Behavior of Double-Tee Flange Connectors Subjected to In-Plane Monotonic and Reversed Cyclic Loads

José A. Pincheira, Ph.D.
Associate 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

Wei Zheng, Ph.D., P.E.
Assistant Professor
Department of Civil Engineering
Jackson State University
Jackson, Mississippi

The results of an experimental investigation to study the behavior of seven mechanical connectors used in 2 and 4 in. (51 and 102 mm) thick double-tee flanges are presented. The study included five connectors commonly used in practice and two new or modified designs. A total of 95 connector tests were conducted under a variety of loading and support conditions with both monotonic and cyclic reversing in-plane shear. The data show that the strength, stiffness, and deformation capacity of connectors vary widely depending on the constraint and loading conditions, which makes analytical prediction of behavior very difficult. Based on the measured response and observed behavior, the performance of each connector is discussed based on serviceability, failure mode, and deformation capacity. Two connectors, a bent wing made of a bent strap of steel and a Vector®, provided the most dependable behavior in terms of strength and deformation capacity.

Double tee girders are widely used as floor systems by the precast/prestressed concrete industry in North America. To form a floor system, the double tees are often joined with mechanical connectors spaced at 4 to 8 ft (1.22 to 2.44 m) along the flanges (see Fig. 1). The main purpose of these connectors is to resist the horizontal shear forces from lateral loads (wind or earthquake), or vertical shear from gravity loads and differential camber adjustment, while also withstanding volume change-induced forces.

The force-resisting functions of all flange-type connectors are basically the same, but their individual details and resistance mechanisms differ. Many connectors have been developed by individual precast manufacturers over the years and, although some have been tested, there is generally limited experimental data on their behavior.

Bonded, cast-in-place concrete topping slabs are also used to join double tees in place of mechanical connectors. Some existing model building codes require topping slabs to form reliable diaphragm action in high seismic regions. Mechanical connectors generally require less labor and are faster to install; therefore, they would be preferable to topping slabs in seismic regions if they could provide similar or better strength and ductility.

The main objective of this project was to study the in-plane behavior of double-tee flange connectors and to supply experimental evidence for predicting the in-plane behavior of double-tee diaphragms using analytical models. The work was conducted as a part of the National Science Foundation sponsored Precast Seismic Structural Systems (PRESSS) research program intended to improve the seismic resistance of, and design methods for, precast concrete buildings.

Only a portion of the information accumulated during the study is presented here because of space limitations. In a preliminary study by Pincheira et al.,[1] a commonly used connector consisting of two reinforcing bars welded to a steel plate embedded in 2 in. (51 mm) thick flanges was tested under a variety of loading conditions. In this paper, the test results of seven additional types of mechanical connectors are presented.

The connectors were tested under loads including in-plane monotonic and reversed cyclic shear loading; vertical out-of-plane shear was applied in some tests; and the effects of lateral restraint against joint opening and an initial opening of the joint were also considered.

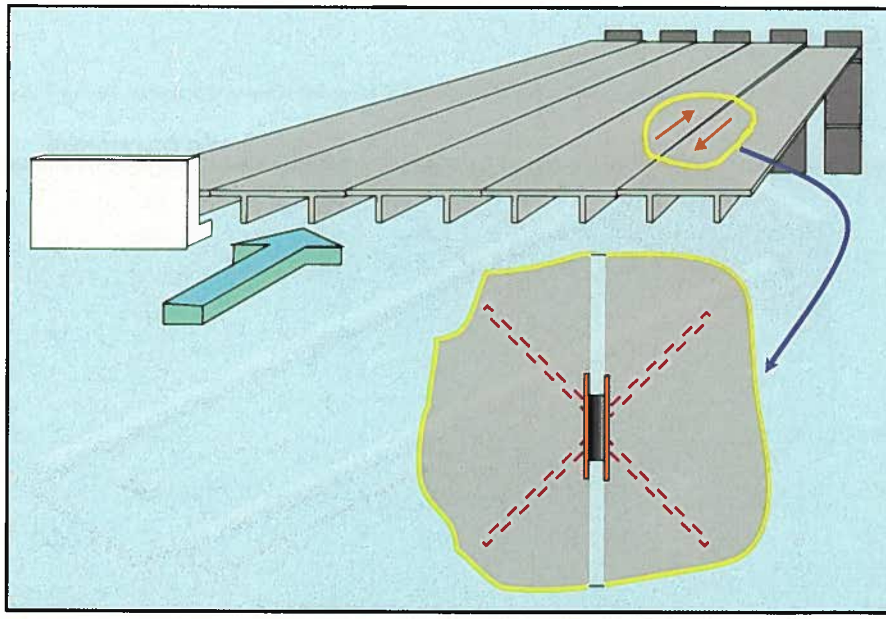

Fig. 1. Precast, prestressed double-tee floor system with mechanical connectors.

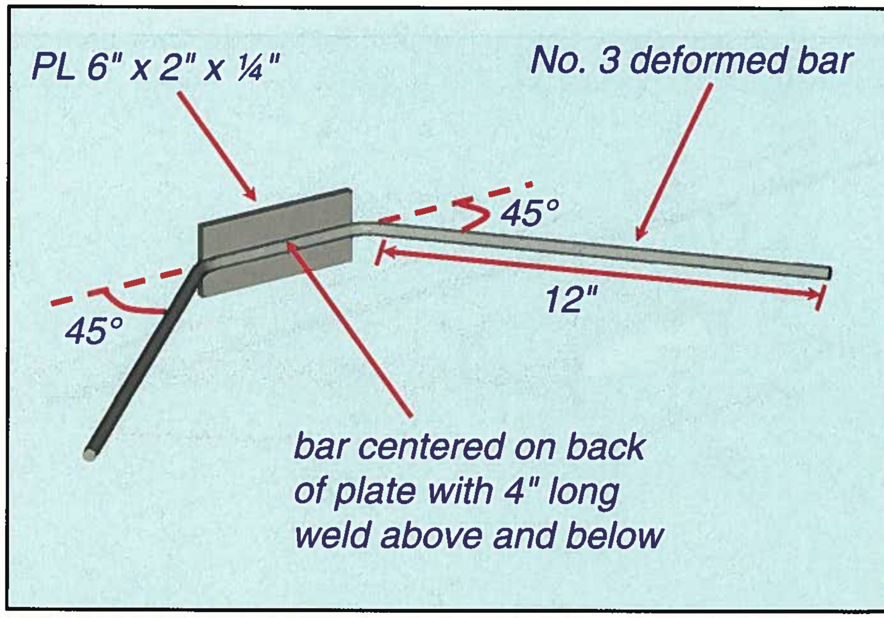

Fig. 2. Details and dimensions of the hairpin connector (2 in. flanges). Note 1 in. = 25.4 mm.

CONNECTORS STUDIED

Following advice from a project advisory panel of professional members of the Precast/Prestressed Concrete Institute (PCI), four types of flange-to-flange connectors were initially selected for testing as part of a National Science Foundation and PRESSS research program.

Two of the four types were commonly used in 2 in. (51 mm) double-tee flanges: the "hairpin" and the "plate with stud-welded deformed bar anchor" connectors. The third connector type was sometimes used in 4 in. (102 mm) flanges and consisted of a structural tee with two welded reinforcing bars. The fourth connector type consisted of a bent strap of steel known commercially as Vector® and could be used either in 2 or 4 in. (51 or 102 mm) thick flanges. Full details of the testing of those connectors are given in a separate report by Zheng.[2]

Three additional connectors were tested in an extension of the initial test program. The additional tests were accomplished through funding provided by a precast concrete manufacturer and two commercial material suppliers. Two of these connectors, a Vector and a bent plate, were intended for use in 4 in. (102 mm) thick pretopped flanges. The other connectors, an angle

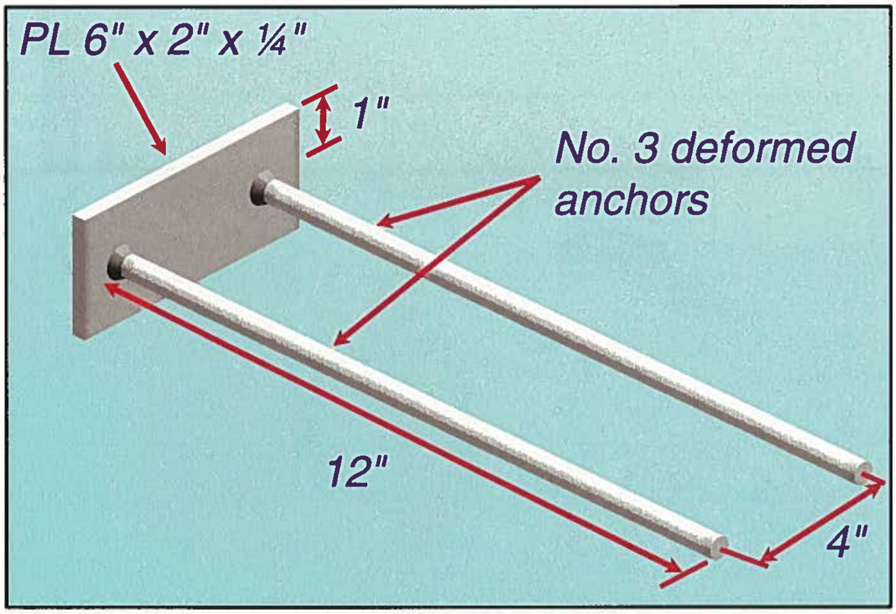

Fig. 3. Details and dimensions of the stud-welded anchor and plate connectors (2 in. flanges). Note: 1 in. = 25.4 mm.

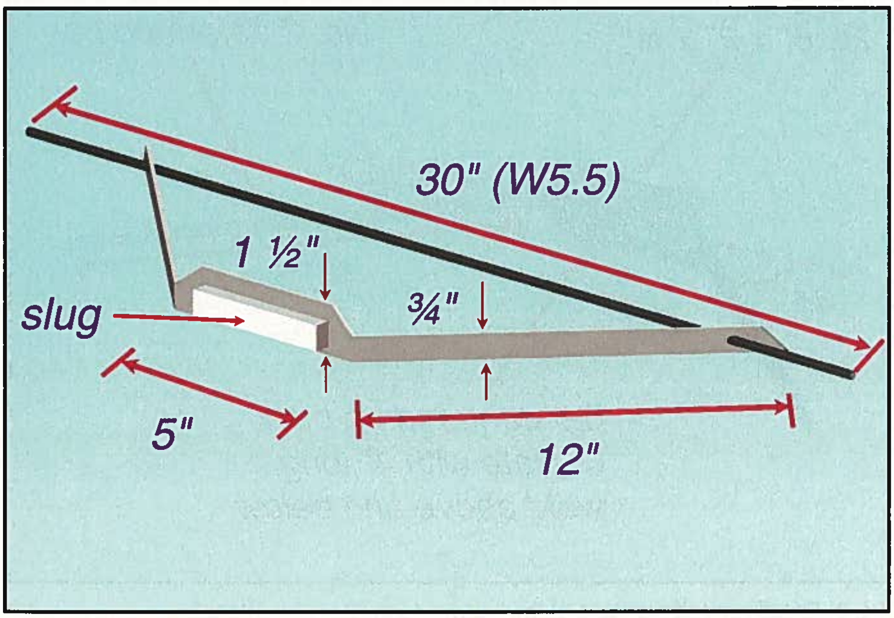

Fig. 4. Details and dimensions of the 3/16 in. thick bent wing connector (2 and 4 in. flanges). Note: 1 in. = 25.4 mm.

attached to the flange mesh reinforcement [mesh and angle (M&A)], had two separate configurations for use in 2 and 4 in. (51 and 102 mm) thick flanges. A more detailed description of the connectors follows.

In each connection, a separate steel bar, or "slug," is placed between the connectors embedded in the two concrete flanges. The slug is then welded to the connectors for adjoining double tees to complete the connection.

One characteristic of all the connectors is important to note. In the pilot test program,[1] round steel bars were used as the slugs that were dropped between the flange connectors and welded into place to create the connection. In some cases, failures were found to occur in the welds between the round bars and the connector plates.

Examination of the welds showed that a consistent thickness groove weld was not being achieved. This was attributed to the difficulty the welders had in accessing the weld location. All of the connectors in the tests described here were intentionally designed to have vertical face plates at the flange edges. Rectangular weld slugs were used to allow fillet welding rather than groove welding.

