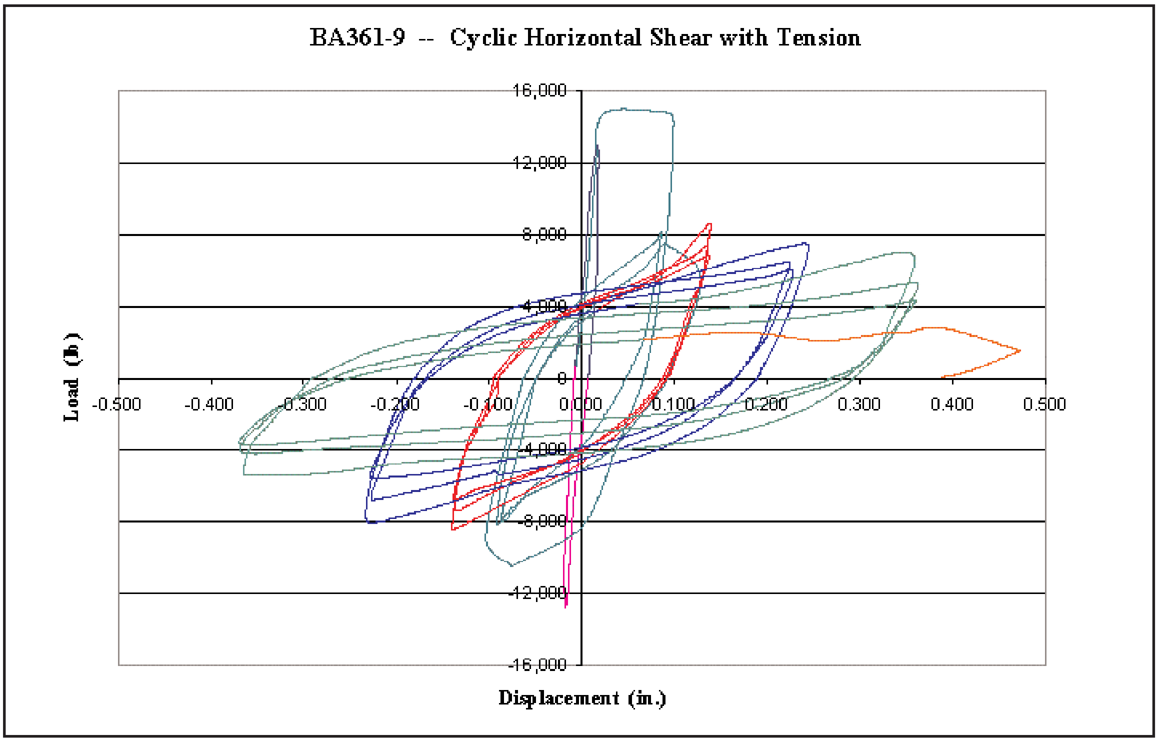

Fig. 13. Data graph: cyclic horizontal shear with tension.

Ductility was the same for both Tests BSS1-3 and BSS1-4, failing on the first cycle down to 0.525 in. (13.34 mm). The mode of failure was similar to those previously described. A photograph of a typical failure and a sample data graph for this type of test can be seen in Figs. 11 and 12.

Monotonic Horizontal Shear with Tension

For this series of tests, horizontal load was applied as previously mentioned. However, for these tests an initial tension force was applied to the connection. All failures for this series of test were again due to pullout of the tension leg as was seen in the same test without a tension force applied to the connection. The intended tension force to be applied to the connection was to cause a joint opening of 0.1 in. (2.54 mm); for various reasons this was not always accomplished, as will be discussed in the following paragraphs, .

A36 steel – Ultimate load (13 kips or 57.8 kN), as well as ductility for Tests BA361-7 and BA361-8 were nearly identical. For Test BA361-7, the tension load was applied to cause a joint opening of 0.1 in. (2.54 mm). As this load was applied, however, further deformation occurred causing a joint opening of 0.15 in. (3.81 mm). Tension load was released and a permanent deformation of more than 0.1 in. (2.54 mm) had occurred.

A load was then reapplied to the connection to place it under tension and held while horizontal testing was initiated. For Test BA361-8, tension load was increased to cause a joint opening of 0.1 in. (2.54 mm), this load was then held and horizontal testing was initiated.

Stainless steel – Differences in applied tension load explain the varying results between Tests BSS1-7 and BSS1-8. For Test BSS1-7, as tension load was applied, the joint opening increased to 0.12 in. (3.05 mm) of permanent deformation. The load was then released and horizontal testing was carried out with no tension force applied to the connection. For Test BSS1-8, tension load was applied to cause a 0.1 in. (2.54 mm) joint opening. This force was then held while horizontal testing was conducted.

Cyclic Horizontal Shear with Tension

Results of this series of tests were again very consistent. For all tests in this series, a tension load was applied which resulted in 0.1 in. (2.54 mm) of joint opening. This load was then held while cyclic horizontal testing was conducted. Failure for these tests was identical to cyclic tests conducted with no tension force applied; rupture at the ends of the face plates.

A36 steel – Ultimate loads, as well as load-deformation behavior for Tests BA361-9 and BA361-10, were nearly identical. Test BA361-9 failed on its first cycle up to 0.540 in. (13.7 mm). Test BA361-10 failed on its final cycle down to 0.360 in. (9.14 mm). Ultimate loads for both tests were approximately 15 kips (66.7 kN). For both tests, the tension load applied to produce a 0.1 in. (2.54 mm) joint opening was 4.6 kips (20.5 kN).

