Tests on Double Tee Flange Connectors Subjected to Monotonic and Cyclic Loading

José A. Pincheira, Ph.D.
Assistant Professor
Department of Civil and
Environmental Engineering
University of Wisconsin, Madison
Madison, Wisconsin

Michael G. Oliva, Ph.D.
Associate Professor
Department of Civil and
Environmental Engineering
University of Wisconsin, Madison
Madison, Wisconsin

Fadjar I. Kusumo-Rahardjo
Design Engineer
Precast Engineering Co.
Elm Grove, Wisconsin

The behavior of a particular double tee flange-to-flange connector was investigated experimentally under in-plane load conditions. The connector consisted of a steel plate with two fillet-welded reinforcing bars embedded in 2 in. (50 mm) thick concrete slabs. Ten tests were conducted by applying a variety of loading conditions. Test results showed that the connector had a dependable and predictable strength under both monotonic and cyclic loading. Its deformation capacity depended on the type of applied loading. Under monotonic loading, moderate to high levels of deformation ductility were observed. Under cyclic load reversals, however, the deformation capacity and ductility of the connector were limited. Based on the test results, the performance of the connector is evaluated and criteria for future testing of similar connectors are presented.

Precast, prestressed concrete double tees are often used as roof and floor systems in commercial buildings and parking garages. In many of these systems, it is common practice to use evenly spaced mechanical flange-to-flange connectors to join adjacent double tees (see Fig. 1).

These mechanical connectors serve to compensate for varying camber and to align the flanges in the out-of-plane direction. But most important, they must resist multiple types of diaphragm forces. Rather than using mechanical connectors, double tees can be overlain with a reinforced, cast-in-place topping slab to form a composite floor system. A combination of both mechanical connectors and a topping slab is, however, sometimes preferred.

The damage incurred by buildings with topped double tee diaphragms (but without mechanical connectors) during the 1994 Northridge earthquake[1] cast doubts on the ability of these systems to perform adequately during a strong earthquake. Subsequent analytical studies[2,3] have also shown the potential weakness of the topped diaphragm system and have further emphasized the importance of the diaphragm in the lateral load resistance of precast concrete buildings. These results and the performance of some precast systems during the Northridge event prompted a re-evaluation of the performance of topped, untopped and pretopped diaphragms.

While millions of mechanical flange connectors are used each year, very little test information exists on which design procedures can be based. Most connectors have been developed through field experience by individual precast concrete manufacturers without standard test methods to determine the strength and deformation capacity. Of the research work completed to date, virtually none has examined the strength, stiffness and deformation capacity of connectors subjected to multi-axial and cyclic loading.

In this paper, the results from a pilot series of tests on 2 in. (50 mm) thick, untopped double tee flange connections are presented. The pilot tests were used to define test methods for an ongoing research program to experimentally investigate the behavior of untopped and topped double tee diaphragm connections under simulated earthquake loads.

In this study, the behavior of a common connector under biaxial monotonic and reversed cyclic loading is investigated. Details of the test program are presented and the observed behavior is described. Based on the test results, the performance of the connector for use in untopped diaphragms is evaluated and criteria for future flange connector tests are presented.

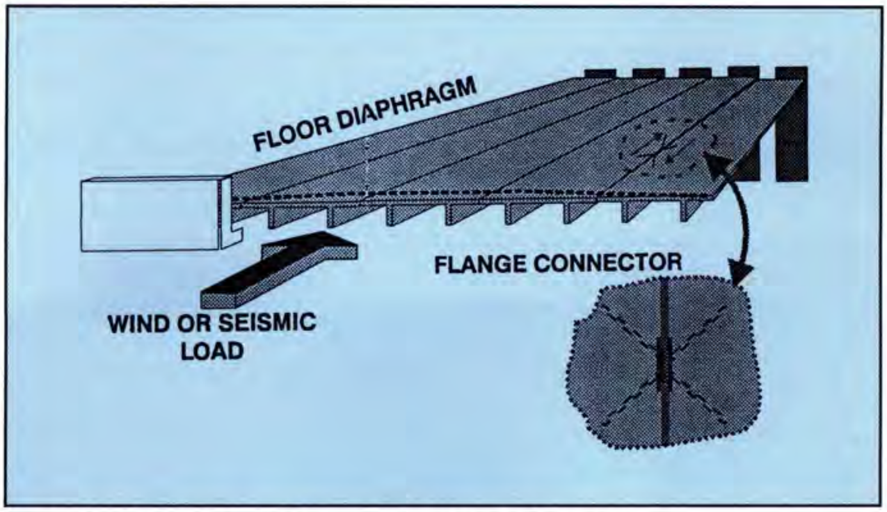

Fig. 1. Double tee floor diaphragm with mechanical connectors.

PREVIOUS INVESTIGATIONS

The largest reported study on flange connectors[4] examined reinforcing bar "hairpins," one of the most common connectors used to adjust for differential camber in thin double tee flanges. In that study, 64 push-off shear tests were conducted under monotonic loading. Forty of those tests involved the typical hairpin shown in Fig. 2, though the dimensions of the hairpin were varied. Twenty-four additional specimens had a similar bent reinforcing bar but also included a small, flat steel plate at the flange edge welded to the bar. Nearly all specimens failed by what was described as bar yield close to the maximum load with subsequent crushing and concrete spalling on the compression side of the connector.

A cyclic shear test on a nearly identical connector was conducted by Aswad.[5] This connector was described as having "good ductile behavior" with a displacement ductility of about 3 and a deformation of 0.06 in. (1.5 mm). The connector was found to be susceptible to serious weakening, however, when a simultaneous vertical shear force was present. Some precast manufacturers have discontinued the use of this popular connector because of the danger of field welds being placed too close to the bar bend resulting in brittle behavior. The plain hairpin is also susceptible to problems from poor placement in the forms before casting and from misalignment during erection in the field.

Sixteen different types of flange connectors have been examined in additional limited tests. Concrete Technology Corporation[6] tested nine specimens using monotonic shear loading. Stanton et al.[7] conducted three tests using monotonic shear combined with joint opening, and Kallros[8] and Spencer[9] have reported on 31 cyclic and 11 monotonic horizontal shear tests.

Spencer's cyclic test series[9] showed that connectors with 45-degree anchor bars in 5 in. (130 mm) thick specimens can be designed to perform adequately as yielding elements. The connector's strength, however, dropped under cyclic loading to a stable level at about 50 percent of the calculated nominal capacity due to buckling of the bar in compression. Following buckling of a reinforcing bar, the connectors showed stable hysteretic response until an average shear displacement of 0.35 in. (9 mm) when concrete spalling or weld failure occurred.

EXPERIMENTAL PROGRAM

Many types of flange connectors are currently in use by the precast concrete industry. While the purpose of these connectors is basically the same,

Fig. 2. Typical "hairpin" connector tested by Venuti and Nazarian (Ref. 4).

their individual qualities differ. The connector studied in these pilot tests consisted of a steel plate placed at the flange edge with two fillet-welded reinforcing bars.

The connection between the flanges is accomplished by field welding a small slug of steel to adjacent steel plates (see Fig. 3). This type of connector is often used in roof systems and was selected for a pilot study in consultation with professional members of the Precast/Prestressed Concrete Institute (PCI), precast concrete manufacturers and members of the project advisory panel.

Specimen and Connector Details

All specimens consisted of 2 in. (50 mm) thick concrete slabs that represented the flanges of an untopped double tee. Normal weight concrete with a design strength of 5000 psi (35 MPa) was used in all slabs. Following common practice, two No. 3 reinforcing bars welded to a 6 x $1^1/_2$ x $^1/_4$ in. (150 x 36 x 6 mm) steel plate formed the connector (see Fig. 3).

Each bar was 12 in. (305 mm) long and was welded at a 45-degree angle in a plane parallel to the double tee flange. The plate was placed at an angle of approximately 10 degrees to the vertical axis as shown in Fig. 3. In this manner, the bars remained parallel to the top and bottom surfaces of the flanges, while the steel plate offered a slight angle to receive the steel slug.

Reinforcing bars were made of ASTM A 615 Grade 60 steel,[10] while the steel plate was fabricated of ASTM A 36 steel.[11] Welding of the bars to the plate was done by preheating the bars as prescribed by AWS D1.4.[12] All connectors were fabricated by a commercial steel fabricator.

A 1 in. (25 mm) diameter round bar of A 36 steel was used as a slug to complete the connection. Welding of the slug to the steel plates was done by a certified welder with the connectors embedded in the slabs. The slug weld was sized to be stronger than the estimated strength of the connector. Accordingly, a minimum of two passes with $^1/_8$ in. (3 mm) E70 electrodes were provided over a length of 4 in. (100 mm), which resulted in an effective weld throat of about $^1/_4$ in. (6 mm).

Despite these precautions, partial fracture of the slug weld was observed in one specimen. Although this result is probably an anomaly caused by a substandard weld, it was decided to extend the weld beyond the reinforcing bar attachment points by increasing the total weld length to 5 in. (125 mm). No slug weld fractures were observed after this modification.

Test Setup and Loading Procedure

As mentioned earlier, the connector was subjected to various loading conditions that represented some of the actions expected in an untopped floor system. The actions considered here were all in the plane of the diaphragm and consisted of shear, tension, and concurrent shear and tension applied under either monotonic or reversed cyclic loading.

Loading under concurrent shear and tension was done using a constant shear-to-tension ratio of 1. This ratio was considered representative of an extreme load combination for a connector located in a diaphragm region where high shear and tension due to bending could occur simultaneously.