Two Inch (51 mm) Flange Connectors

Hairpin—Main dimensions and details of this connector are shown in Fig. 2. A 32 in. (813 mm) long ASTM A706 No. 3 reinforcing bar was bent in the form of a "hairpin" with two, 12 in. (305 mm) long legs. The center portion of the bar was then welded to an ASTM A36 steel plate with a 4 in. (102 mm) long, 1/4 in. (6.4 mm) thick flare-bevel weld above and below the bar.

The weld was terminated about 1 in. (25.4 mm) from the start of the bar bend, more than the two-bar diameter recommendation to avoid potential crystallization as suggested by PCI.[3] One connector was fabricated using AISI 304 stainless steel for the plate. All connectors were embedded in the concrete with the intention of having the anchor bar at mid-depth. A minimum cover of 3/4 in. (19 mm) from the top surface of the concrete to the bar was always maintained.

Stud-Welded DBA—Two, 12 in. (305 mm) long, No. 3 deformed bar anchors (DBA) were stud-welded perpendicular to a plate (see Fig. 3). The DBAs were made of ASTM A496 steel, while the plate was fabricated of ASTM A36 steel. The DBAs were also placed at mid-depth of the flange.

Bent Wing—Main dimensions and details of this connector are shown in Fig. 4. The bent wing connector was designed at the University of Wisconsin and made by cutting and bending a 3/16 in. (4.8 mm) thick steel strap. A 30 in. (762 mm) long, 0.264 in. (6.7 mm) diameter, W5.5 steel wire was fed through a slot (one in each wing) 1 in. (25.4 mm) from the end of the "wings."

This detail was needed to properly anchor the wings and was based on a previous study by Seo[4] that showed that the best anchorage could be obtained with a mechanical bar interlock. All connectors were placed in the forms with the top of the face plate at the top of the flange and 3/4 in. (19 mm) cover over the anchor leg.

M&A—This was an unusual connector in that it did not rely on separate anchor bars to transfer forces into the concrete of the flange. The connector was commercially produced as part of the reinforcing mesh normally placed in flanges. It consisted of a steel angle at the flange edge as shown in Fig. 5. The mesh was welded to the bottom of the angle during mesh production. The angle was $^3/_4 \times {}^3/_4 \times {}^1/_8$ in. thick ($19 \times 19 \times 3$ mm) made of ASTM A36 steel and extended the full length of the double-tee flange. The joint slug was welded directly to the angle.

Four Inch (102 mm) Flange Connectors

Structural Tee—This connector consisted of two, 16 in. (406 mm) long, No. 4 reinforcing bars welded at a 45-degree angle to an ASTM A36 structural steel tee section (see Fig. 6). The bars were made of ASTM A706 steel and were welded with 4 in. (102 mm) long, $^3/_{16}$ in. (4.8 mm) thick, flare-bevel welds on both sides of the bar to the bottom of the stem of the structural tee connector. A $1^5/_8$ in. (41 mm) cover was used between the face of the bar and the top concrete surface with the flange of the structural tee connector at the top surface.

Bent Wing—The dimensions and details of this connector are similar to those used in the 2 in. (51 mm) flanges described previously (see Fig. 4.) The plate material was also ASTM A36 steel, except that one specimen was made of AISI 304 stainless steel. The concrete cover was 1 in. (25.4 mm) from the top of the concrete surface to the wing anchor bar with the face plate $^1/_4$ in. (6.4 mm) below the flange surface.

Bent Plate—The bent plate was a simple connector fabricated by punching a rectangular hole into, and then bending, a $^1/_4$ in. (6.4 mm) thick steel plate (see Fig. 7). The hole provided concrete anchorage and the bend left a 5 in. (127 mm) long exposed steel face plate at the edge of the double tee for subsequent welding with a slug. The face plate was set $^1/_2$ in. (12.7 mm) below the flange surface.

Vector—A bent strap, shown in Fig. 8, is a commercially marketed connector referred to as the Vector connector and is similar in concept to the University-developed bent wing. While the Vector connector also had a provision for threading a rod through the anchor end, it was tested without the rod present to see if sufficient anchorage existed. Both carbon steel and stainless steel styles of the connector were tested. The face plate of the Vector connectors was set $^1/_4$ in. (6.4 mm) below the flange surface.

M&A—A second version of the M&A connector for 4 in. (102 mm) flanges, similar to that used in the 2 in. (51 mm) flange thickness (see Fig. 5), was commercially produced and tested.

Fig. 5. Overall view of mesh and angle (M&A) connector (2 and 4 in. flanges). Note: 1 in. = 25.4 mm.

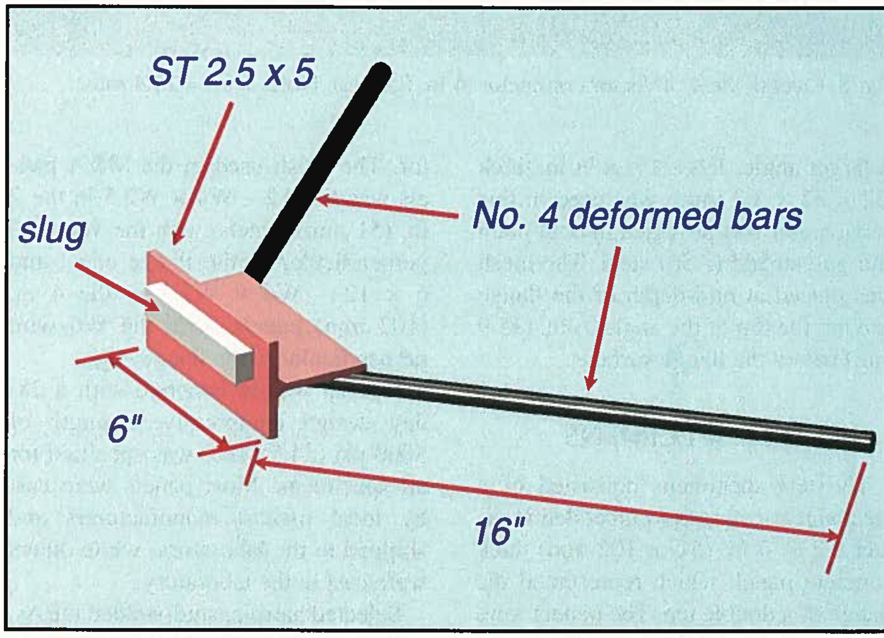

Fig. 6. Details and dimensions of the structural tee connector (4 in. flanges). Note: 1 in. = 25.4 mm.

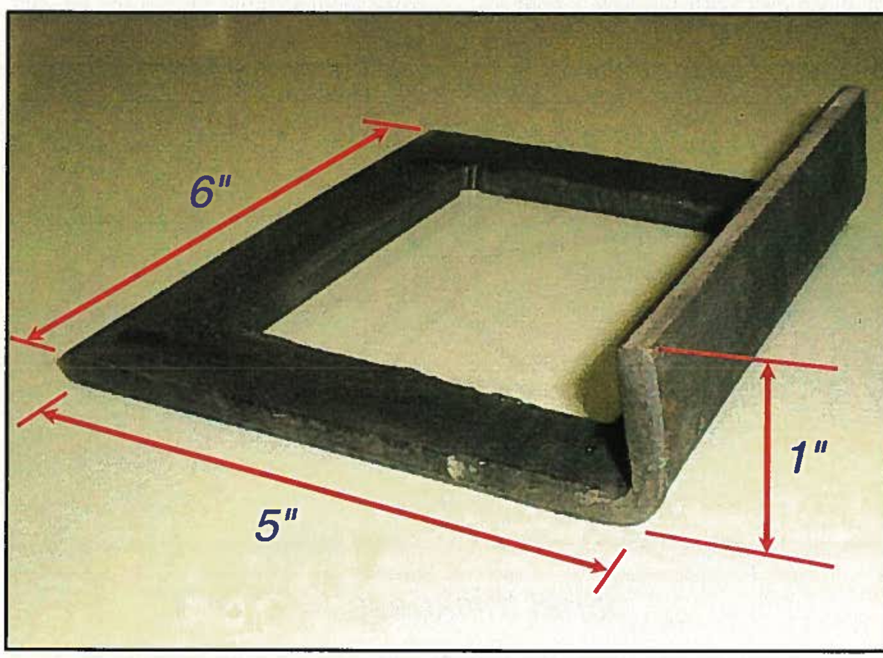

Fig. 7. Overall view of bent plate connector. Note: 1 in. = 25.4 mm.

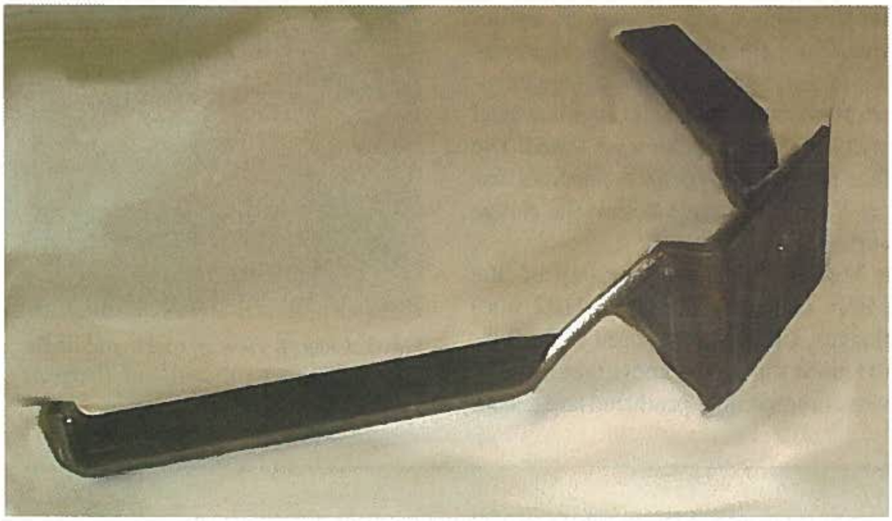

Fig. 8. Overall view of Vector connector (4 in. flanges). Note: 1 in. = 25.4 mm.

A larger angle, $1^1/_4 \times 1^1/_4 \times {}^1/_8$ in. thick (32 × 32 × 3.2 mm), was used in this version and was provided in both plain and galvanized (A36) steel. The mesh was placed at mid-depth of the flange leaving the top of the angle $^5/_8$ in. (15.9 mm) below the flange surface.

TEST SPECIMENS

The test specimens consisted of a mechanical connector embedded in either a 2 or 4 in. (51 or 102 mm) thick concrete panel, which represented the flange of a double tee. The panels were 4 × 4 ft (1.22 × 1.22 m) and were reinforced with 6 × 6 – W2.1 × W2.1 welded wire reinforcment, except for those cast with the M&A connector. The mesh used in the M&A panels was 6 × 12 – W4 × W2.5 in the 2 in. (51 mm) panels, with the W4 wire perpendicular to the flange edge, and 6 × 12 – W6 × W2.5 in the 4 in. (102 mm) panels, with the W6 wire perpendicular to the flange edge.

Normal weight concrete with a 28-day design compressive strength of 5000 psi (34.5 MPa) was specified for all specimens. Most panels were cast by local precast manufacturers and shipped to the laboratory, while others were cast in the laboratory.

Selected hairpin, stud-welded DBAs, and structural tee connectors were provided with foam inserts at the ends of the connector plates, or over the end cross section in the case of the tee, before they were embedded in the panels. The inserts were $^1/_4$ in. (6.4 mm) thick and served to prevent direct bearing of the plates (or the structural tee) on the surrounding concrete during the initial stages of loading.

Based on previous experience,[1] a single panel with one connector was used in this test program (see Fig. 9) instead of a two-panel test configuration. In practice, the connection between double-tee flanges is often accomplished by welding a bar or slug of steel to two adjacent connectors.

To form the test connection, a rectangular slug was welded to the connector on one side and to a loading beam on the other side (details of the test setup are provided in other sections of this paper). This configuration allowed an easier, more rapid installation and testing of the specimens, and savings in materials and fabrication costs with respect to a two-panel test configuration.

The slug weld was sized to be stronger than the estimated strength of the connector. Accordingly, the slug was welded with a $^1/_4$ in. (6.4 mm) fillet weld. A weld length of 4 in. (102 mm) was used for all connectors. The slug was a rectangular steel bar and varied in thickness from $^3/_8$ to $^3/_4$ in. (9.5 to 19.0 mm), as needed to bridge the joint gap width. Since the face plates were vertical, the slug could not be simply dropped into the joint as is often done when sloped face plates are used. The slugs were held at a position $^5/_{16}$ in. (7.9 mm) below the top of the steel connector face plate and welded.