Stainless steel – Results for Tests BSS1-9 and BSS1-10 varied. Ultimate load ranged from 17 to 21 kips (75.6 to 93.4 kN). Ductility was comparable, with Test BSS1-9 failing on its first cycle up to 0.525 in. (13.3 mm) and Test BSS1-10 failing on its first cycle down to 0.525 in. (13.3 mm). For both tests, the approximate tension force applied to produce a 0.1 in. (2.54 mm) joint opening was 6.4 kips (28.5 kN).

A sample data graph for this type of test can be seen in Fig. 13. Photographs of typical failures have not been included for this series of tests since they are identical to tests done without tension applied to the connection.

Monotonic Tension

Results for this series of tests were again consistent. As the tension load was applied to the connection, concrete around the face plate broke off and the face plate began to fold downward. Once the face plate reached an approximate angle of 30 degrees from its original position, the weld began to tear from each end, propagating rapidly towards the center. This mode of failure occurred for all monotonic tension tests.

A36 steel – Ultimate loads for Tests BA361-5 and BA361-6 ranged from 6.3 to 7.5 kips (28.0 to 33.4 kN). Load-deformation behavior was also very similar, with cracking occurring at 6.0 kips (26.7 kN).

Stainless steel – Ultimate loads for Tests BSS1-5 and BSS1-6 were nearly identical at approximately 11.5 kips (51.2 kN). Load-deformation behavior was also similar with cracking around 6.0 to 7.0 kips (26.7 to 31.1 kN). A typical failure for this type of test can be seen in Fig. 14.

Because all failures in stainless steel tests were due to weld rupture, one additional test, BSS1-15x, was conducted. This specimen had extra welds placed on the top and bottom of the slug to determine the achievable connector strength by prevention of weld failure. Initial cracking for Test BSS1-15x occurred at an applied load of 8.7 kips (38.7 kN). Failure for this test was similar to those described for previous tension-only tests; however, the connector face plate did not fold down. Due to the symmetry caused by welds on the top and bottom, the face plate remained vertical throughout testing. The ultimate load for Test BSS1-15x was 13.9 kips (61.8 kN). Failure was due to pullout of one of the connector legs.

Vertical Shear

Vertical shear test results were also remarkably consistent. All failures were due to spalling of the concrete on the bottom surface of the slab below the connector. All failures were brittle due to failure of the concrete; thus, the ultimate load was the same as the cracking load.

Fig. 14. Typical monotonic tension test failure.

A36 steel – Ultimate loads for Tests BA361-12 and BA361-13 ranged from 5.5 to 5.9 kips (24.5 to 26.2 kN).

Stainless steel – Tests BSS1-11 through BSS1-13 were all performed on slabs with similar construction. Ultimate loads were 6.0 kips (26.7 kN) for each test. Specimen BSS1-14 was constructed with the connector placed below the welded wire fabric. This is the probable cause for the reduction in ultimate load to 5.4 kips (24.0 kN). Two tests were also conducted on 6.0 in. (152 mm) slabs. Slab – 1 had an ultimate load of 6 kips (26.7 kN). Slab – 2 had $^{3}/_{4}$ in. (19 mm) more concrete cover below the connector than the previous slab, which may explain the increase in ultimate load to 6.7 kips (29.8 kN); both Slabs 1 and 2 were 6.0 in. (152 mm) specimens.

OBSERVATIONS

Twenty-seven tests were conducted on single connectors, each embedded in 48 x 48 x 4 in. (1220 x 1220 x 102 mm) thick concrete slabs. The remaining two tests were conducted on the same connector embedded in 6.0 in. (152 mm) thick slabs. The connector installation in the slabs modeled typical flanges of precast double-tee members. The connectors were made of A36 structural grade steel and A304 stainless steel. Based on a review of the results of this testing, the following observations are made:

1. The results of identical tests within each load category showed remarkable consistency, which should enable better predictability in the design process.

2. The horizontal shear strength with tension load across the joint was consistently about 70 percent of the corresponding strength without tension load.

3. In all cyclic horizontal shear tests, failure occurred due to rupture of the connector face plate from the legs, demonstrating sufficient anchorage in concrete.

4. In the tension-only tests, the failure occurred due to rupture of the weld and, thus, the connector tension strength could not be achieved. Pretest visual examination of the specimens and welding did not reveal any obvious flaws or cracking in the weld zones.

5. Spalling of concrete at the bottom of the slab occurred in all vertical shear tests. Use of auxiliary reinforcement such as welded wire fabric is recommended to help prevent such spalling.

RECOMMENDATIONS FOR FUTURE RESEARCH AND DEVELOPMENT

The tests carried out as part of this study should provide significant data for the design profession to formulate appropriate procedures. These tests also point to the need for additional information to support the design process. With this objective, the following recommendations are offered for further research and development:

1. Testing in this program and in previous programs has focused on single connectors. It would be desirable to carry out some multiple connector tests to verify the superposition of individual strengths for design. This would be particularly important in vertical shear transfer since the load will reach connections via bending and twisting of the double-tee flange.

2. Finite element modeling would be a useful approach to optimize connector geometry, especially with respect to its anchorage in concrete.

3. Guidelines for design of welding should be developed to achieve full connector strength by preventing weld failures.

4. Testing for pullout strength of connector legs embedded in concrete at different angles would be useful to better define anchorage capacity.

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