Testing of the specimens under different load combinations was accomplished by varying the line of action of the applied force with respect to the connector. As a result, the dimensions in plan and reinforcing details of the slabs were varied (see Figs. 4a, b and c). For the shear specimens, four connectors were cast in one slab, two connectors along each longitudinal edge of the slab (see Fig. 4a). Note that only one connector was welded and tested at a time. This was done to reduce the number of slabs and the time required to conduct the tests. Further details of the test specimens and the test setup can be found elsewhere.[13]

Under monotonic loading, the specimens were tested using force-controlled increments up to a load level corresponding to 75 percent of the estimated strength of the connector. Afterwards, the specimens were subjected to prescribed increments of displacements until failure. Under cyclic loading, the test procedure rec-

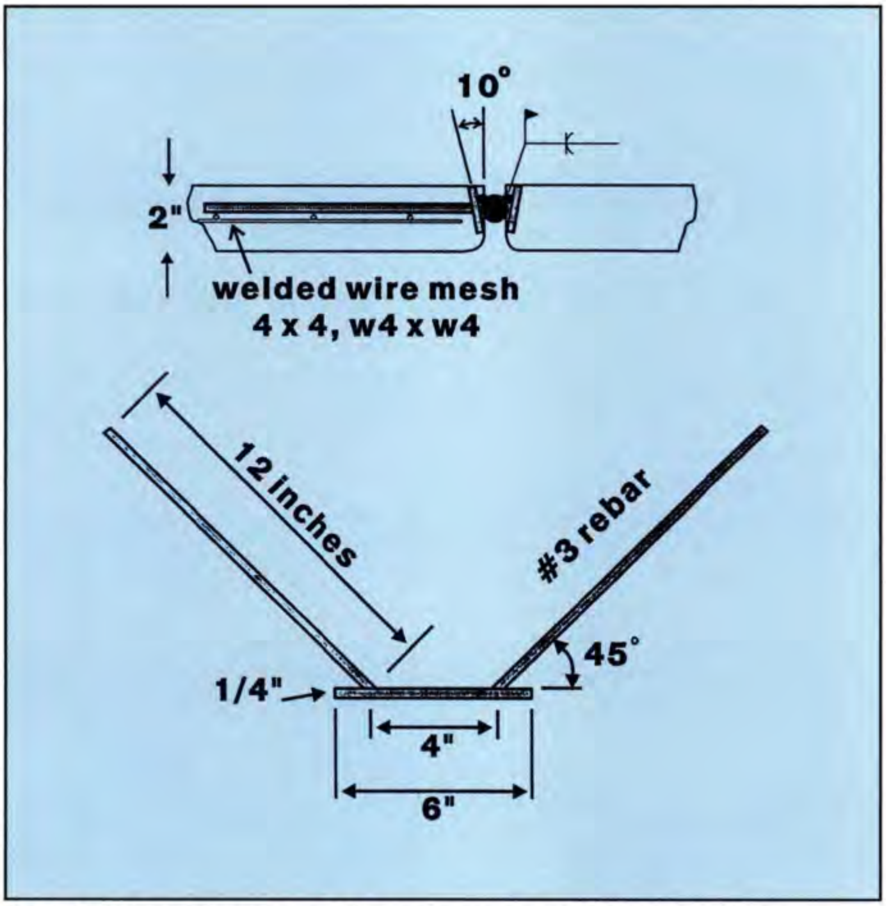

Fig. 3. Details and dimensions of the connector studied. **Note:** 1 in. = 25.4 mm.

ommended by the technical committee of the PRESSS program[14] was used unless otherwise noted.

Special attention was paid to recording visual evidence of distress during the tests as a means of establishing maximum service load capacities for use in design. Measured deformations included the relative displacement of the slabs parallel to the connector, hereafter referred to as "shear displacement," and normal to the connector, hereafter referred to as "joint opening or closing."

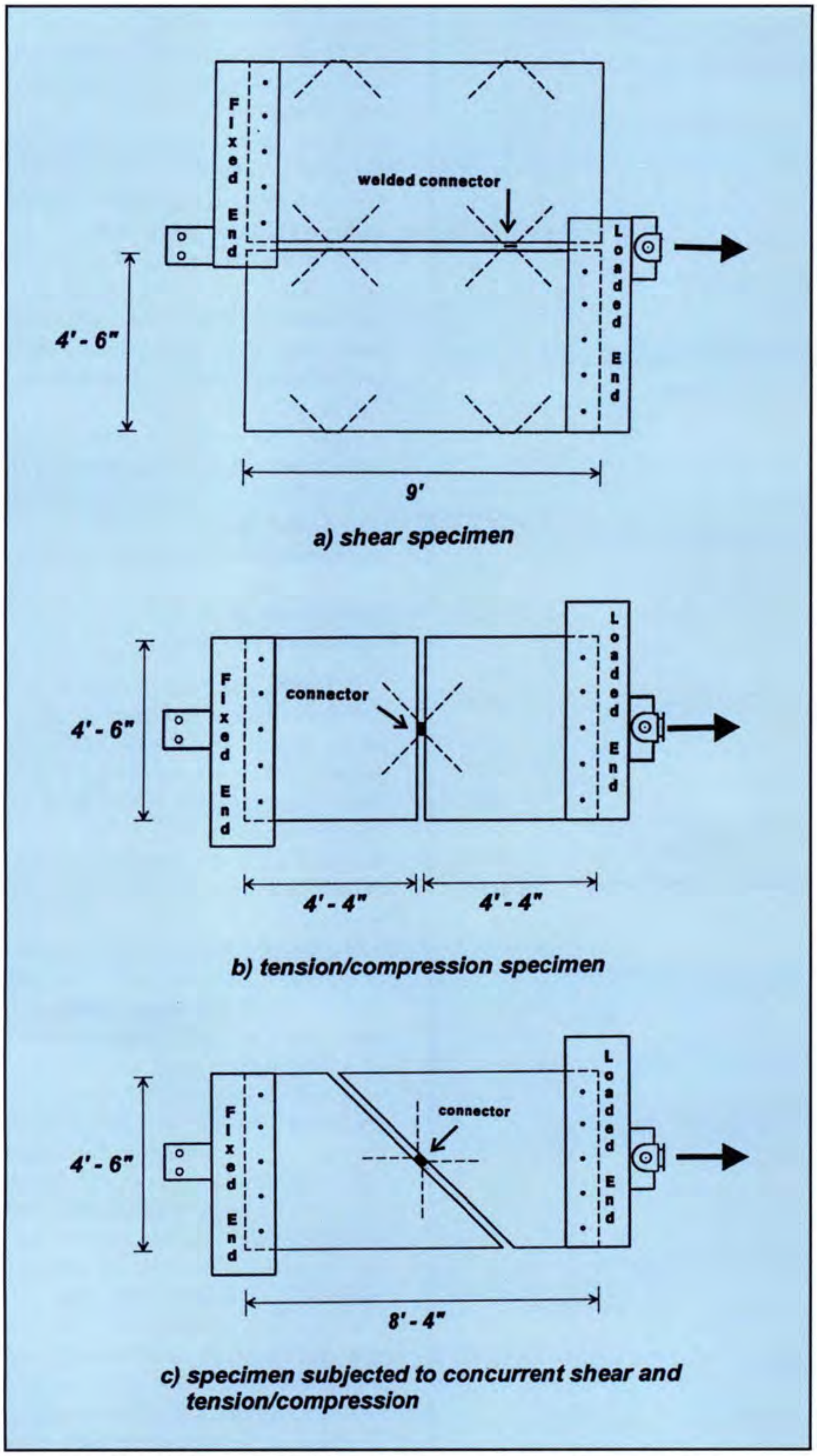

Fig. 4. Details and dimensions of the test specimens. **Note:** 1 in. = 25.4 mm.

Shear displacements were recorded with a Linear Variable Differential Transformer (LVDT) affixed to the panels over a length of 36 in. (915 mm). Similarly, joint opening (or closing) was recorded with two LVDTs symmetrically located on opposite sides of the connectors, over a length of 2 ft (610 mm), as shown in Fig. 5.

Ten tests were conducted in this pilot study. Each test specimen was designated with an alphanumeric character to describe the loading condition and test number as follows: L A #, where L corresponds to the loading condition [either monotonic loading (M) or reversed cyclic loading (C)] and A represents the type of action applied to the connector [shear (V), tension (T), or concurrent shear and tension or compression (VT)]. The symbol # indicates the test number for the given loading condition applied to the connector.

Table 1 shows details of the test program. Also shown in the table is the average concrete strength of each specimen measured at the day of testing on standard 6 x 12 in. (150 x 300 mm) cylinders. Tension tests conducted on 12 bar samples resulted in an average measured stress (based on the nominal bar diameter) of 67 and 102 ksi (462 and 704 MPa) for the yield and tensile strengths, respectively.

OBSERVED BEHAVIOR AND TEST RESULTS

The main results for all the specimens are summarized in Table 2. The results are presented in terms of the measured initial stiffness, load corresponding to the development of cracks, maximum strength, deformability and displacement ductility developed by the connector.

The stiffness values reported in Table 2 were computed from the measured load and deformation relation as the secant through the origin to a load level of 75 percent of the maximum measured strength (see Fig. 6). For specimens tested under cyclic loading, the stiffness values correspond to those recorded during the cycle in which 75 percent of the estimated strength was first applied. The value reported as strength corresponds to the maximum load applied to the specimen.