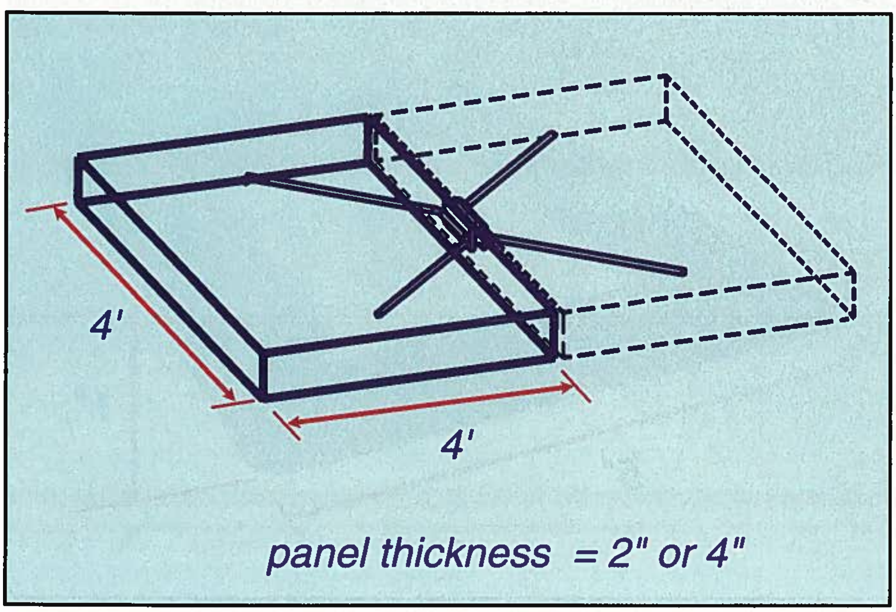

Fig. 9. Test specimen dimensions. Note: 1 in. = 25.4 mm.

LOADING AND SUPPORT CONDITIONS

The connectors were subjected to several loading conditions to emulate some of the actions expected in a double-tee floor system. These actions consisted of separate or combined in-plane (horizontal) shear and out-of-plane (vertical) shear. In-plane shear was applied to simulate the forces in the connectors caused by lateral loads (wind or earthquake).

Out-of-plane shear was intended to simulate the forces caused by removal of differential camber in the double tees or by the wheel loads traversing a joint in the floors of parking structures.

A vertical load of 1.5 kip (6.7 kN) was selected to represent one-quarter of a sport-utility vehicle's (SUV) weight and one connector was conservatively assumed to transfer this force.

In practice, volumetric changes associated with temperature and shrinkage may cause contraction of the diaphragm and opening of the joints between double tees. This contraction will induce an in-plane tension in the connectors, which may affect their behavior when subjected to shear. To study the effects of this loading condition, some connectors were "preloaded" in tension by opening the joint a prescribed amount before applying the in-plane shear load.

It was also recognized that the joints in a double-tee diaphragm are not always free to open when in-plane shear loads are applied because they are restrained by the adjacent double tees. For this reason, the opening of the joint was restrained during the tests of some specimens.

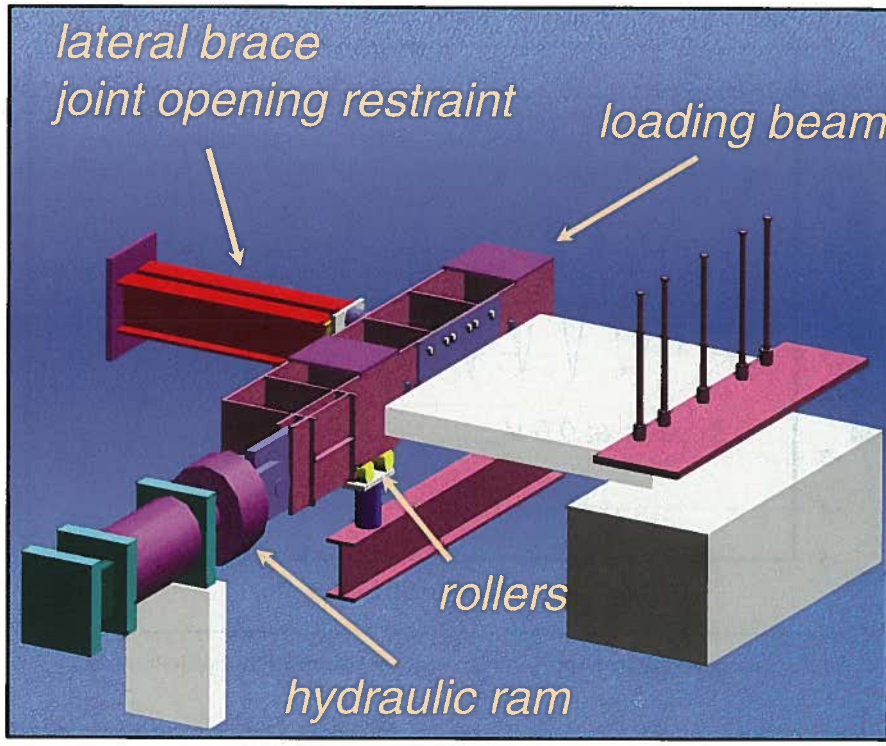

Fig. 10. Test setup for specimens subjected to horizontal shear.

TEST SETUP

An overall view of the test setup for specimens subjected to in-plane shear is shown in Fig. 10. The load was applied using a large, rigid steel beam welded to the connector with a rectangular slug of steel. At the opposite end, the panel was tied down to the laboratory floor with the aid of a concrete block. In each test, special care was taken to ensure that the concrete specimen was not restrained in the vicinity [within 16 in. (406 mm)] of the connector to avoid spurious effects on the response of the connectors.

To simulate horizontal restraint against joint opening, the loading beam was laterally braced (see Fig. 10). A small pre-compressive force of approximately 400 lb (1.8 kN) was applied in the direction perpendicular to the panel edge to ensure an initial tight contact that would later develop higher passive lateral restraint force and prevent joint opening during selected tests.

The effects of joint opening were simulated by creating an initial gap between the loading beam and the panel edge after welding of the steel slug. This opening was held constant during the tests by two round bars (rollers) placed between the loading beam and the panel. Joint openings of $^1/_{16}$, $^1/_8$, or $^1/_4$ in. (1.6, 3.2, or 6.4 mm) were used.

The larger values of opening were chosen as upper bounds for the joint opening that may accumulate from temperature contraction and volume changes over a group of double tees. (Note that because only one side of a real joint was tested, the joint opening of a real diaphragm would be twice the above values.)

Table 1. Number of tests conducted according to connector type and load condition.

Connector type	Load condition					Total
	In-plane shear		In-plane and out-of-plane shear		Tension	
	Monotonic	Cyclic	Monotonic	Cyclic	Monotonic	
Hairpin	5	4	—	—	1	10
Stud-welded anchor	4	4	—	—	1	9
Structural tee	4	4	1	3	1	13
Bent wing	7	10	1	2	1	21
Mesh and angle (2 in. flanges)	5	4	1	—	1	11
Mesh and angle (4 in. flanges)	3	4	—	1	—	8
Vector	10	9	—	—	4	23
Total	38	39	3	6	9	95

Note: 1 in. = 25.4 mm.

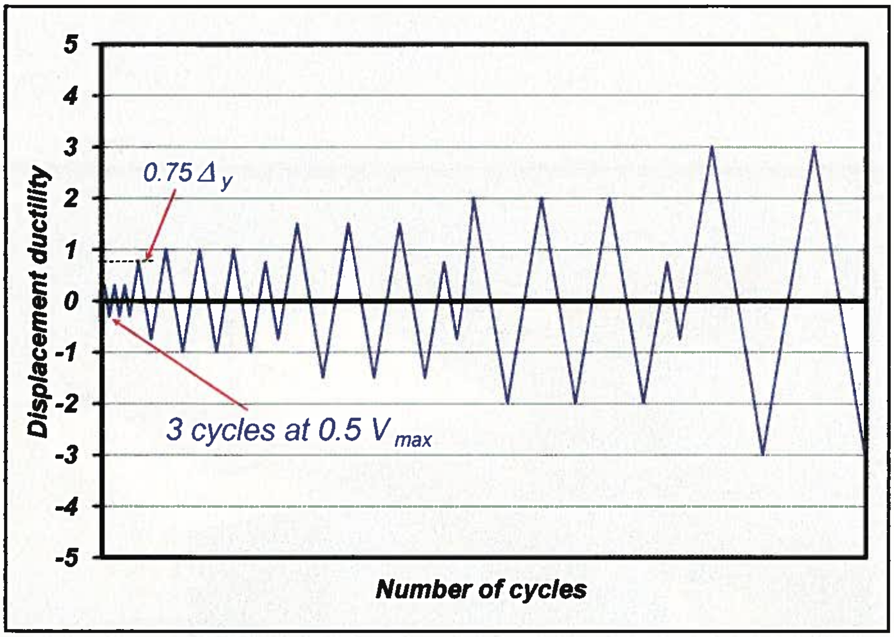

Fig. 11. Deformation pattern applied to specimens subjected to cyclic loading.

To apply the vertical (out-of-plane) load, the roller supports under the steel loading beam were removed and hydraulic jacks applied a downward load of 1.5 kip (6.7 kN). This load was estimated to be typical of the wheel load induced in a connector of a parking structure. This load was applied in selected 4 in. (102 mm) thick specimens. Similarly, the effects of vertical upward shear were simulated by applying a downward load to the panel (on the side of the connector) and by supporting the loading beam.

To test a connector under pure axial tension, the specimen was held in a different position. A 1½ in. (38 mm) thick steel loading plate was welded to the slug and attached to a hydraulic ram to apply an axial tension force to the connector. Both the specimen and the steel

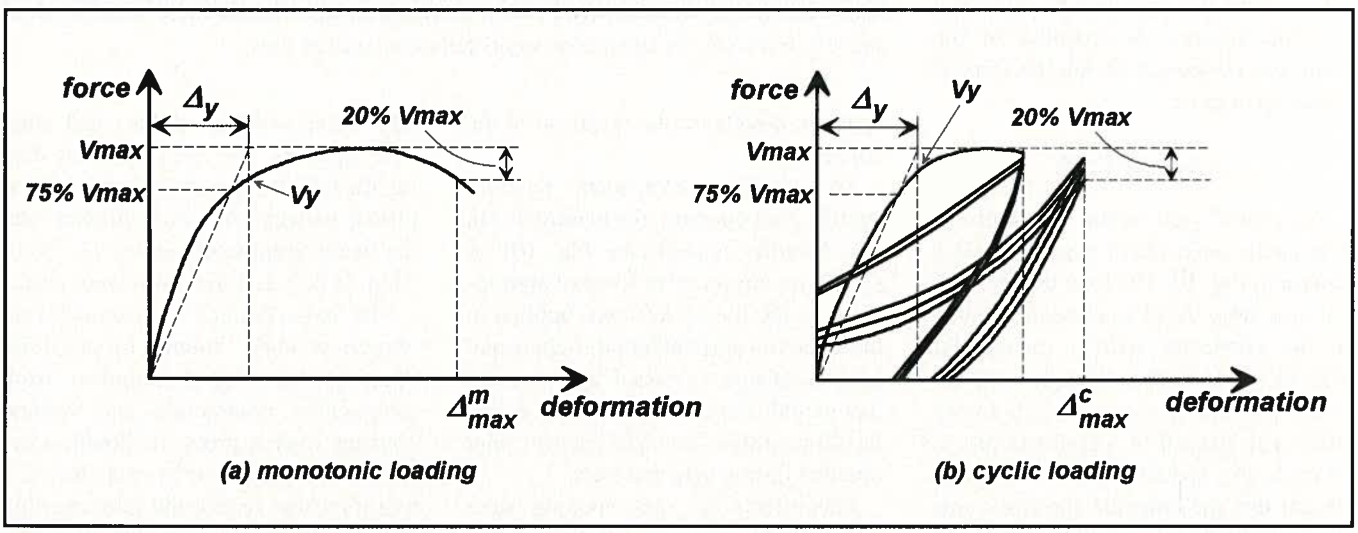

Fig. 12. Definition of stiffness, yield and maximum strength, and yield and maximum deformation.

Table 2. Summary of test results – Hairpin connectors.

Load condition		Specimen	Joint restraint	In-plane tension opening (in.)	In-plane shear strength (kip): Bond-splitting crack load	Yield strength	Peak strength	Initial stiffness (kip/in.)	Δ_{max} (in.)	μ	Failure mode
Shear	Monotonic	SV1HP1	No	None	17.0	15.7	19.3	948	0.08	3.9	Anchor rupture
		SV2HP2R	Yes	None	10.0	19.9	21.5	1011	0.29	13.6	Anchor rupture
		SV1HPSL^{SS}	No	None	No cracks	15.1	15.1	724	0.02	1.1	Anchor rupture
		SV3HP5RI^{F}	Yes	None	13.0	13.5	22.1	896	0.24	9.6	Concrete spalling
		SV3HP3RT	Yes	1/16	0.0^{w}	14.1	16.2	408	0.27	6.7	Bar pullout
	Cyclic	CV1HP3R	Yes	None	15.7	16.5	21.0	696	0.06	1.8	Anchor rupture
		CV2HP4	No	None	15.2	14.4	16.6	1029	0.05	3.2	Anchor rupture
		CV2HP5T	Yes	1/16	12.0	13.9	16.8	590	0.09	3.1	Anchor rupture
		CV3HP6T	Yes	1/16	14.6	14.1	17.2	583	0.08	2.8	Anchor rupture
Tension	Monotonic	ST1HP7	N/A	None	5.0	7.8	7.8	145	0.10	1.8	Anchor rupture

Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN; w denotes racks developed shortly after welding of the slug; SS denotes stainless steel plate; F denotes foam inserts at plate edges; and N/A denotes not applicable.

loading plate were vertically supported on Teflon pads, so that they could move freely in the axial loading direction.

Table 1 shows a summary of the test protocol used in this study with the type and number of specimens tested under each loading and support condition described previously.

LOADING PROCEDURE

Under monotonic loading, the specimens were tested using force-controlled increments until 75 percent of the estimated strength of the connector was reached. Subsequently, the specimens were subjected to prescribed increments of displacement until failure.

For reversed cyclic loading, a prescribed sequence of displacement ductility increments was used based on the test procedure recommended by the technical committee of the PRESSS program.[5] The displacement pattern intended for the tests is shown in Fig. 11. In some cases, however, it was not possible to follow the exact displacement pattern desired because of premature softening or partial failure of the connector.

INSTRUMENTATION

The panels were instrumented with two linear variable differential transducers (LVDTs) to measure the shear deformation. Two additional devices were used to measure the opening of the joint gap. This setup allowed measurement of not only deformation of the connector itself, but also deformations in the anchors and the concrete surrounding the connector. A similar instrumentation setup was used in the out-of-plane shear and pure in-plane tension tests but the deformations in different directions were measured, as appropriate.

The applied load was recorded by a 200 kip (890 kN) load cell attached to the hydraulic ram. Test data were continually recorded by a data acqui-

Table 3. Summary of test results – Stud-welded DBA connectors.

Load condition		Specimen	Joint restraint	In-plane tension opening (in.)	In-plane shear strength (kip)			Initial stiffness (kip/in.)	Δ_{max} (in.)	μ	Failure mode
					Bond-splitting crack load	Yield strength	Peak strength				
Shear	Monotonic	SV1SP1	No	None	6.1	10.5	13.1	660	0.06	2.9	Weld fracture
		SV2SP2R	Yes	None	10.0	10.3	12.2	647	0.09	4.9	Weld fracture
		SV3SP4RI[F]	Yes	None	9.0	9.6	10.8	550	0.08	4.1	Concrete spalling
		SV4SP6RT	Yes	1/8	0.0[w]	4.0	4.7	74	0.18	2.8	Concrete spalling
	Cyclic	CV1SP3R	Yes	None	11.0	9.6	16.5	634	0.05	2.1	Weld fracture
		CV2SP5RI[F]	Yes	None	8.1	9.0	10.9	482	0.04	1.5	Weld fracture
		CV5SP7T	Yes	1/16	0.0	9.9	11.3	445	0.04	1.5	Weld fracture
		CV6SP8T	Yes	1/16	5.0	NR	NR	NR	NR	NR	Weld fracture
Tension	Monotonic	ST1SP8	N/A	None	7.0	7.4	8.6	263	0.20	6.0	Weld fracture

Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN; w denotes racks developed shortly after welding of the slug; F denotes foam inserts at plate edges; NR denotes not recorded due to equipment malfunction; and N/A denotes not applicable.

Table 4. Summary of test results – Structural tee connectors.

Load condition		Specimen	Joint restraint	Out-of-plane shear (upward or downward) (kip)	In-plane tension opening (in.)	In-plane shear strength (kip)			Initial stiffness (kip/in.)	Δ_{max} (in.)	μ	Failure mode
						Bond-splitting crack load	Yield strength	Peak strength				
Shear	Monotonic	SV1ST1	No	None	None	20.0	30.3	30.5	1201	0.03	1.0	Panel shear crack
		SV2ST2R	Yes	None	None	25.0	25.3	28.7	1618	0.07	3.8	Panel shear crack
		SV1ST1R[F]	Yes	None	None	17.0	22.4	30.0	1005	0.09	2.9	Panel shear crack
		SV2ST4T[F]	Yes	None	1/8	20.0	26.8	31.2	425	0.43	5.9	Anchor rupture
		SV3ST8RU[F]	Yes	2.1 up	None	25.0	24.8	30.7	1061	0.22	7.6	Panel shear crack
	Cyclic	CV1ST2R[F]	Yes	None	None	14.1	24.6	26.8	1164	0.03	1.4	Panel shear crack
		CV2ST3R[F]	Yes	None	None	15.0	23.7	27.4	1454	0.04	1.8	Concrete spalling
		CV3ST5T[F]	Yes	None	1/8	22.0	22.5	26.6	598	0.16	3.6	Anchor rupture
		CV4ST7T[F]	Yes	None	1/8	23.0	23.1	26.7	494	0.16	2.9	Anchor rupture
		CV5ST9RU[F]	Yes	1.7 up	None	22.5	21.2	23.7	1464	0.02	1.2	Anchor debond
		CV6ST10RD[F]	Yes	1.7down	None	15.0	14.1	16.9	639	HH	HH	Spalling of bottom cover
		CV7ST11RD[F]	Yes	1.9 down	None	19.0	20.6	23.1	772	HH	HH	Spalling of bottom cover
Tension	Monotonic	ST1ST10	N/A	None	None	12.0	11.9	13.6	322	0.22	5.2	Anchor rupture

Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN; F denotes foam inserts at edges of flange and stem of structural tee; HH denotes specimen failed abruptly at first cycle to 1.5 Δ_y; and N/A denotes not applicable.

Table 5. Summary of test results - Bent wing connectors.

Load condition		Specimen	Joint restraint	Out-of-plane shear (upward or downward) (kip)	In-plane tension opening (in.)	Panel thickness (in.)	In-plane shear strength (kip)			Initial stiffness (kip/in.)	Δ_{max} (in.)	μ	Failure mode
							Bond-splitting crack load	Yield strength	Peak strength				
Shear	Monotonic	SV1BW1	No	None	None	4	4.0	9.4	11.3	621	0.28	15.3	Anchor rupture
		SV1BW2R	Yes	None	None	4	5.0	11.3	13.2	590	0.77	34.2	Anchor rupture
		SV1BW7T	Yes	None	1/4	4	11.5	10.2	11.7	395	0.33	11.3	Anchor rupture
		SV2BW9T	Yes	None	1/8	4	10.4	8.9	10.4	525	0.66	33.1	Anchor rupture
		SV5BW13R	Yes	2.5 up	None	4	No cracks	10.2	11.2	621	0.38	21.2	Anchor rupture
		SV1BWSLR[SS]	Yes	None	None	4	No cracks	16.2	20.5	561	1.04	28.5	Anchor rupture
		SV2BW2R	Yes	None	None	2	No cracks	10.4	12.4	337	0.28	7.7	Anchor rupture
		SV2BW3RT	Yes	None	1/8	2	No cracks	10.9	12.0	297	0.30	7.5	Anchor rupture
	Cyclic	CV1BW3R	Yes	None	None	4	12.2	10.7	12.3	786	0.08	4.9	Anchor rupture
		CV2BW4R	Yes	None	None	4	No cracks	9.4	11.1	417	0.10	3.9	Anchor rupture
		CV1BW8T	Yes	None	1/4	4	No cracks	8.6	9.5	396	0.10	3.6	Anchor rupture
		CV2BW10T	Yes	None	1/4	4	No cracks	9.1	9.9	179	0.07	1.3	Anchor rupture
		CV3BW11T	Yes	None	1/8	4	9.6	9.1	9.9	735	0.03	2.5	Anchor rupture
		CV4BW12T	Yes	None	1/8	4	No cracks	8.4	10.0	572	0.06	3.3	Anchor rupture
		CV5BW14R	Yes	1.7 up	None	4	No cracks	8.4	10.4	496	0.06	2.9	Anchor rupture
		CV6BW15R	Yes	1.8 up	None	4	No cracks	9.5	11.3	740	0.03	1.9	Anchor rupture
		CV1BW2R	Yes	None	None	2	No cracks	9.2	11.3	339	0.11	3.4	Anchor rupture
		CV2BW3R	Yes	None	None	2	No cracks	8.5	10.5	373	0.09	3.5	Anchor rupture
		CV3BW4T	Yes	None	1/8	2	8.6	8.3	10.0	316	0.16	5.4	Anchor rupture
		CV4BW5T	Yes	None	1/8	2	No cracks	7.7	10.1	243	0.18	4.2	Anchor rupture
Tension	Monotonic	ST1BW6	N/A	None	None	2	No cracks	3.5	4.4	32	1.14	8.3	Anchor rupture

Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN; SS denotes stainless steel plate; and N/A denotes not applicable.

sition system until failure of a specimen. During the tests, special attention was paid to recording visual evidence of cracking as a means of establishing maximum service load capacities for use in design.

OBSERVED BEHAVIOR AND TEST RESULTS

The main test results are summarized in Tables 2 through 7. These results are presented in terms of the initial stiffness, the yield and peak strength, the deformability, and the displacement ductility of the connectors. The definition of most of these parameters is illustrated in Fig. 12. The initial stiffness was computed from the recorded load and displacement response as the secant through the origin to a point on the load and displacement plot at 75 percent of the maximum resistance (see Fig. 12a).

The yield strength was calculated from the measured load and displacement response as the load corresponding to the yield displacement, Δ_y. The latter was defined as the intersection between the horizontal line that passes through the point of maximum resistance and the secant defined previously to compute the initial stiffness (see Fig. 12).

The deformability of the connectors, Δ_{max}, was defined as the displacement corresponding to strength loss of 20 percent of the maximum applied load (see Fig. 12a). Under cyclic loading, the deformability of the specimens was defined in the same way except using the measured response after the third cycle applied at a given displacement

Table 6. Summary of results: average values for mesh and angle connectors.

Load condition	Joint restraint	Out-of-plane shear (upward or downward) (kip)	Panel thickness (in.)	In-plane shear strength (kip)			Initial stiffness (kip/in.)	Δ_{max} (in.)	μ	Failure mode
				Bond-splitting crack load	Yield strength	Peak strength				
Monotonic Shear	No	None	2	N/A	14.8	19.5	640	0.31	14.4	Angle rupture
Cyclic Shear	No	None	2	N/A	15.2	19.8	744	0.08	3.0	Mesh weld
Monotonic Shear	No	None	4	N/A	23.6	28.4	666	0.17	4.1	Mesh weld
Cyclic Shear	No	None	4	N/A	22.6	29.2	1062	0.08	3.0	Mesh weld

Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN; and N/A denotes not applicable.

amplitude (see Fig. 12b). The displacement ductility, μ, was defined as the ratio between the deformability, Δ_{max}, and the yield displacement, Δ_y, defined previously.

Cracks frequently appeared on the top (or bottom) surface of the flanges above or below the embedded anchor legs of connectors. These cracks are referred to here as splitting cracks or anchorage splitting cracks. The load corresponding to the onset of splitting cracks and the observed failure mode are also reported in each table.

Under cyclic loading, the yield and peak resistance, stiffness, deformability, and ductility reported in the tables correspond to the average of the measured values in both directions of loading. Since only one side of a real joint was tested, the connectors' stiffness reported in Tables 2 through 7 is twice that of a real floor diaphragm. Similarly, the actual deformation will be twice that reported in the tables.

Because of the large number of parameters investigated for all seven connectors, only the main test results and principal observations are presented in the following section. The bent plate connector showed very brittle behavior and poor overall performance, and thus its behavior will not be discussed further here. Full descriptions of the tests on the bent plate and the rest of the connectors are reported elsewhere.[2, 6-9]

Load-Deformation Response

As might be expected, the response of the connectors varied depending on the connector type and loading condition. Some general trends, however, were observed and are described in the following. The response obtained under monotonic in-plane shear alone is used as a benchmark for evaluating the influence of other loads, joint openings, and lateral restraint conditions on the connectors' performance.