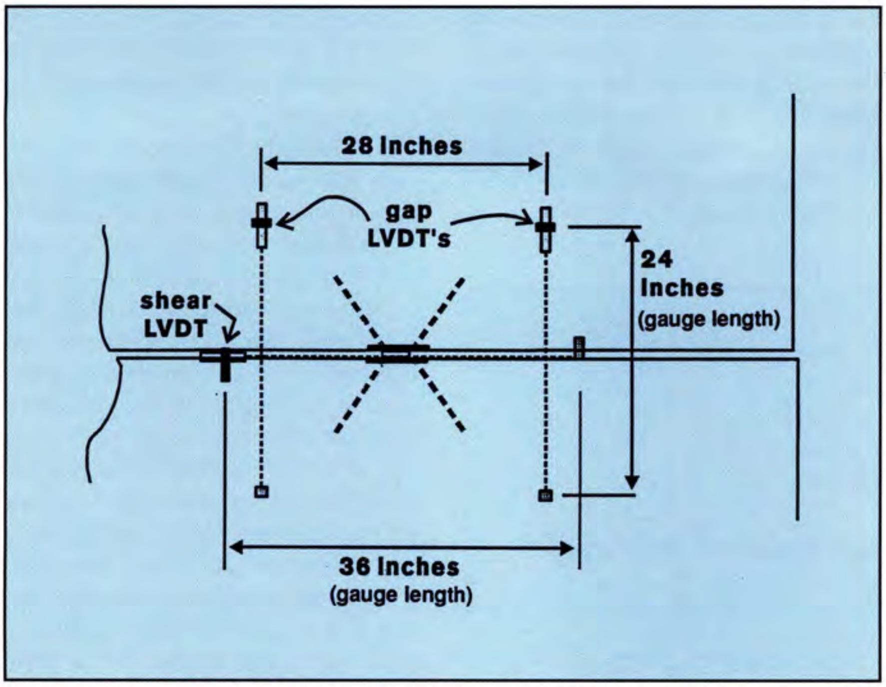

Fig. 5. Location of LVDTs to measure specimen deformations.
Note: 1 in. = 25.4 mm.

Table 1. Specimen materials and details.

Load condition		Specimen designation	f_c' (ksi)
Shear	Monotonic	MV1 MV2 MV3 MV4	6.09 6.22 5.50 6.35
	Cyclic	CV1	6.20
Tension	Monotonic	MT1	5.88
Alternating tension and compression	Cyclic	CT1 CT2	6.11 5.89
Shear and tension	Monotonic	MVT1	6.47
Shear and alternating tension or compression	Cyclic	CVT1	6.42

Note: 1 ksi = 6.895 MPa.

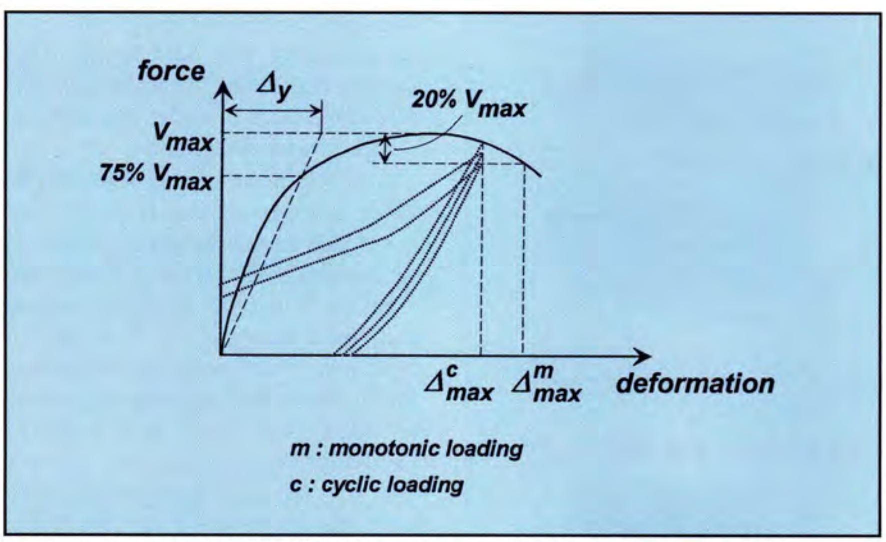

Fig. 6. Definition of yield and maximum deformation for the test specimens.

The deformability of the connector, Δ_{max}, under monotonic loading was calculated as the measured deformation corresponding to a strength decay of 20 percent of the maximum recorded strength (see Fig. 6). Under cyclic loading, the reported value is based on a 20 percent strength loss after completion of three cycles to the same deformation level (see Fig. 6).

The displacement ductility developed by the connector, μ_m, was calculated as the ratio between the connector deformation, defined above, and the yield deformation. The latter was estimated from the measured response under monotonic loading and is defined by the intersection of a horizontal line that passes through the point of maximum strength and the secant through the point corresponding to 75 percent of the maximum strength (see Fig. 6).

Finally, the observed mode of failure is briefly described in the last column of Table 2. The main characteristics of the observed response are summarized in the following sections.

In-Plane Shear — Monotonic Loading

In Fig. 7, the measured shear load and displacement responses of Specimens MV2, MV3 and MV4 are shown. The response of Specimen MV1 is not included in the figure because shear deformations were recorded using a different instrumentation layout that resulted in dissimilar readings.

The initial behavior up to the point of maximum shear strength was very similar for all specimens. For Specimens MV1 and MV3, the appearance of cracks along the bars at a shear load of 9 and 10 kips (40 and 44 kN), respectively, was the first indication of distress of the connector. Specimens MV2 and MV4, however, developed short cracks along the bars and near the edge of the steel plate before testing because of thermal expansion caused during welding of the slug.

As the load was applied to the last two specimens, the cracks propagated along the length of the bars and through the thickness of the slabs. Cracking in the region where the edge of the steel plate bears against the con-

Table 2. Summary of test results.

Load condition		Specimen designation			Stiffness (kips/in.)	Bond-splitting crack strength (kips)	Strength (kips)	Δ_{max} (in.)	μ_m	Failure mode
Shear	Monotonic	MV1			—	10.0	15.8	—	—	Bar fracture
		MV2			540	6.0[W]	15.4	0.15	5.3	Bar fracture
		MV3			390	9.0	17.1	0.15	3.4	Bar-to-plate weld fracture
		MV4			420	3.0[W]	14.3	0.16	4.7	Bar fracture
	Cyclic	CV1			360	11.8	16.8	0.06	1.5	Bar fracture
Tension	Monotonic	MT1			210	10.0	14.6	0.32	4.6	Bar fracture
Alternating tension and compression		CT1		T	200	10.6	10.9	0.22	3.0	Partial fracture of slug-to-plate weld
				C	—	—	-10.9	—	—	
		CT2		T	330	10.9[W]	14.2	0.29	5.8	Bar fracture
				C	—	—	-50.0	—	—	
Shear and tension	Monotonic	MVT1		S	300	6.7[W]	8.5	0.13	4.6	Bar fracture
				N	330	—	8.5	0.16	6.5	
Shear and alternating tension or compression		CVT1	S	T	320	4.2[W]	7.6	0.13	4.7	Bar fracture
				C	870	—	-29.3	-0.05	1.5	
			N	T	535	—	7.6	—	—	
				C	—	—	-29.3	—	—	

Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.
S – component parallel to slab edge (shear)
N – component normal to slab edge (tension or compression)
T – tension
C – compression
W – these specimens developed short cracks after welding of the slug. The value reported in the table is the load corresponding to the propagation of existing cracks.

crete was also observed. As a result of these events, the stiffness of the connectors was gradually reduced with increasing shear deformations.

The average measured shear strength for the connector was 15.5 kips (69 kN) and was always accompanied by spalling of the concrete at the bearing plate edge, as shown in Fig. 8. Immediately after reaching the maximum strength, all specimens displayed gradual strength decay upon a further increase in deformations (see Fig. 7).

Subsequent behavior, however, varied for each specimen. For Specimen MV3, an abrupt and almost total loss of strength occurred due to fracture of a bar-to-plate weld at a displacement of about 0.15 in. (3.8 mm). A similar sequence of events was observed for Specimen MV1 (not shown in Fig. 7), where bar fracture occurred just outside the weld.

In contrast, Specimens MV2 and MV4 showed almost constant strength degradation with increasing deformation until a displacement of about 0.3 in. (7.6 mm). Although not apparent during the test, rupture of one or more bars also occurred in the last two specimens, as was evident by removal of the concrete after the test.

In-Plane Shear — Reversed Cyclic Loading

Only one specimen was tested under reversed cyclic loading (see Table 1). Initial loading of this specimen was done for one full cycle to a load level of 50 percent of the strength of the connector. This strength was estimated from the measured response under monotonic loading. After this loading cycle, minor cracking adjacent to the edge of the steel plate in one slab was observed.

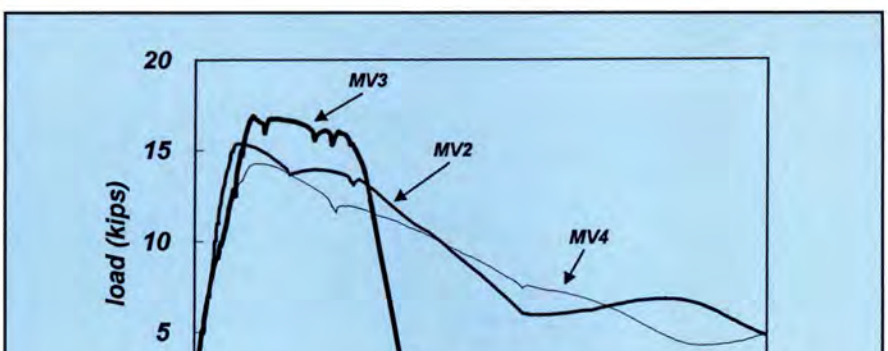

Fig. 7. Load-deformation response of connector subjected to monotonic shear loading. **Note:** 1 in. = 25.4 mm; 1 kip = 4.448 kN.

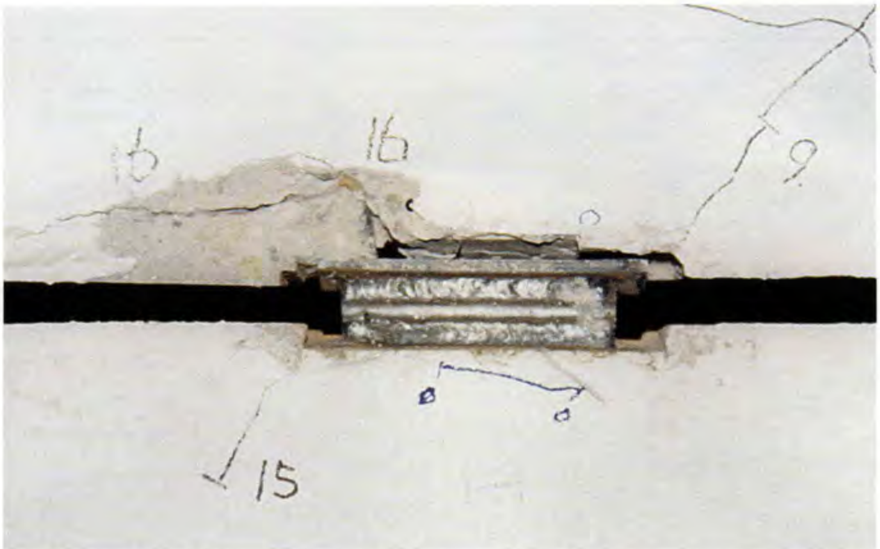

Fig. 8. Specimen MV3 after the test. Note concrete spalling in the region adjacent to the edge of steel plate in one slab where compression bearing stresses were high.

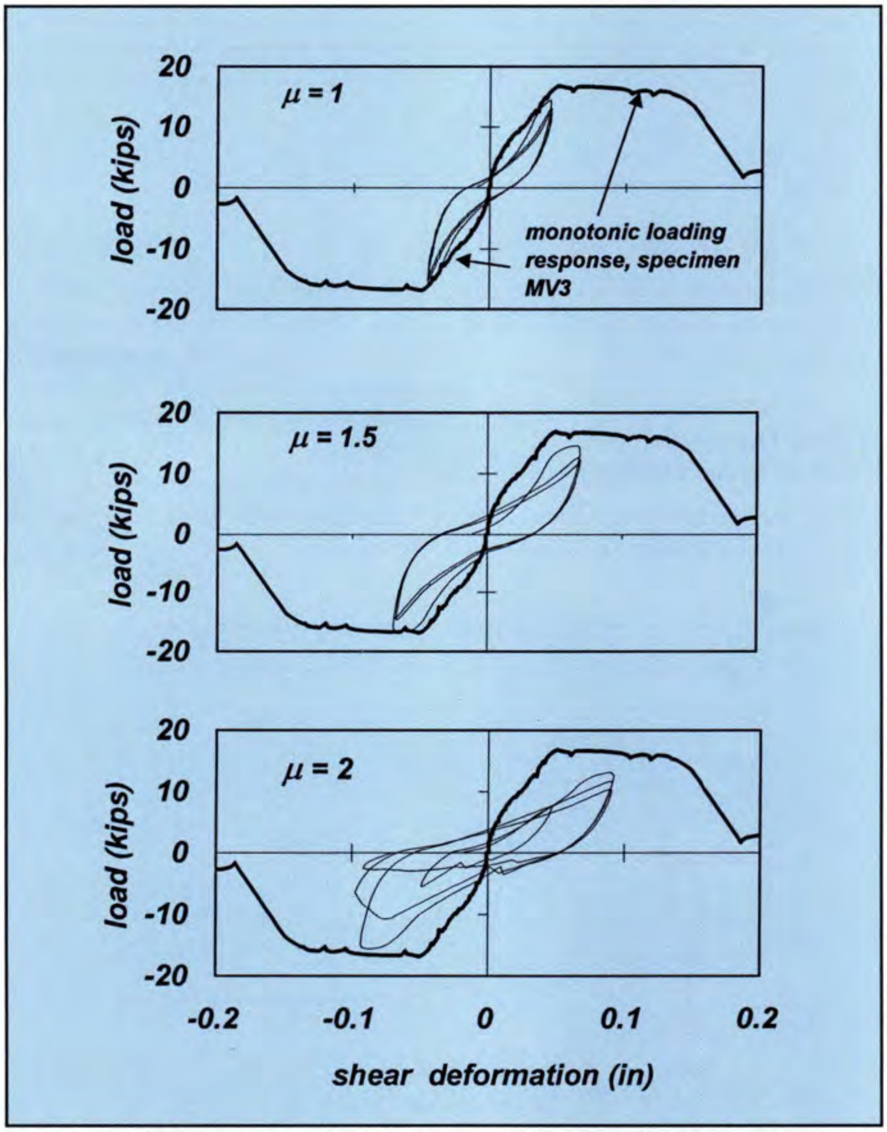

Fig. 9. Load-displacement response at selected displacement ductilities for Specimen CV1 subjected to reversed cycles of shear force. **Note:** 1 in. = 25.4 mm; 1 kip = 4.448 kN.

During the subsequent cycle to a load level of 11.8 kips (52 kN) or 75 percent of the estimated strength, cracks developed along the bars carrying tension. Further loading of the specimen to the target yield displacement level caused propagation of these cracks and concrete spalling next to the steel plate bearing areas in both slabs.

The measured response at selected levels of displacement ductility is presented in Fig. 9. For comparison, the response measured for Specimen MV3 under monotonic loading is reproduced in the figure. Until a displacement ductility of 1.5, the response of the connector was characterized by moderate pinching of the hysteresis loops and a strength loss of 20 percent or less after three cycles to the same amplitude level.

During the second cycle, corresponding to a target displacement ductility of two, significant strength and stiffness deterioration were observed and, by the end of the third cycle, the connector had lost most of its strength in the negative direction. This behavior was due to the fracture of one of the bars carrying tension and extensive concrete spalling in the connector region. In the opposite direction of loading, the connector did not show the same level of degradation.

Tension — Monotonic Loading

Fig. 10 shows the measured load and displacement response of the connector under monotonic tension loading (Specimen MT1). In this figure, the displacement shown corresponds to the average joint opening recorded by the two LVDTs mounted on the slabs (see Fig. 5). The first sign of connector distress observed in this test was the separation of the steel plates from the slab edges at a load of about 4.5 kips (20 kN).

A further increase in load caused outward bending of the plates between the bar attachment points at 8 kips (36 kN), and cracks along the four anchor bars on the bottom surface of the slabs at 10 kips (44 kN). At this load level, the connector showed evident signs of nonlinear behavior, probably due to yielding of one or more bars.

At a load of about 12 kips (53 kN), two cracks (one on each slab) perpen-

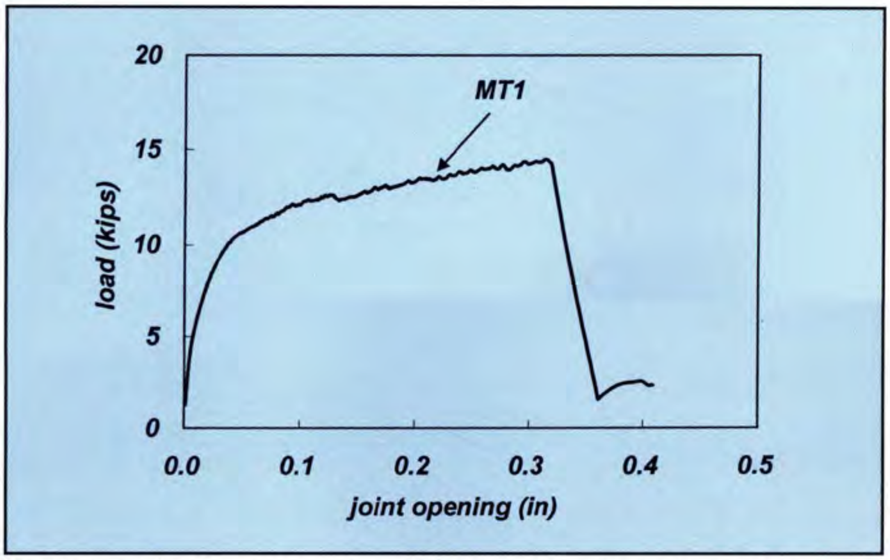

Fig. 10. Load-displacement response of Specimen MT1 subjected to monotonic loading in tension. **Note:** 1 in. = 25.4 mm; 1 kip = 4.448 kN.

dicular to the applied load appeared at about 2 in. (13 mm) from the steel plates and extended approximately 5 in. (130 mm). These cracks propagated with a further increase in the imposed displacement and resulted in concrete spalling in the plate region. The existing cracks along the length of the bars also propagated.

During this stage, the connector showed a moderate increase in strength until the sudden drop in load at a joint opening of about 0.32 in. (8 mm). Removal of the concrete after the test revealed that fracture of one bar near the weld had caused the abrupt failure of the connector. Fig. 11 shows a close-up view of the connector region after the test.

Alternating Tension and Compression

Two specimens were tested under simulated reversals of cyclic tension and compression loading. Because the strength and stiffness of the connector in compression were expected to be much larger than those in tension, a symmetrical ductility based loading history could not be used in this case. For the first specimen, CT1, the loading history for cycles in tension was the same as that described earlier, e.g., the specimen was loaded to specified increments of displacement ductility. In compression, however, it was decided to apply the same force level reached during the current cycle in tension.

For the second specimen, CT2, the same loading history was used for cycles in tension, but for cycles in compression, a more severe loading pattern was applied. In this test, the maximum compressive load was about five times the yield strength of the connector in tension. This load level was only exploratory and was selected arbitrarily based on the results from the previous specimen where the compression load level applied did not appear to affect the response of the connector.

Despite the significant difference in the maximum applied compression load, both specimens showed essentially the same response in tension well into the nonlinear range. For the first specimen (CT1), however, the full deformation capacity of the connector could not be realized due to a partial weld fracture between the slug and the plate. Here, the behavior of only Specimen CT2 will be discussed in further detail.