Monotonic Loading—In Fig. 13, the in-plane shear load and displacement response of most connectors embedded in 2 in. (51 mm) thick panels are shown. Fig. 14 shows similar plots for the connectors embedded in 4 in. (102 mm) thick panels.

In general, restraint of joint opening tended to increase the deformability of the connectors under in-plane shear, but

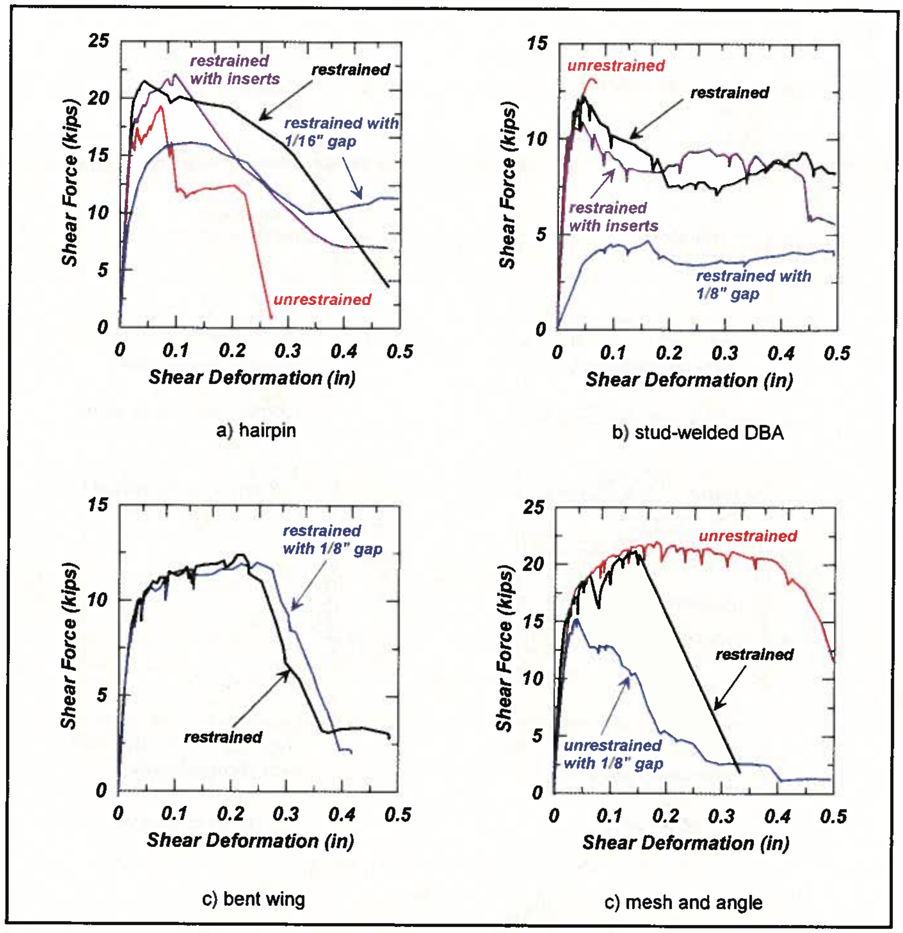

Fig. 13. Load and deformation response of connectors in 2 in. thick flanges: (a) hairpin; (b) stud-welded DBAs; (c) bent wing; and (d) mesh and angle. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

it had no appreciable effect on the shear strength or on the stiffness of the connectors. An exception to this result was the response of the hairpin connector in the 2 in. (51 mm) thick panels and the bent wing connector embedded in the 4 in. (102 mm) thick panels, where an increase in strength of approximately 20 percent was observed.

With a pre-applied joint opening or in-plane gap, the in-plane shear yield and peak shear strengths were generally reduced. Connector deformability, however, either increased or remained about the same. An initial joint opening was especially detrimental to the strength of the stud-welded DBAs (see Fig. 13b) and the M&A connector (see Fig. 13d).

A concurrent upward out-of-plane shear load had virtually no effect on the horizontal shear yield or peak strength of the structural tee, bent wing, and M&A connectors (see Tables 4 and 5 and Fig. 14). The in-plane shear deformability of the connectors was reduced, however, when compared to that of companion units with similar lateral restraint but without out-of-plane shear.

The connectors fabricated with a steel plate (i.e., hairpin, stud-welded DBAs, and bent plate) or with a structural tee showed severe cracking and spalling of the concrete in the region surrounding the connector after yielding—as the peak strength was approached. This behavior had also been observed in an earlier study of similar connectors.[1]

In an effort to reduce the damage and improve the behavior of those connectors, foam inserts were provided along the ends of the connector's plate and the structural tee to prevent direct bearing against the concrete. These inserts reduced the load corresponding to the onset of nonlinear response, but had no significant effect on the peak strength and deformability of the hairpin and the stud-welded DBA connectors (see Fig. 13).

The deformability of the structural

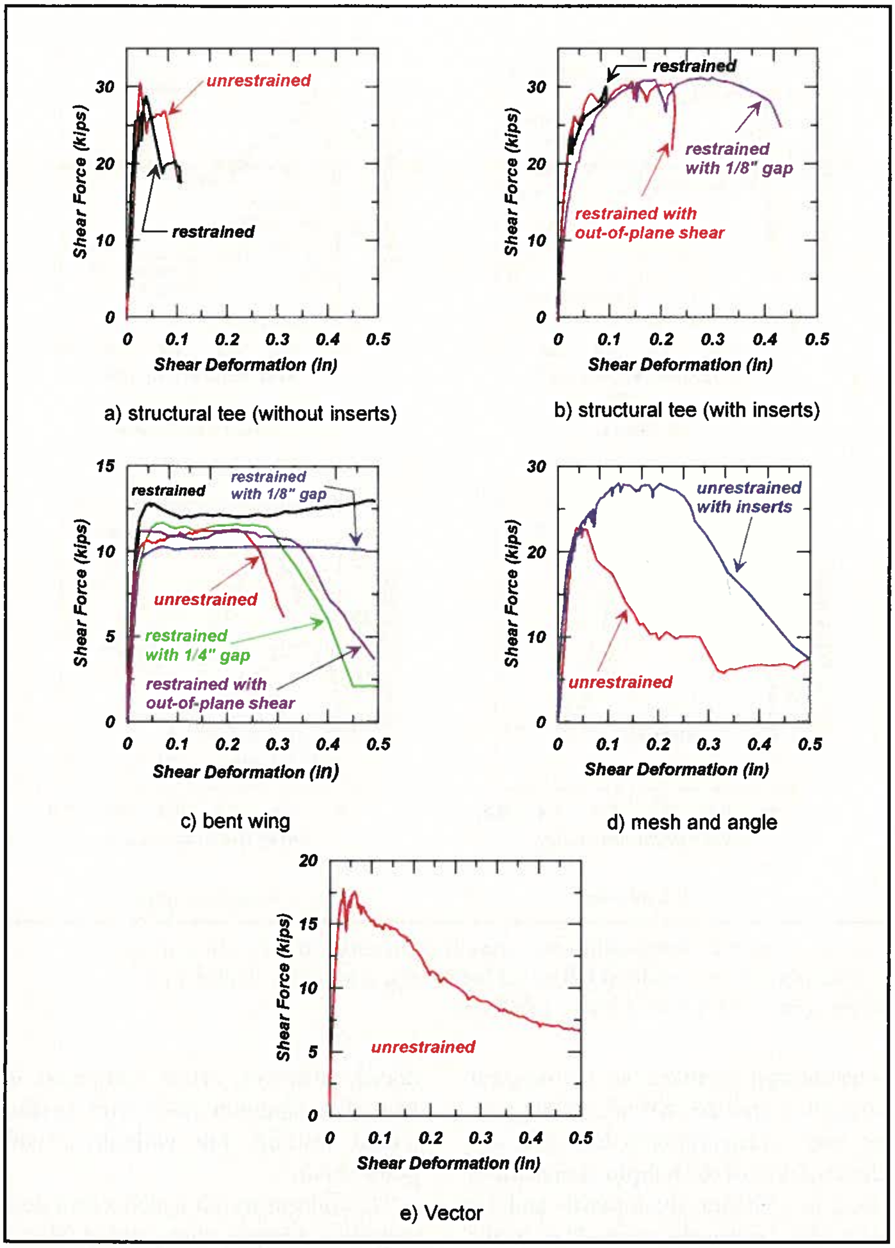

Fig. 14. Load and deformation response of connectors in 4 in. thick flanges: (a) structural tee (without inserts); (b) structural tee (with inserts); (c) bent wing; and (d) mesh and angle; and (e) Vector. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

tee connectors, however, increased significantly when foam inserts were provided (see Fig. 14). Furthermore, the foam inserts changed the crack pattern and failure mode of the connectors, as discussed in a following section.

Foam inserts were also placed adjacent to each of the mesh bars of the M&A connector, in the 4 in. (102 mm) panels, where they were welded to the angle. The aim of the insert was to increase the flexibility of the connector when subjected to in-plane shear loading. The results, plotted in Fig. 14d, show that the use of inserts increased the connector's strength and deformability.

Cyclic Loading—Figs. 15 through 21 show representative plots of the response of the connectors subjected to reversed cyclic in-plane shear. Also shown in the plots is the response of the companion specimen tested under monotonic loading for comparison.

- **Hairpin**—The response of the hairpin connectors (see Fig. 15) showed nearly full hysteresis loops at low levels of in-plane shear deformation. This behavior may be attributed to yielding of the anchor bars with only minor cracking of the concrete. The peak load was reached when the concrete adjacent to the connector plate reached its bearing capacity. Upon increasing the in-plane shear deformation, the connectors typically showed a decrease in strength as the concrete began to spall off with each loading cycle. The post-peak resistance of the connector was reduced with re-

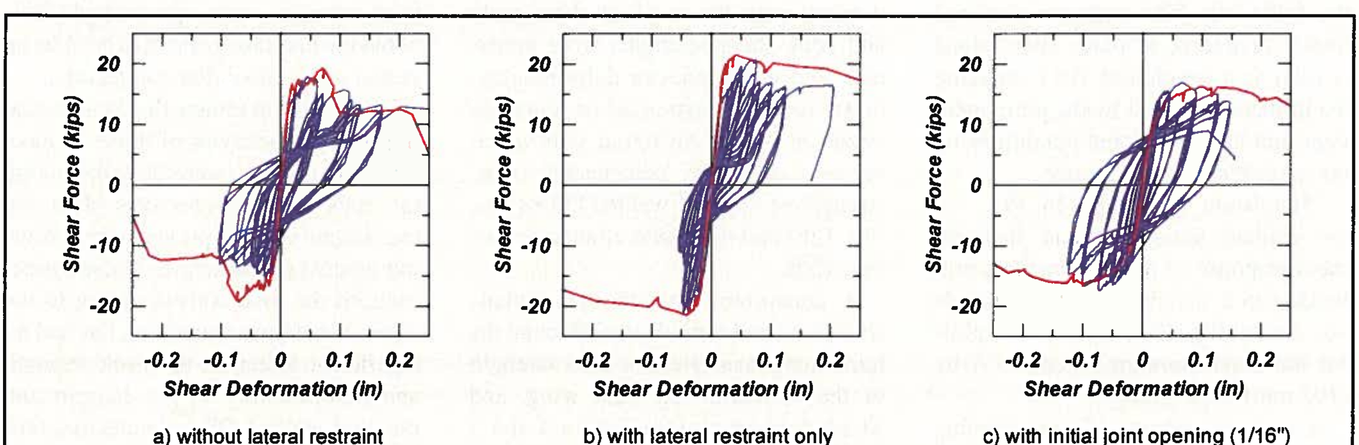

Fig. 15. Load and deformation response of the hairpin connectors under cyclic loading. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

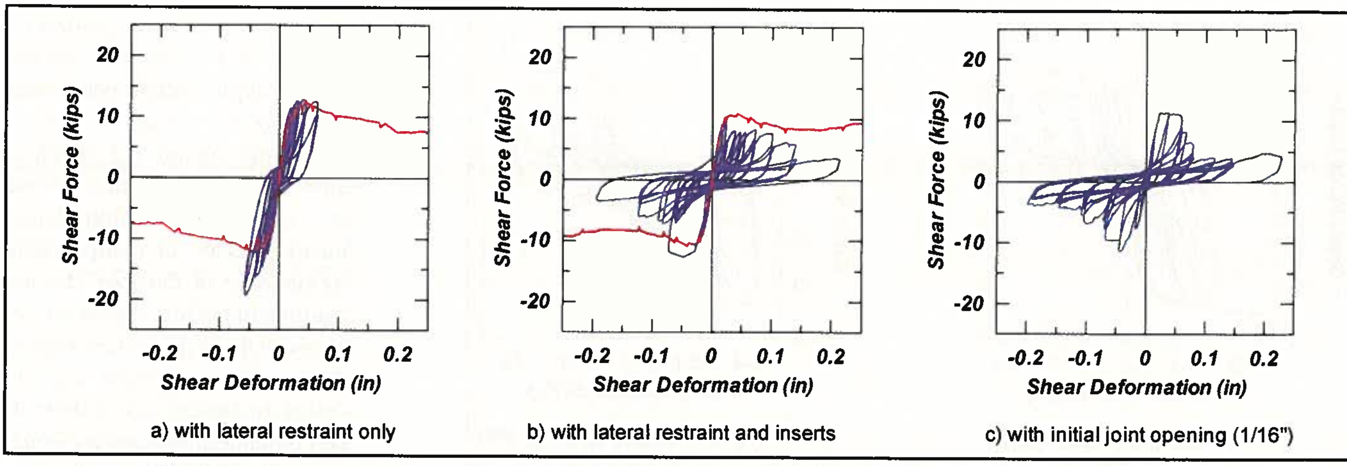

Fig. 16. Load and deformation response of the stud-welded DBAs under cyclic loading. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

spect to that measured under monotonic loading and the hysteretic loops became pinched with reduced energy dissipation. Lateral restraint increased the peak resistance (see Fig. 15b), but reduced somewhat the ductility of the connector with respect to that observed for the unrestrained unit (see Fig. 15a). Joint opening (see Fig. 15c) did not result in an appreciable difference in the strength or the deformation capacity of the connector, but it did result in fuller hysteretic cycles.