In Fig. 12, the measured load and joint opening (or closing) response of Specimen CT2 at selected displacement ductilities is presented. Note that the ductility values reported in the plots are based on the deformations measured for loading cycles in tension. Also shown in this figure is the response measured under monotonic loading in tension of Specimen MT1 for comparison.

Initial behavior of the connector in tension was essentially the same as that observed under monotonic loading. The propagation of cracks along the four reinforcing bars at a tension load of 10 kips (44 kN) was the only apparent damage up to the target yield displacement. Note that this specimen also had initial cracks due to welding of the slug (see Table 2).

Upon further loading, new cracks next to the steel plates and perpendicular to the applied load appeared during cycles in tension, while the cracks along the bars stopped propagating.

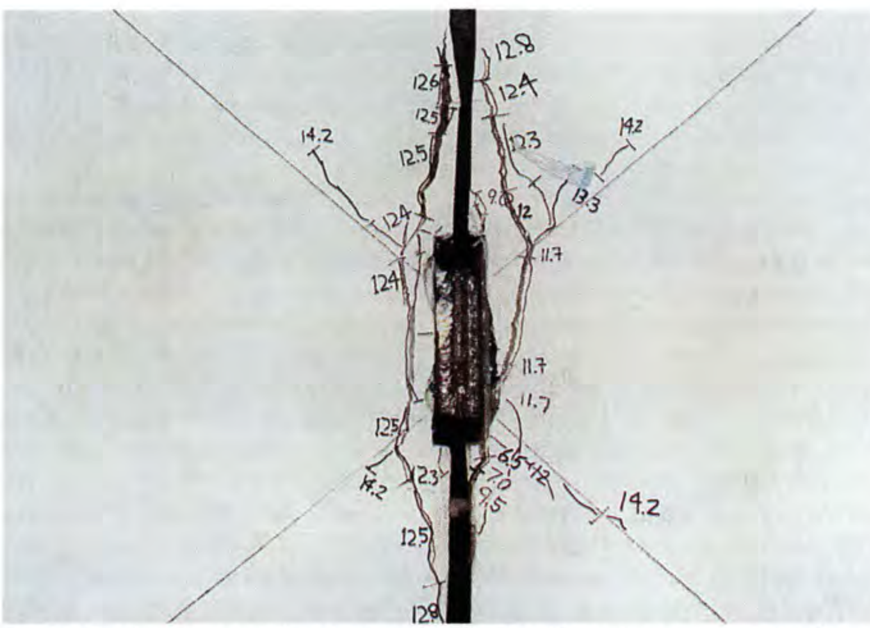

Fig. 11. Crack pattern at failure for Specimen MT1 subjected to monotonic loading in tension.

These new cracks lengthened and widened with further load reversals, and eventually led to deterioration of the concrete in the connector region.

The high compression load and the damage induced in the connector region resulted in marked pinching of the hysteresis loops. Peak strength, however, was maintained in both directions of loading until the target displacement ductility of six (see Fig. 12). At this stage, bar fracture near the weld suddenly occurred during the second loading cycle in tension causing a complete loss of strength. Note that the load and deformation relation obtained under monotonic loading provides an approximate envelope to the peak tensile strength measured in each cycle.

In-Plane Shear Combined with Tension or Compression

Two specimens were tested under concurrent shear and tension or compression. The first specimen (MVT1) was subjected to monotonically increasing loading under shear and tension, while the second (CVT1) was tested under reversed cycles of shear combined with alternating tension or compression. In both specimens, a constant shear-to-tension (or compression) force ratio of one was maintained during the tests.

Monotonic loading — Fig. 13 illustrates the measured load and deformation response for Specimen MVT1. In this figure, the deformation components of shear and tension are presented in separate plots. Points A, B, and C in the plots illustrate corresponding stages of deformation in shear and tension. The measured responses for Specimen MV3 tested under pure shear and for Specimen MT1 tested under pure tension are reproduced in the figure for comparison.

Overall, the load resisting mechanisms and sequence of events displayed by Specimen MVT1 were similar to those observed under either pure shear or tension. In this specimen, short cracks on one slab as well two cracks (one on each slab) parallel and beside the steel plates appeared after welding of the steel slug. At a net shear or tension force of 6.7 kips (30 kN), new cracks perpendicular to the

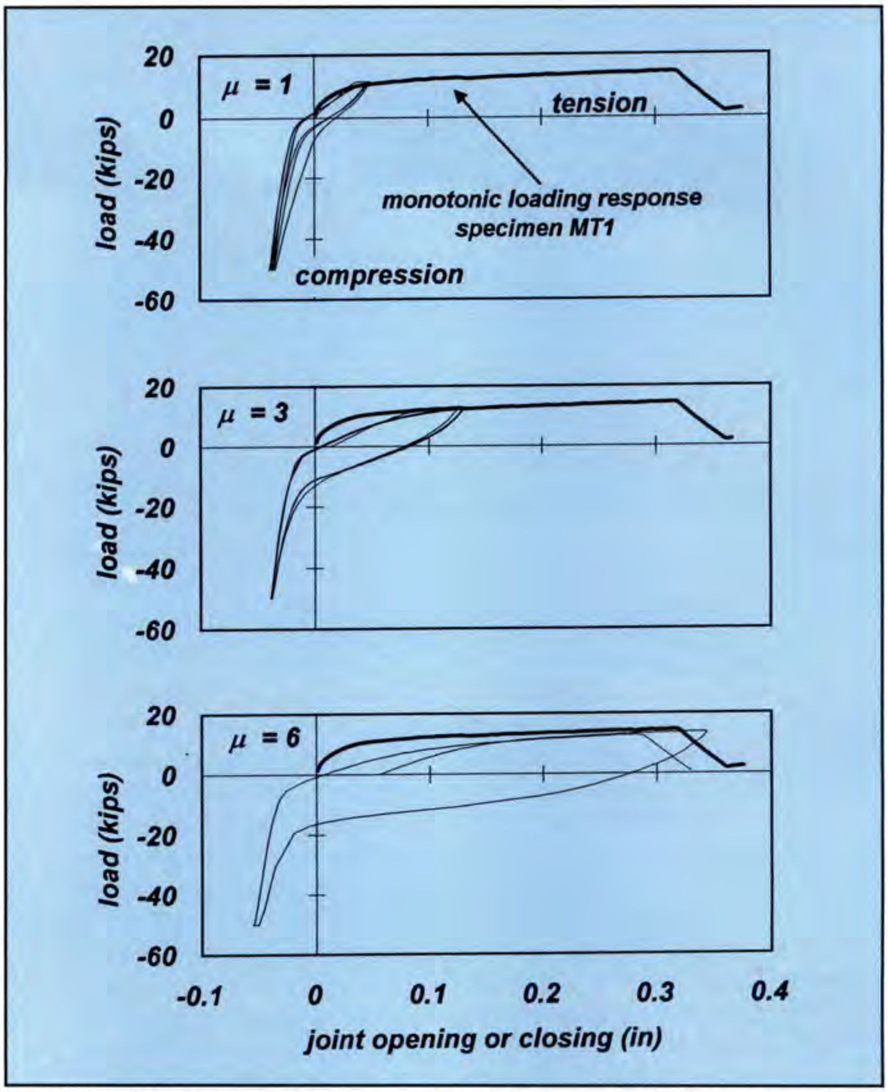

Fig. 12. Load-displacement response at selected displacement ductilities for Specimen CT2 subjected to cycles of alternating tension and compression.
Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

applied load appeared in both slabs just beyond the end of the anchor bars. These cracks were about 12 to 15 in. (305 to 380 mm) long, as shown in Fig. 14.

Peak strength was accompanied by concrete spalling, a behavior similar to that observed for the specimens tested under pure shear. Failure of the connector occurred abruptly at a shear displacement of approximately 0.12 in. (3 mm) due to the fracture of one of the anchor bars. The maximum measured shear or tension load was 8.5 kips (38 kN) or about half the measured strength under either pure shear or tension for this connector. The shear deformation corresponding to bar fracture for Specimen MVT1 (Point C in Fig. 13) was comparable to that of Specimen MV3 subjected to shear only. In contrast, joint opening was considerably lower than that recorded for the connector under tension only.

Reversed cyclic shear combined with alternating tension or compression — The loading history used in this test followed the pattern recommended by the PRESSS program[14] by applying equal increments of shear displacement in both directions of loading. To define the shear displacement increments at the recommended ductility levels, the yield deformation estimated from the results of the monotonic loading test under shear and tension were used. This specimen also showed minor cracking adjacent to the steel plate and short cracks along the bars after welding of the steel slug.

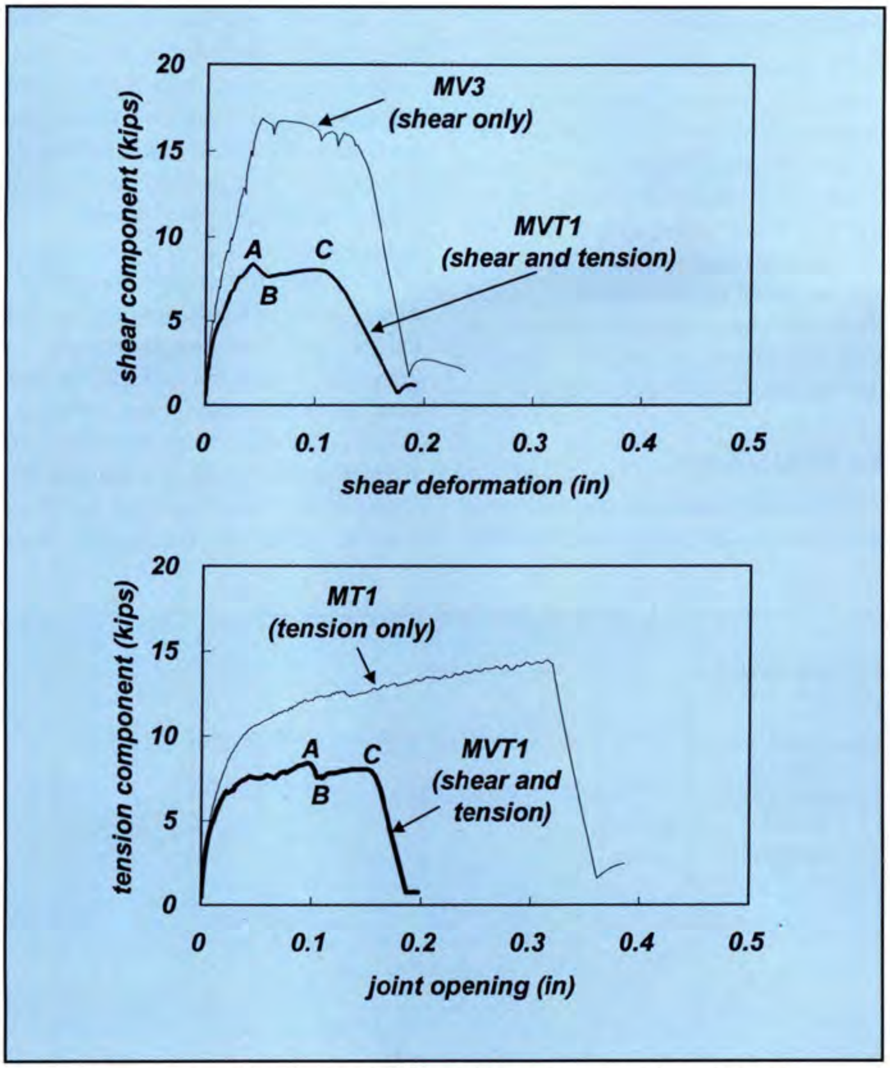

Fig. 13. Comparison of the load and deformation responses under monotonic loading for: Specimen MVT1 subjected to concurrent shear and tension; Specimen MV3 subjected to shear only; and Specimen MT1 subjected to tension only. **Note:** 1 in. = 25.4 mm; 1 kip = 4.448 kN.

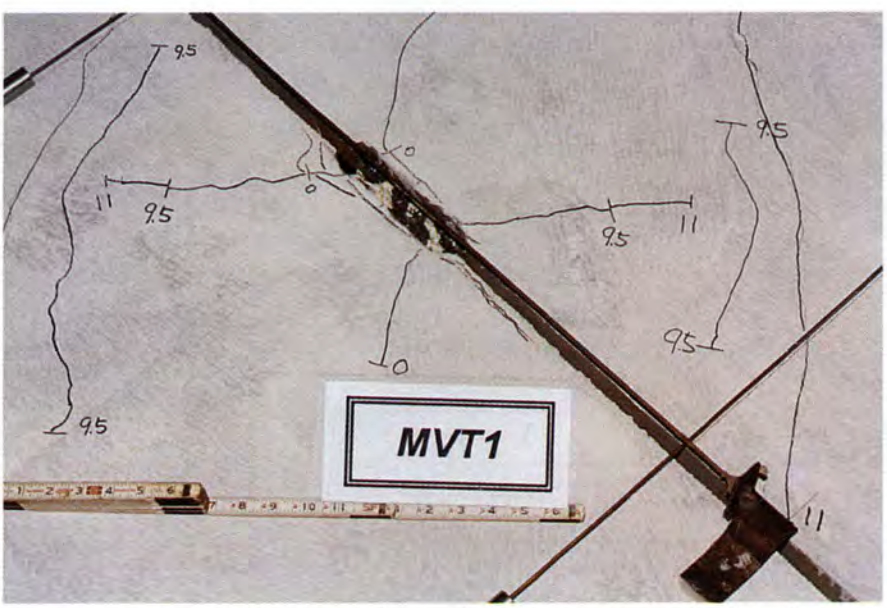

Fig. 14. Crack pattern of Specimen MVT1 subjected to monotonic loading of combined shear and tension.

The measured response for this specimen (CVT1) at selected displacement ductilities is presented in Fig. 15 in terms of the shear load and deformation component. The measured response under monotonic loading is also shown in the figure. Under shear and tension, new cracks along the anchor bars appeared at a shear load of about 6 kips (27 kN). In the opposite direction of loading, under shear and compression, the response of the connector remained linear.

Loading to a target displacement ductility of one caused no appreciable reduction in the load-carrying capacity under shear and tension (see Fig. 15). Under shear and compression, however, the peak load reached during the second cycle was only 70 percent of that in the first cycle. This loss of strength, also accompanied by stiffness decay, is attributed to concrete crushing and spalling in front of the edge of the steel plate, the region of highest compressive stresses. Despite this event, the connector showed stable hysteretic response during the third cycle. A similar behavior was observed during subsequent loading cycles to displacement ductilities of 1.5, 2 and 3, though strength reductions of about 7 percent of the peak load were observed under shear and tension.

At a target displacement ductility level of four, the strength under shear and compression dropped with every loading cycle and reached a shear load of only 10 kips (44 kN) during the third cycle (see Fig. 15). Strength decay under shear and tension, however, continued to be modest at this stage (10 percent of the maximum strength). Close inspection of the connector revealed that one of the bars had fractured near the weld. This result suggests that the strength under shear and tension was primarily provided by the tension bars alone as the contributions from concrete bearing and shear friction had probably been eroded under repeated cycles of shear and compression.

Loading of the specimen to a target ductility of five caused strength decay in shear and tension and under shear and compression. Repeated cycles at this ductility level caused fracture of another bar and significant loss of strength in both directions of loading (see Fig. 15). The test was stopped during the subsequent loading cycle when the connector showed almost no residual capacity.

DISCUSSION OF TEST RESULTS

Although the measured strength varied depending on the load condition considered, several common load-resisting mechanisms could be identified. In the following sections, the interaction between these mechanisms and the observed failure mode is discussed.

Anchorage of Bars

All specimens developed cracks along the reinforcing bars. Despite the appearance of these cracks, which extended the full length of the bar in some cases, all specimens failed by fracture of one or more bars. These results indicate that the provided anchorage length of 12 in. (305 mm), which meets the non-seismic provisions of ACI 318-95,[15] is sufficient to develop the full strength of the No. 3 bars.

Bar Weld Strength

Only one specimen showed evidence of bar-to-plate weld fracture after the test, namely, Specimen MV3 tested under monotonic shear load. Despite this event, this specimen had the highest strength and reached a maximum deformation comparable to that of the rest of the specimens tested under monotonic shear loading (see Table 2).

To further investigate the performance of these welds, four samples of bars welded to a plate were tested in air under monotonic loading in tension. These samples were manufactured using the same materials and specifications used to fabricate the connectors. Three samples failed by fracture of the bar. One sample frac-

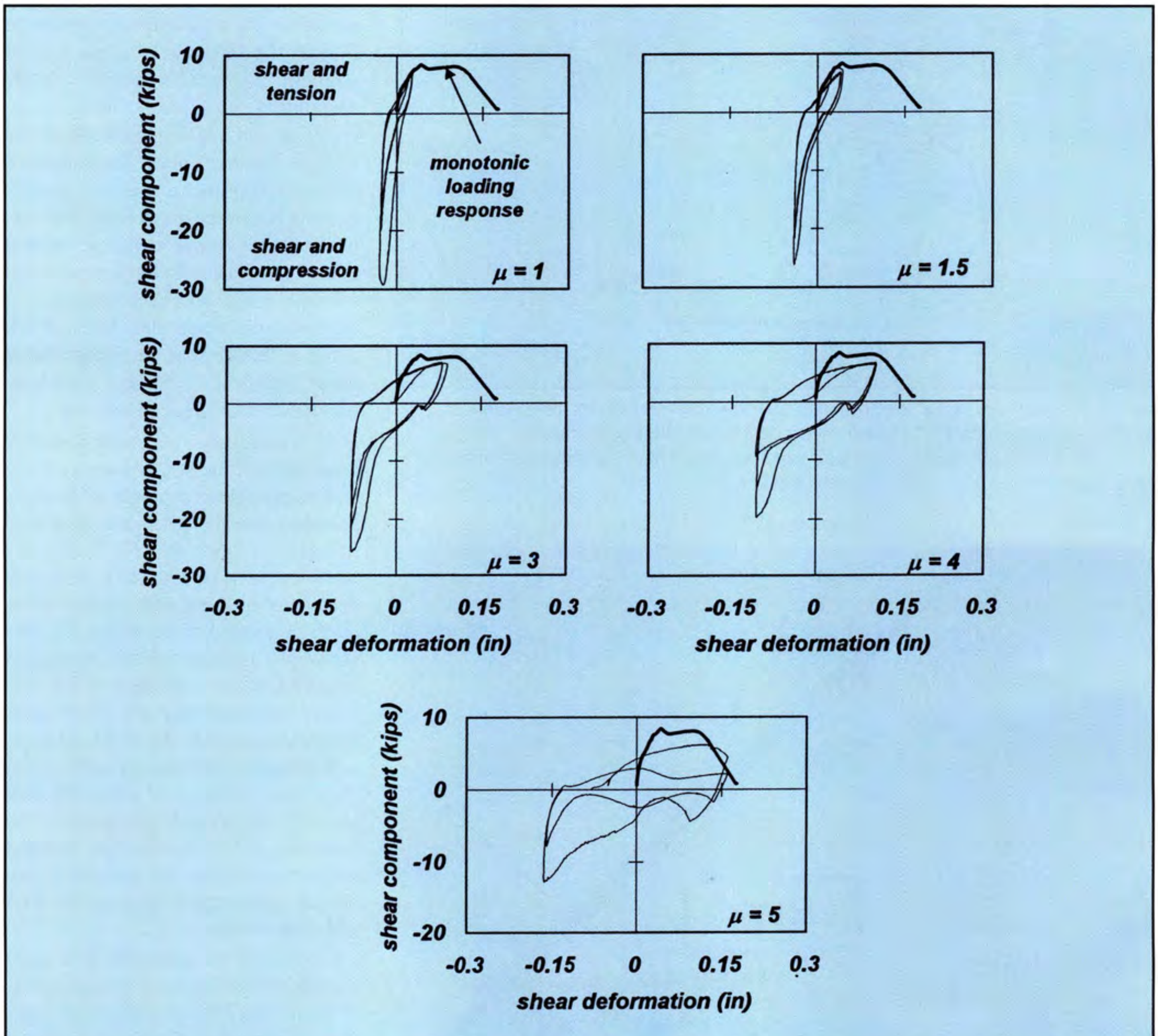

Fig. 15. Load-displacement response at selected displacement ductilities for Specimen CVT1 subjected to reversed cycles of shear with tension or compression. **Note:** 1 in. = 25.4 mm; 1 kip = 4.448 kN.

tured at the weld. The average tensile strength was 10.9 kips (48 kN) or 97 percent of the measured strength of the bars alone. It must be noted that the sample that failed by weld fracture reached yielding of the bar and attained a strength of 10.9 kips (48 kN), the same as the average strength.