- **Stud-Welded DBA**—The response of the stud-welded DBA connectors was characterized with highly pinched hysteresis loops (see Fig. 16) regardless of the lateral restraint condition or the amount of initial joint opening. Similar to the behavior observed for the hairpin, the stud-welded DBA connectors showed rapid strength decay with loading cycles after reaching the peak load. Joint opening did not have a significant influence on the cyclic response of the stud-welded DBA connectors, but it changed their failure mode. Both units with an initial joint opening (Specimens CV5SP7T and CV6SP8T) failed by spalling of the concrete over the anchor bars with loss of anchorage, while those without joint opening failed by premature fracture of the stud-to-plate weld.
- **Structural Tee**—The response of the units with a structural tee connector varied drastically depending on the amount of initial joint opening and vertical load condition. The structural tee connectors tested under in-plane cyclic shear loading were all provided with foam inserts along the edges of the flange and the stem of the tee section. Without an initial joint opening (Specimens CV1ST2RI and CV2ST3RI), the response of the connector was characterized by a high stiffness and strength (see Fig. 17a). The connector transferred sufficient force to develop failure of the flange with large panel shear cracks, which limited the measured strength and, as a result, the connector's strength could not be fully mobilized. In structural tee connectors with an initial joint opening (Specimens CV3ST5TI and CV4ST7TI), the stiffness of the connector was reduced to one-half or less (see Table 4). The connector's response was characterized by full hysteresis loops with large energy dissipation. The loops exhibit a shape similar to the Bauschinger effect (a gradual softening of stiffness near yield) observed for reinforcing

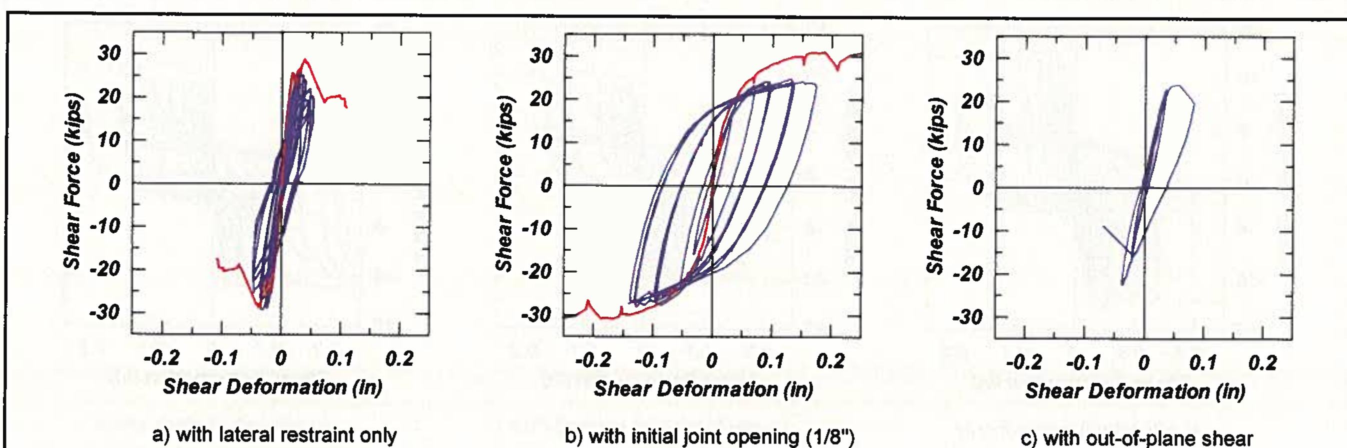

Fig. 17. Load and deformation response of the structural tee connectors under cyclic loading.
Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

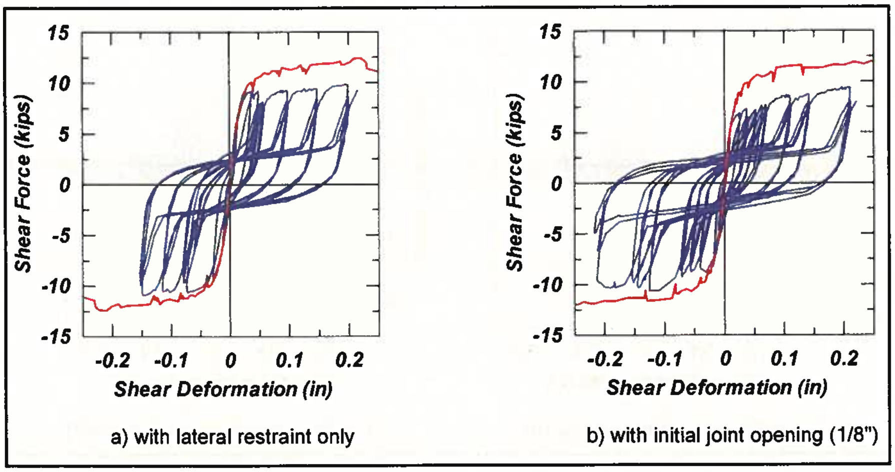

Fig. 18. Load and deformation response of the bent wing connectors in 2 in. flanges under cyclic loading. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

steel under cyclic loading (see Fig. 17b). The strength of these specimens, though similar to that of the specimens that failed in flange shear without joint opening, was reached when one of the anchors fractured in tension. The presence of out-of-plane shear on the structural tee connectors (either upward—Specimen CV5ST9RUI, or downward—Specimens CV6ST10RDI and CV7ST11RDI) was much more detrimental for cyclic in-plane shear response than under monotonic loading. Out-of-plane shear caused the concrete cover to break out and spall off above (or below) the tee section. As a result, the specimen showed severe strength and stiffness degradation shortly after reaching the peak strength (see Fig. 17c). While the strength of the connector was comparable to that of the test specimens without out-of-plane shear, their deformability and energy dissipation was significantly reduced when out-of-plane shear was applied (see Table 4).

- **Bent Wing**—The typical response of the bent wing connectors embedded in 2 in. (51 mm) panels is shown in Fig. 18. Initially, the connector showed nearly linear elastic response up to yielding of one of the legs. As the in-plane shear displacement increased, the response showed highly pinched hysteresis loops characterized by a nearly constant resistance of about 2.5 kip (11.1 kN) followed by a sharp increase in stiffness upon reloading. This behavior, observed in all test samples of this configuration, is similar to the behavior of high slenderness ratio cross-braces made of steel; where one anchor leg yields in tension (tension brace) while the other leg (compression brace) buckles in compression. Pre-opening of the joint did not significantly affect the cyclic response of the bent wing connector. While a lower strength was observed for Specimen CV4BW5T, the companion specimen (Specimen CV3BW4T) reached a strength comparable to or higher than that of the specimens without a joint opening (Specimens CV1BW2R and CV2BW3R). This observation suggests that the difference in strength may simply be due to natural variability in material strength. Minor variations in the thickness of the steel straps and cold working of the steel can cause changes in the strength of the connector. Failures in all bent wing connectors occurred by fracture of the strap near the bend. The response of the bent wing connector in 4 in. (102 mm) slabs was very similar to that observed in 2 in. (51 mm) panels (see Fig. 19). Except for Specimen CV1BW3R, the strength of the specimens was comparable to that measured in the 2 in. (51 mm) panels. As with other connectors, the data suggest that an initial joint opening did not have a sig-

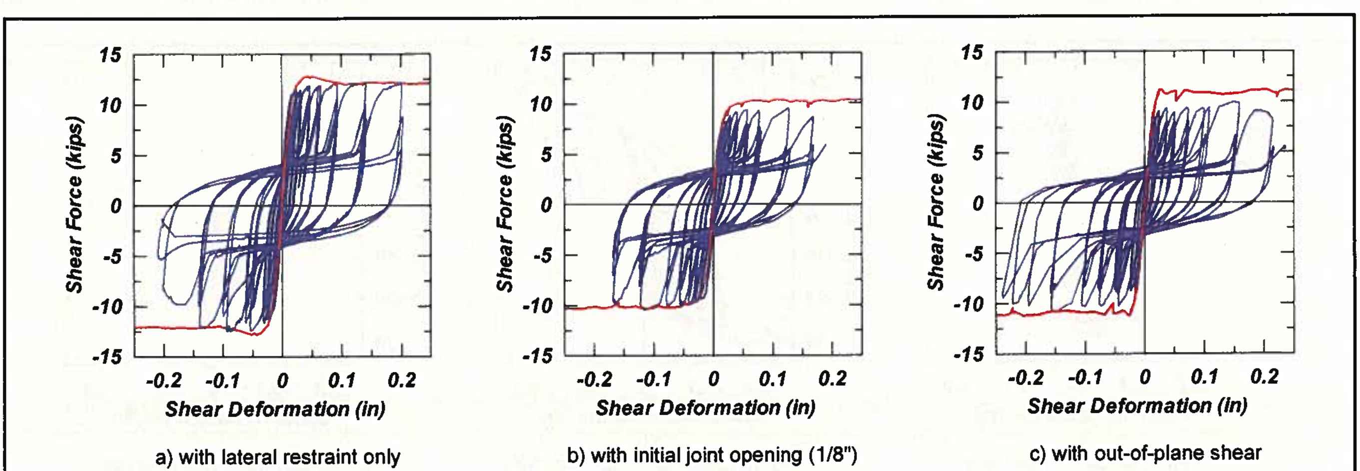

Fig. 19. Load and deformation response of the bent wing connectors in 4 in. flanges under cyclic loading.
Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

nificant influence on the strength or on the deformability of the bent wing connectors (see Fig. 19b). Two bent wing specimens were tested with concurrent in-plane and out-of-plane shear loads (Specimens CV5BW14R and CV6BW15R). Unlike the response of the structural tee connector, an out-of-plane shear load did not have a significant influence on the response of the bent wing connector. The strength, deformability, and shape of the hysteresis loops were comparable to those observed for the specimens without vertical load (see Fig. 19c).

- **M&A**—In the M&A connector, the cyclic in-plane shear loads caused failure of the welds between the mesh wires and the steel angle. As loading increased or cycled, the welds could be heard to "pop" as they progressively broke starting in the vicinity of the welded slug. Under cyclic loading, the connector's deformability was reduced severely, as compared to the monotonic loading, due to the different failure mechanism; however, the cyclic specimen exhibited nearly the same peak strength as seen in the monotonic loading (see Fig. 20). When joint opening was combined with cyclic in-plane shear, there was a slight (6 percent) reduction in strength of the M&A connector, but its response was nearly identical to that without joint opening.
- **Vector**—Under cyclic in-plane shear loading, the Vector connector exhibited the same strength and deformation characteristics as in the monotonic tests, though slightly more degradation in strength with deformation may be seen in Fig. 21a. It is noted that pinching of the hysteresis loops is much less pronounced for the Vector connector than it is for the bent wing specimen (see Figs. 19 and 21a). The stainless steel Vector connector behaved particularly well, with increased strength, deformability, and energy dissipation as shown in Fig. 21b.

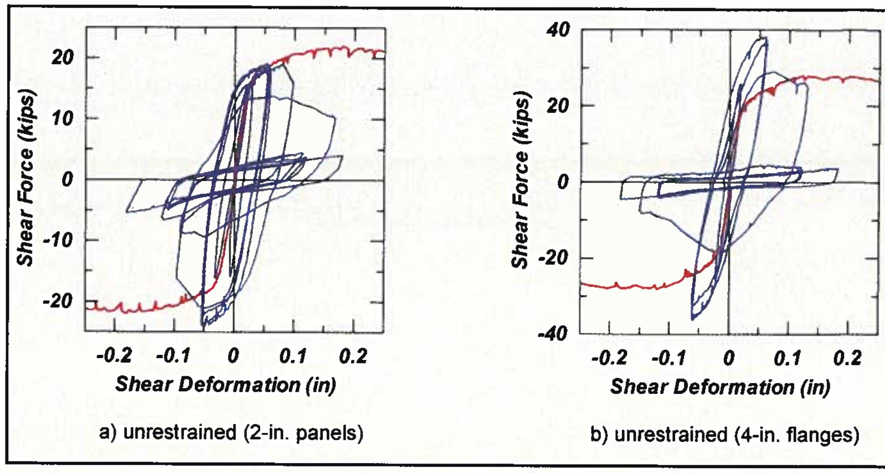

Fig. 20. Load and deformation response of the mesh and angle connectors under cyclic loading: (a) unrestrained (2 in. flanges); and (b) unrestrained (4 in. flanges). Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

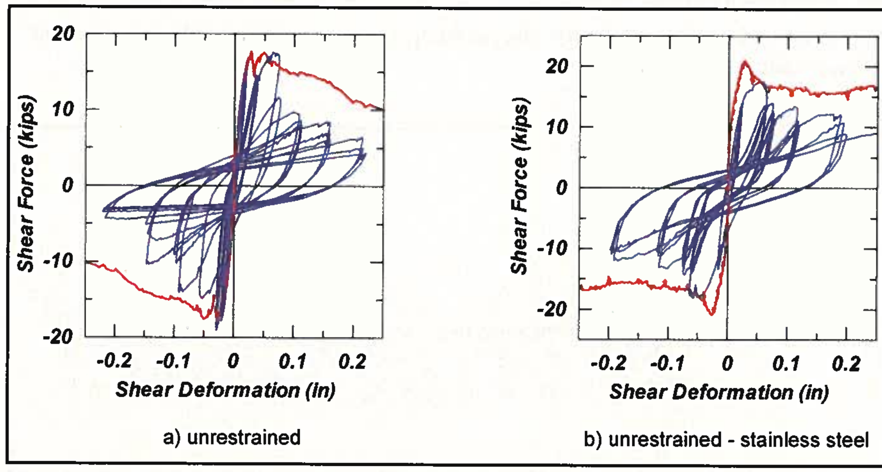

Fig. 21. Load and deformation response of the Vector connector in 4 in. flanges under cyclic loading: (a) unrestrained; and (b) unrestrained - stainless steel. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

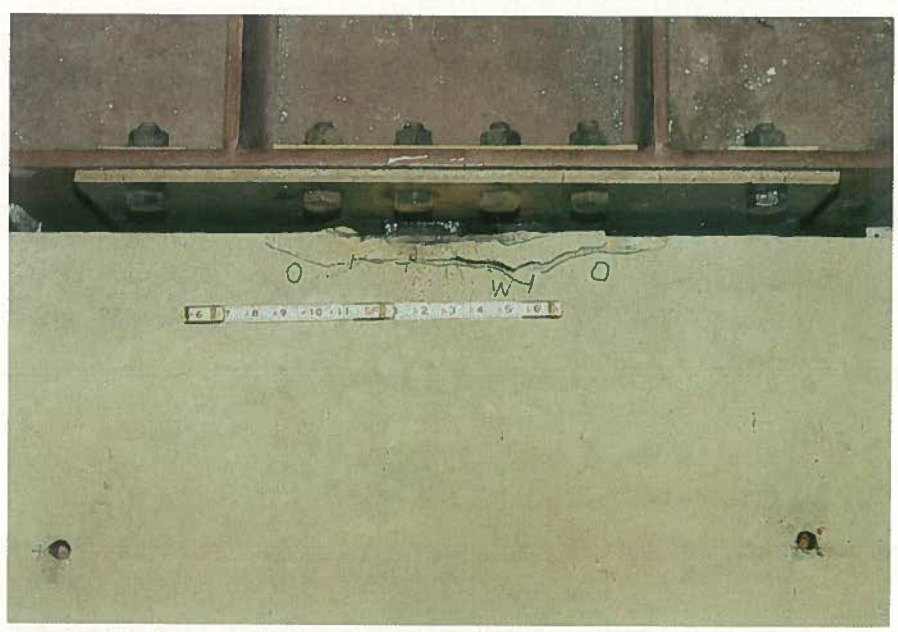

Fig. 22. Typical crack pattern after slug welding and pre-opening of the joint; panels with a hairpin connector.

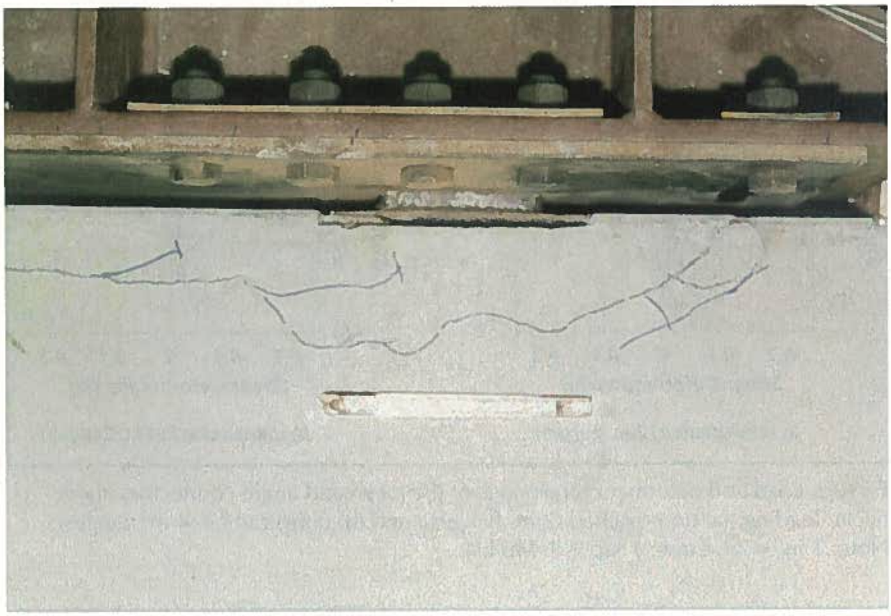

Fig. 23. Typical crack pattern after slug welding and pre-opening of the joint; panels with a structural tee connector.

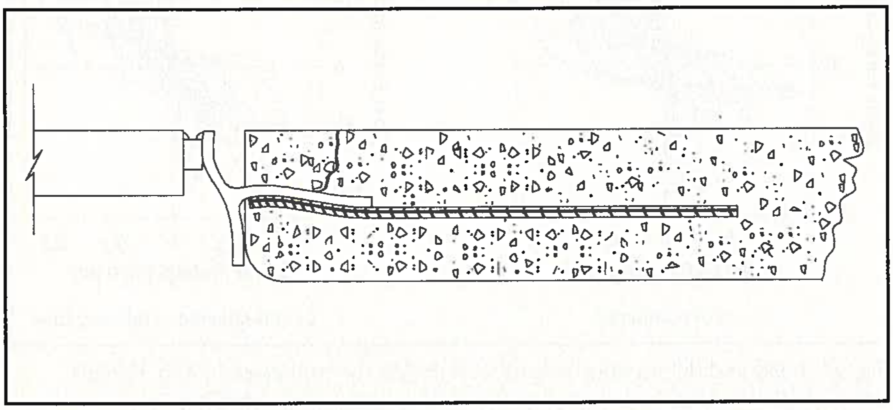

Fig. 24. Prying action in panels with a structural tee connector due to eccentricity between the slug weld and the reinforcing bars.

Fig. 25. Typical crack pattern at failure for specimens with a hairpin connector.

Crack Patterns

All specimens with hairpin connectors in 2 in. (51 mm) thick panels showed hairline cracks after welding of the slugs, just behind the plate and parallel to the panel edge. In contrast, none of the panels with stud-welded DBA, bent wing, structural tee, M&A, or Vector connectors showed this type of cracking.

Joint Opening—In the specimens with a hairpin connector, pre-opening the joint by applying tension across the joint caused existing cracks to propagate further and new cracks to develop (see Fig. 22). A similar crack pattern was observed for the stud-welded DBA connectors, but splitting cracks were also developed above each DBA.

The specimens with a structural tee connector also showed several cracks on the top surface of the panel upon opening of the joint (see Fig. 23). These cracks, however, occurred farther from the panel edge and are attributed to the eccentricity between the slug weld and the embedded reinforcing bars, which caused the stem of the tee to pry upward (see Fig. 24). These initial cracks may be the main cause of the reduced in-plane shear stiffness observed in the response of the specimens with joint opening, as noted earlier (see Fig. 17).

The specimens with a bent wing connector embedded in either 2 or 4 in. (51 or 102 mm) panels showed no cracks upon pre-opening of the joint. The Vector specimens, however, did develop cracks parallel to the edge of the flange, and 8 to 10 in. (203 to 254 mm) in length, when the joint was pre-opened 0.1 in. (2.5 mm).

Out-of-Plane Shear Load—A vertical shear of 1.5 kip (6.7 kN), either upward or downward, did not cause cracking of any of the specimens with a bent wing or a structural tee connector [vertical shear was not applied to the connectors in 2 in. (51 mm) panels].

In-Plane Shear Load—Nearly all specimens with a hairpin, stud-welded DBA, or structural tee connector developed bond-splitting cracks along the anchor bars upon in-plane shear loading, irrespective of the joint restraint or loading condition. Only seven of the twenty-one specimens with a bent wing connector, however, showed bond-splitting cracks along the embedded legs. Furthermore, the cracks were always shorter and narrower than those observed with the other connectors using the same concept for anchoring the face plate.

At failure, spalling of the concrete within the connector region was typical, except in those panels with the M&A connector. The extent of the damage depended on the connector type and the loading condition. Figs. 25 and 26 show photos of the crack pattern observed at failure for specimens with a hairpin and a bent wing connector, respectively, under in-plane shear.

In general, specimens with a bent wing, a Vector, or the M&A connector showed fewer cracks and less damage than those with other connectors under the same loading condition.

All specimens with a hairpin or a structural tee connector developed diagonal cracks when loaded with in-plane shear. These cracks appeared at a load of about 10 kip (44.5 kN) for the hairpin connector [in 2 in. (51 mm) thick panels] and at about 25 kip (111 kN) for the structural tee connector [in 4 in. (102 mm) thick panels, see Fig. 27].

In the specimens with a hairpin connector, the diagonal cracks were narrow and did not significantly influence the response nor the failure mode of the connector. On the other hand, diagonal cracking of the specimens with a structural tee connector was much more severe and often controlled the strength of the specimen (see Table 4).

In general, the cracking observed with the Vector connector was similar to that of the bent wing connector, except for the difference noted previously under tension opening. Under in-plane shear load, an initial crack would develop at a load between 15 and 17 kip (67 and 76 kN) just above the connector anchor leg and propagating along the leg 1 to 2 in. (25.4 to 51 mm) in from the free edge of the concrete. These cracks grew in length under continued loading.

Virtually no cracking developed in the M&A connectors until the connector capacity had been reached and the angle began slipping parallel to the panel edge.

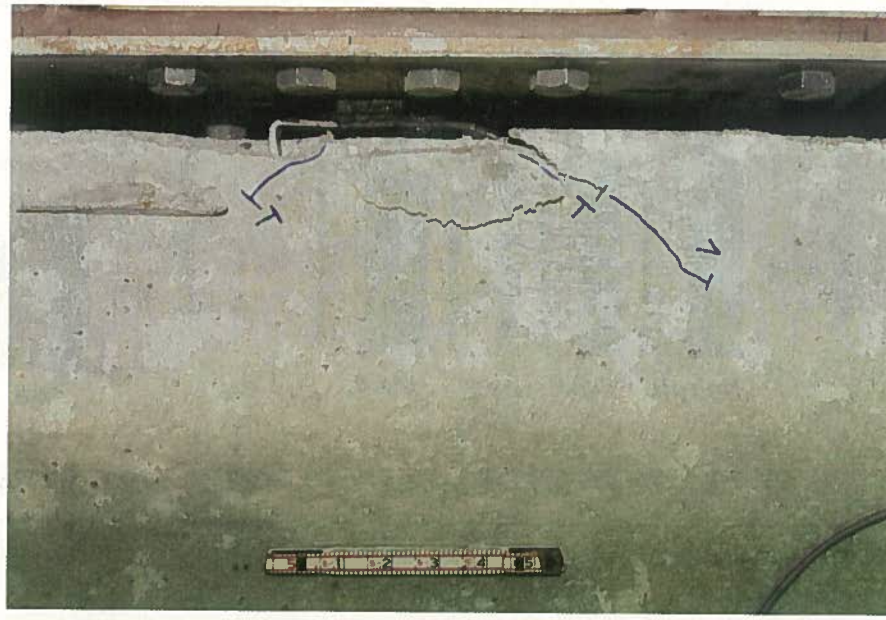

Fig. 26. Typical crack pattern at failure of specimen with a bent wing connector.