Clearly, additional tests are needed to provide a statistically reliable measure of the performance of these welds. The results of the bar tests and the bar fractures observed for the connectors embedded in the slab specimens suggest that fillet welding combined with preheating of the reinforcing bars per AWS D1.4[12] is adequate to develop the strength of the bars.

Load-Resisting Mechanisms

In-plane shear — The observed resistance mechanisms of the connectors subjected to pure shear were loadbearing between the edge of the steel plate and the surrounding concrete and axial forces in the anchor bars. Flexure and shear of the bars may have also contributed to the strength, particularly at large displacements, as was evidenced by bending of the bars that did not fracture and were still attached to the steel plate after the test. Friction between the steel plate and the concrete may have developed as well. These mechanisms are illustrated in Fig. 16.

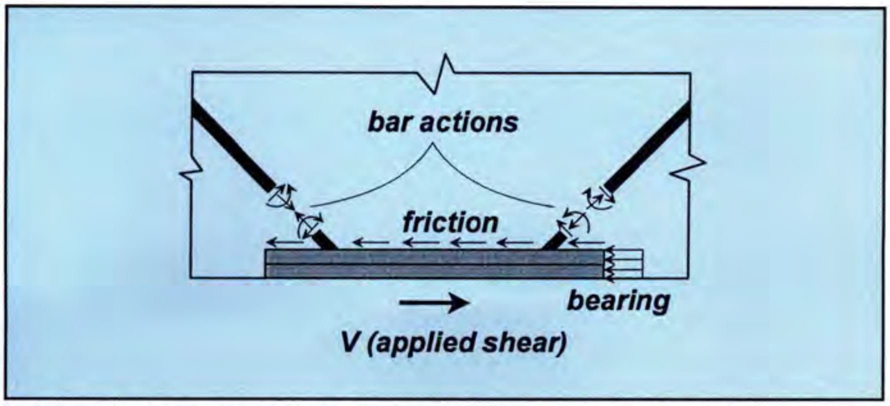

Fig. 16. Probable resisting mechanisms of the connector subjected to shear loading.

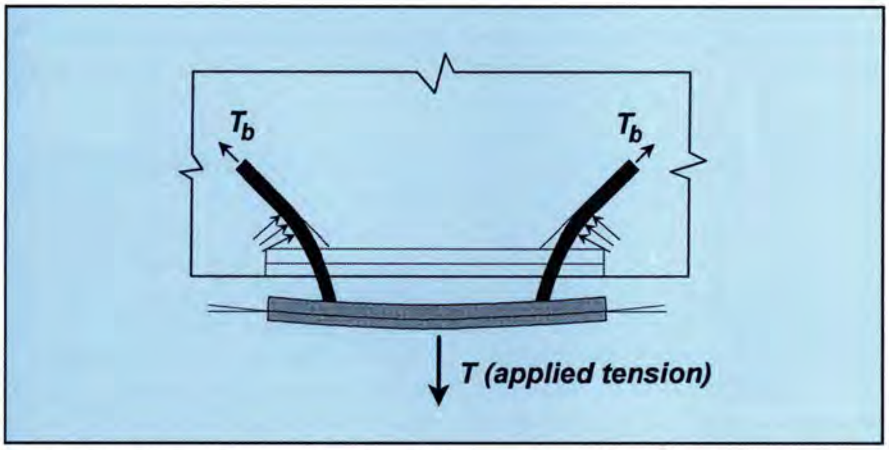

Fig. 17. Probable resisting mechanisms of the connector subjected to tension loading.

The relative contribution to strength of each mechanism was not quantified in this study. Based on the test results and visual inspection of the specimens during the tests, however, it is believed that plate bearing was the main source of resistance for the connector at low load levels. As the load increased and cracking in the connector region began to develop, bar resistance was probably mobilized.

The maximum shear force was reached when the bearing capacity began to degrade, as became evident by spalling of the concrete in front of the edge of the steel plate. From this stage, it is believed that the shear resistance decreased gradually as the load carried by bearing was progressively shared by the anchor bars. Eventually, failure of the connector would occur by fracture of one or more reinforcing bars.

Tension only and alternating tension and compression — The resistance mechanism of the connectors in tension was mainly provided by tension in the bars with bending of the steel plate as the joint opened. Additional resistance may have been provided by bar bending and by bar bearing against the concrete as the steel plate pulled away from the concrete flange, as shown in Fig. 17. Alternate cycles of tension and compression did not affect the failure mode nor the load transfer mechanisms. The strength and deformation capacity under cyclic loading were almost the same as those under monotonic tension loading (see Fig. 12).

For cycles in compression, the load resistance was provided by axial compression forces in the bars and bending of the steel plate until the joint closed. Afterwards, plate bearing against the edge of the slab provided the main source of stiffness and strength. This behavior is believed to have caused the marked pinching of the loops observed in the load-deformation response upon reloading during a compression cycle (see Fig. 12).

Combined shear and tension (or compression) — The measured response for Specimens MVT1 and CVT1 showed that the shear strength of the connectors was strongly influenced by the presence of a tension force, or vice versa (see Fig. 13). For the shear-to-tension ratio of one used in the tests, the shear strength was reduced to one-half of that recorded under pure shear. Under cyclic loading, this strength was reduced only 10 percent, despite extensive concrete spalling around the connector region. These results suggest that the strength under shear and tension was mostly provided by the anchor bars with a small contribution (10 percent or less) from bearing of the steel plate edge against the concrete.

The shear strength under shear and compression was probably provided by plate bearing, combined actions in the bars, and friction between the steel plate and the concrete. Unlike the response of the specimens subjected to

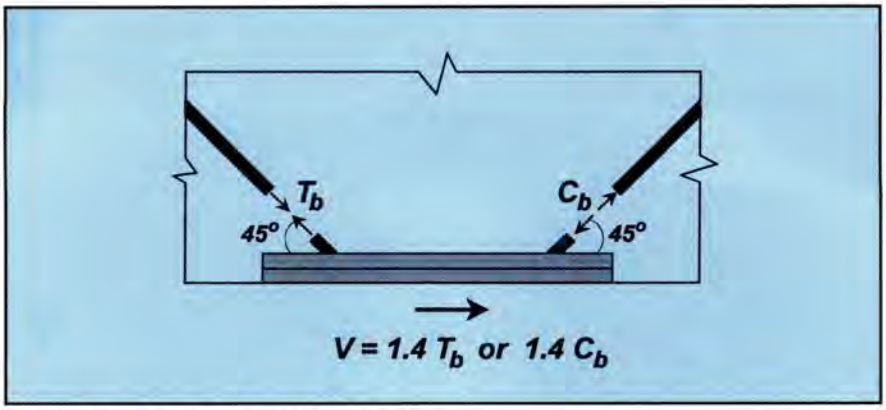

Fig. 18. Simplified truss model used in design.

shear only, the contribution to shear strength of the friction mechanism was greatly increased when the compression force was present. This behavior became apparent as the measured shear strength under concurrent shear and compression was about twice as high as that measured under shear alone (see Table 2).

Peak shear strength was reached when the concrete in front of the edge of the steel plate began to spall off, in a manner similar to that observed for the specimens tested under pure shear. Following this event, it is plausible that friction combined with actions in the bars gradually became the main source of resistance.

DESIGN IMPLICATIONS

This section describes the design implications associated with connector strength, connector deformability, connector stiffness and serviceability.

Connector Strength

The simplest and most widely used design method to calculate the shear strength of the connectors is to assume that the shear force is carried entirely by the anchor bars. For connectors with bars at a 45-degree angle, it is common practice[16] to employ a truss model where the load is resisted by the components of bar axial forces parallel to the applied shear (see Fig. 18).

In Table 3, the measured strength for all the specimens tested under pure shear is compared with that computed using the truss model. In this table, the design strength, V_{DESIGN}, was calculated using the nominal yield strength of the bars. As shown in Table 3, measured-to-design strength ratios ($V_{MEASURED}/V_{DESIGN}$) range from 1.53 to 1.83.

These results show that the common design procedure provides a conservative estimate of the strength of this connector in pure shear. The additional strength observed in the tests may have been provided by stresses higher than the nominal yield and other resistance mechanisms such as plate bearing and shear and flexure of the bars, which are not considered in the traditional design approach.

The truss model may also be used to estimate the resistance of the connector under pure tension. The computed measured-to-design strength ratios are presented in Table 4. As before, the design strength, T_{DESIGN}, was calculated using the nominal yield strength of the bars. Again, the results show that the design procedure yields a conservative estimate of the strength of the connector.

The observed behavior under combined shear and tension (or compression) shows a strong interaction among these actions. For the constant shear-to-tension ratio of one used in two tests, the shear strength was reduced by about 50 percent. Based on this result, it would be prudent to reduce the design shear strength whenever concurrent tension forces are anticipated in the connector.

Connector Deformability

In-plane shear — In current design practice,[16] all connectors along a joint are assumed to carry an equal portion of the shear load. Thus, a minimal deformation ductility is desirable in individual connectors to ensure that all connectors can reach their strength. The measured deformation and ductility are shown in Table 2. The adequacy of these values cannot be fully established without an appropriate estimate of expected demands.

Concurrent research is underway to identify needed levels of ductility. It is likely, however, that the moderate to high ductility values obtained under monotonic loading are sufficient to allow force redistribution to occur among the connectors within any particular flange-to-flange joint.

Under cyclic loading, the maximum deformation and corresponding ductility of 1.5 were less than half of that recorded under monotonic loading. A limited force redistribution among connectors can be expected in this case. Additional experimental evidence is necessary to verify this behavior.

Tension only and alternating tension and compression — Moderate to high ductility values were obtained under both monotonic and cyclic loading. The ductility and deformation capacity exhibited by the connector is desirable and may be essential in long diaphragms where volume changes due to creep, shrinkage or seasonal temperature variations can cause a joint to open.

Combined shear and tension (or compression) — The maximum deformation and ductility under shear and tension were comparable to those under monotonic shear loading, but smaller than those observed in tension only. Cyclic loading resulted in reduced deformation capacity and ductility (see Table 2). Additional tests are needed to corroborate these observations, but the data suggest that the deformations under shear alone represent a lower bound of the deformability of the connector under shear and tension.

Connector Stiffness

The measured stiffness (secant to 75 percent of maximum strength) of the connectors is listed in Table 2. For the specimens tested under cyclic loading, very limited or no stiffness degradation was observed prior to a load level of 75 percent of the estimated strength. Thus, the measured stiffness under monotonic and cyclic loading may be assumed to be the same below this load level.

While the stiffness values reported in Table 2 undeniably show some scatter, they do provide an indication of the initial stiffness for this type of connector under the loading conditions studied. Preliminary analytical studies at the University of Wisconsin based on the measured connector stiffness suggest that diaphragms with these types of connectors may be more flexible than originally anticipated. Accordingly, the assumption of a rigid diaphragm often used in design may be inappropriate and may need to be revised. Further experimental and analytical studies are required before more specific design recommendations can be made.

Table 3. Comparison of calculated and measured strengths of specimens tested in shear.

Load condition	Specimen	V_{DESIGN} (kips)	$\frac{V_{MEASURED}}{V_{DESIGN}}$
Monotonic	MV1 MV2 MV3 MV4	9.3	1.69 1.65 1.83 1.53
Cyclic	CV1	9.3	1.80

Note: 1 kip = 4.448 kN.

Table 4. Comparison of calculated and measured strengths of specimens tested in tension or under alternating tension and compression.

Load condition	Specimen	T_{DESIGN} (kips)	$\frac{T_{MEASURED}}{T_{DESIGN}}$
Monotonic	MT1	9.3	1.57
Cyclic	CT1 CT2	9.3	1.17* 1.53

Note: 1 kip = 4.448 kN.

* Full tensile strength was not developed for this connector due to premature fracture of slug-to-plate weld.

Serviceability

Visible concrete cracking is usually unacceptable to owners of projects with exposed structural members. In some specimens, short cracks along the bars or near the edge of the steel plate appeared after welding of the slug but before testing. These cracks were most likely the result of multiple welding passes without allowing the weld to cool. In practice, these types of cracks may be avoided by preventing overheating of the connection during welding.

In other specimens, cracks along the bars developed at a load that varied between 50 to 70 percent of the measured strength (see Table 2). These results suggest that these cracks may develop under wind loading or due to forces associated with the volume changes induced by shrinkage, creep, or temperature variations that may occur in roof diaphragms. Thus, it seems reasonable to use the load level corresponding to the development of such cracks as a service load design criterion.

CONCLUSIONS

Ten double tee flange connectors were tested under various in-plane loading conditions. Based on the observed behavior and evaluation of the test results, the following conclusions can be drawn:

1. Tests of flange connectors, used to provide design information, must include combined load types and cyclic loading. Interaction of loads may decrease strength and cyclic loading may increase deterioration and reduce deformability.

2. An anchorage length of 12 in. (305 mm) for the No. 3 reinforcing bars used in the connectors is adequate to develop the strength of the bars. The development of cracks along the bars did not appear to affect the capacity of the connector.

3. The traditional truss model used in design provided conservative estimates of the measured strength under pure shear or pure tension for both monotonic and cyclic loading.

4. The interaction between shear and tension forces can be significant. For the ratio of shear-to-tension force investigated, the strength of the connector was reduced to one-half of that under shear alone. It is felt that the strong interaction between these actions cannot be ignored and should be considered in design when such a combination of forces is anticipated.

5. The measured deformation ductility in shear is believed to be sufficient to allow force redistribution among individual connectors and to ensure that all connectors reach their strength within a given flange-to-flange joint. Similarly, the joint opening exhibited by the connector in tension appears to be adequate to accommodate the deformation demands caused by volume changes in precast concrete diaphragms.

6. Additional research is needed to better characterize the behavior of this and similar connectors under combined in-plane and out-of-plane forces. The effects of cyclic loading also need to be further examined.

CURRENT AND FUTURE TEST PROGRAM

The pilot study reported here considered a specific connector for joining adjacent flanges of double tees. Many other similar connectors are currently in commercial use and may perform as well as or better than the connector studied here. Future tests will examine different types of connectors embedded in 2 in. (50 mm) thick untopped flanges and 4 in. (100 mm) thick pretopped flanges. A vertical shear force will be applied to simulate the effects created by camber adjustment and live loading in combination with diaphragm forces. The behavior of cast-in-place concrete toppings will also be investigated.

ACKNOWLEDGMENTS

The research reported in this paper was funded by the Precast/Prestressed Concrete Institute through a Daniel P. Jenny Research Fellowship. Additional funding was provided by the National Science Foundation (NSF) through Grant CMS-9412906 as part of the Precast Seismic Structural Systems (PRESSS) program. Dr. M. P. Singh was the cognizant program official at NSF.

The technical input provided by the industry advisory panel to this project is gratefully acknowledged. Special thanks are extended to J. W. Peters & Sons, Inc. for donating the welded wire mesh used in the test specimens.

The authors also wish to thank the PCI JOURNAL reviewers for their constructive comments and suggestions. The opinions, findings, and conclusions reported in this paper are solely those of the authors and do not necessarily represent the views of the sponsors or those of the advisory panel.

REFERENCES

1. Holmes, William T., and Sommers, Peter (Editors), "Northridge Earthquake of January 17, 1994, Reconnaissance Report, Volume 2," Earthquake Engineering Research Institute, *Earthquake Spectra*, Supplement C to V. 11, January 1996.
2. Wood, S. L., Stanton, J. F., and Hawkins, N. M., "Performance of Precast Parking Garages in the Northridge Earthquake: Lessons Learned," *Building an International Community of Structural Engineers*, Proceedings of Structures Congress XIV, V. 2, American Society of Civil Engineers, New York, NY, 1996, pp. 1221-1227.
3. Fleischman, R. B., Sause, R., Pessiki, S., and Rhodes, A. B., "Seismic Behavior of Precast Parking Structure Diaphragms," PCI JOURNAL, V. 43, No. 1, January-February 1998, pp. 38-53.
4. Venuti, J. W., and Nazarian, D., "Diaphragm Shear Connectors Between Flanges of Prestressed Concrete T-Beams," Report No. ST-0007-68, San Jose State College, San Jose, CA, 1968.
5. Aswad, A., "Selected Precast Connections: Low Cycle Behavior and Strength," Proceedings of the Second U.S. National Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Stanford, CA, August 22-24, 1979.
6. Concrete Technology Corporation, In-house Technical Report, Tacoma, WA, 1974.
7. Stanton, J. F., Anderson, G. R., Dolan, C. W., and McCleary, D. E., "Moment Resistant Connections and Simple Connections," Research Project 1/4, Precast/Prestressed Concrete Institute, Chicago, IL, 1986.
8. Kallros, M. K., "An Experimental Investigation of the Behavior of Connections in Thin Precast Concrete Panels Under Earthquake Loading," M.S. Thesis, Civil Engineering Department, University of British Columbia, British Columbia, Canada, April 1987.
9. Spencer, R., "Earthquake Resistant Connections for Low Rise Precast Concrete Buildings," Seminar on Precast Concrete Construction in Seismic Zones, V. 1, NSF/Japan Society for the Promotion of Science, October 1986.
10. ASTM, "Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement (A 615/A 615M-96a)," *1997 Annual Book of ASTM Standards*, V. 01.04, American Society for Testing and Materials, Philadelphia, PA, 1997.
11. ASTM, "Standard Specification for Carbon Structural Steel (A 36/A 36M)," *1997 Annual Book of ASTM Standards*, V. 01.04, American Society for Testing and Materials, Philadelphia, PA, 1997.
12. AWS, "Structural Welding Code-Reinforcing Steel," ANSI/AWS D1.4-92, American Welding Society, Miami, FL, March 1992.
13. Kusumo-Rahardjo, F. I., "Behavior and Design of Connectors for Precast Double-Tee Members," M.S. Independent Study Report, University of Wisconsin, Madison, WI, May 1996.
14. Priestley, M. J. N., "The U.S.-PRESSS Program Progress Report," Third Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Precast Seismic Structural Systems (JTCC-PRESSS), San Diego, CA, November 18-20, 1992.
15. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)," American Concrete Institute, Farmington Hills, MI, 1995.
16. *PCI Design Handbook*, Fourth Edition, Precast/Prestressed Concrete Institute, Chicago, IL, 1992.