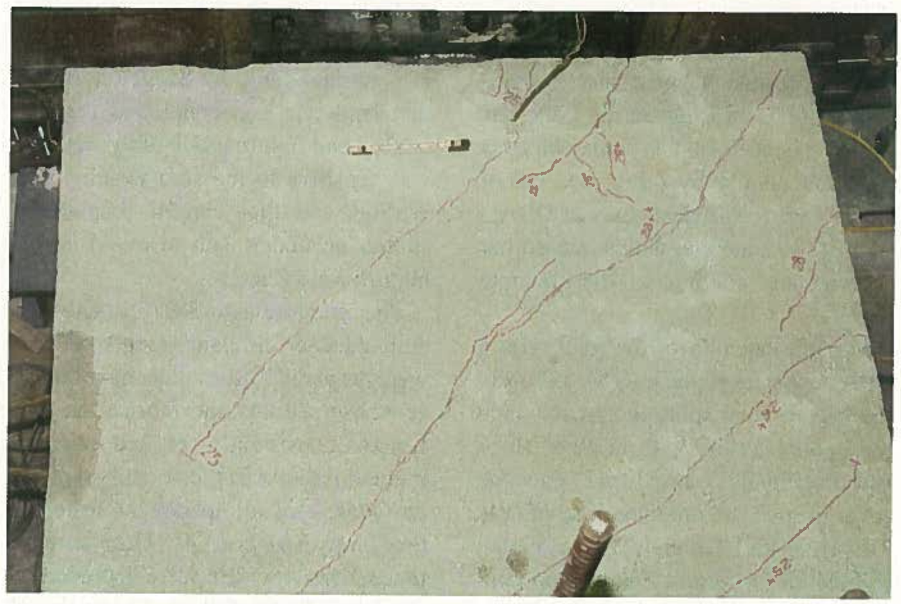

Fig. 27. Diagonal cracks in panels with a structural tee connector.

Failure Mode

The observed failure modes of the connectors can be classified into six main categories: anchor or leg fracture, plate fracture, weld fracture, anchor pullout, concrete spalling/crushing, and concrete flange shear cracking.

Hairpin—The typical failure mode of the hairpin connectors (see Table 2) was by fracture of the anchor (reinforcing bar) in tension near the weld. Two specimens showed, however, a different failure mode. Specimen SV3H-P3RT, tested with an initial joint opening of $^1/_{16}$ in. (1.6 mm), failed by bar pullout. Specimen SV3HP5RI, which had foam inserts, lost its resistance due to excessive spalling of the concrete cover over the anchor bars, but at a higher load than other hairpin connectors. It should be noted, however, that all specimens with a hairpin connector showed spalling of the concrete over the anchors, irrespective of the failure mode (see Fig. 25).

Stud-welded DBA—Two of the specimens with stud-welded DBA connectors failed by fracture of the plate-to-DBA weld (Specimens SV1SP1 and SV2SP2R). Specimen SV3SP4RI, with foam inserts at both edges of the plate, failed due to excessive spalling of the

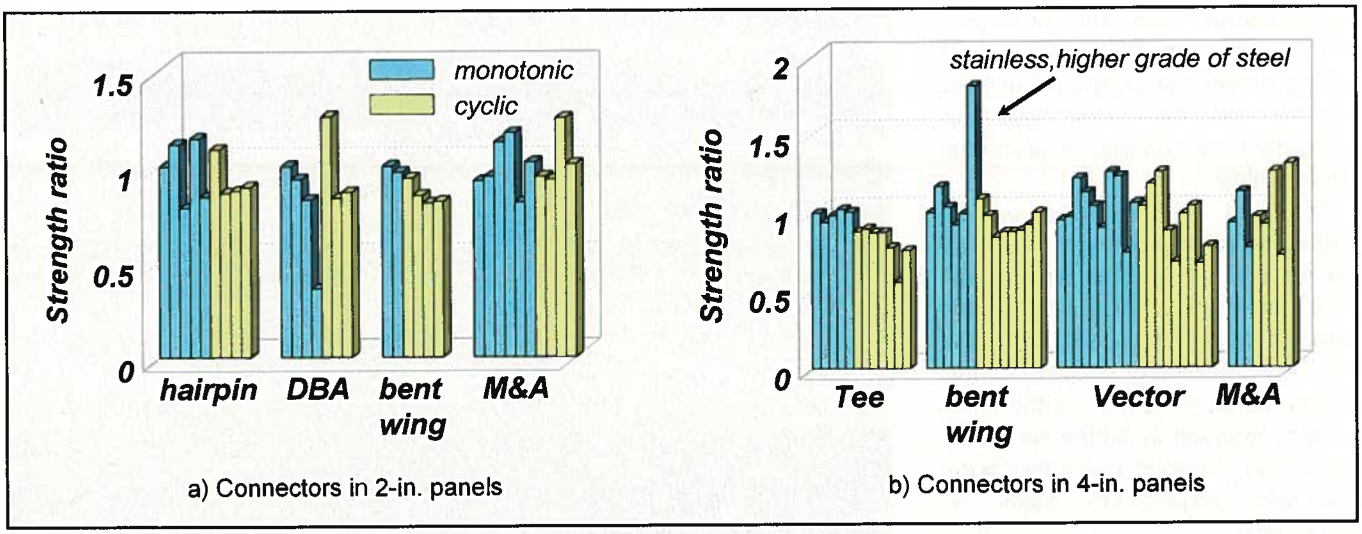

Fig. 28. Strength ratio: (a) Connectors in 2 in. flange panels; and (b) Connectors in 4 in. flange panels. Note: 1 in. = 25.4 mm.

concrete cover directly above the anchors. Note that this failure mode was also observed for the hairpin connector when foam inserts were present.

These results suggest that by preventing plate bearing on the concrete, shear resistance of the connector was provided mainly by direct bearing of the anchors (reinforcing bars or DBAs) against the concrete, which caused bar bending and spalling of the concrete cover.

In-plane opening of the joint with a DBA connector (Specimen SV4SP6RT) resulted in bond splitting cracks which propagated further with in-plane shear and eventually caused the concrete cover to spall off at a shear load of only 3.4 kip (15 kN). Failure of this specimen (SV4SP6RT) occurred as the top concrete cover spalled off above the anchor bars destroying their anchorage.

It must be mentioned that the strength and behavior of stud-welded anchors subjected to tension were investigated in a parallel study by Strigel et al.[10] In that study, it was concluded that the quality and failure mode of the anchors was sensitive to the stud welding gun settings and that careful preparation should be undertaken to avoid a premature weld fracture.

The stud-welded DBA connectors embedded in the panels reported here were prepared by the same manufacturer whose settings and fabrication procedures consistently resulted in yielding and fracture in the anchors—not in the welds—of all specimens tested in tension by Strigel et al.[10] Despite these precautions—which were intended to provide a ductile failure—seven out of nine specimens failed by fracture of the weld (see Table 3) rather than by yielding and rupture of the DBA connector.

Bent Wing—All bent wing connectors, embedded in either 2 or 4 in. (51 or 102 mm) thick panels, failed by fracture of the leg carrying tension adjacent to the knee bend. An identical failure was noted in the Vector connectors, but only under cyclic loading.

The bent wing leg carrying compression was bent significantly at the knee as shown in Fig. 26. It must be noted that, despite the small cover over the embedded legs, no pullout failures were observed in any of the bent wing specimens tested in this study. This result, when compared with the behavior of the Vector connector, indicates that the cross wire provided at the end of each leg of the bent wing connector

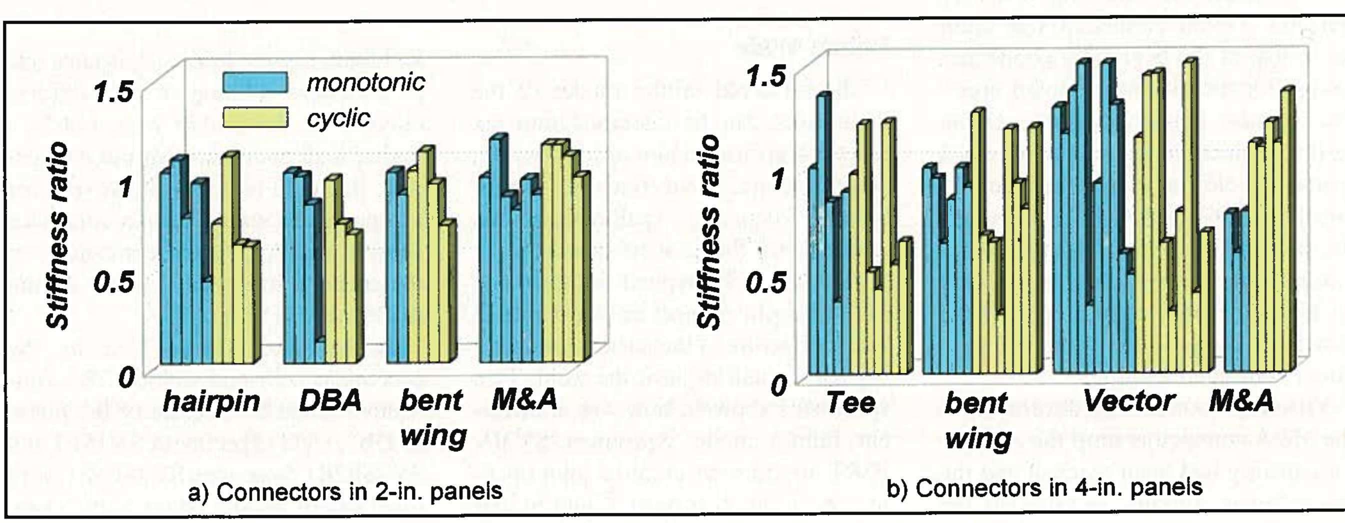

Fig. 29. Stiffness ratio: (a) Connectors in 2 in. flange panels; and (b) Connectors in 4 in. flange panels. Note: 1 in. = 25.4 mm.

(see Fig. 4) was effective in providing additional anchorage beyond that provided through bond, thus preventing pullout of the legs.

Structural Tee—The structural tee connector exhibited several different failure modes. The specimens with an initial joint opening (Specimens SV2STN4T, CV3ST5T, and C4ST7T) failed by fracture of the tension anchor bar. An out-of-plane upward shear force caused debonding of the anchors when combined with simultaneous in-plane cyclic loading (Specimen CV5ST9RU), while a downward shear force caused the bottom cover of concrete to spall off (Specimens CV6ST10RD and CV7ST11RD). The latter specimens showed abrupt failure as they were being loaded in the first in-plane shear cycle to a displacement ductility of 1.5 Δ_y (see Table 4).

The remainder of the structural tee specimens all experienced shear failure of the concrete flange. Following common practice, these panels were reinforced with a light mesh (6 × 6 - W2.1 × W2.1) and, therefore, these panels failed in shear soon after the development of shear cracks. Since the structural tee is a strong connector, the premature shear failure of the panels prevented the development of the connector's full strength. A similar failure might be expected between double tees when the total force transferred by the connectors exceeds the shear capacity of the concrete flange.

In practice, when double-tee flanges develop shear cracks, there may be a redistribution of shear forces within the diaphragm to less stressed areas or connectors, a condition that was not represented by the tests conducted in this investigation. Additionally, full-scale tests involving several connectors at a joint are needed to elucidate the behavior of such a system.

M&A—The M&A connector failure mode depended on whether the load was applied monotonically or cyclically, but it was similar in both the 2 and the 4 in. (51 and 102 mm) thick panels. Under monotonic load, a rupture surface developed in the steel angle around the perimeter of the slug weld. With cyclic loading, however, the mesh wires individually broke off the angle at their welds as the load was reversed.

Vector—The Vector connector also exhibited a different failure mechanism under cyclic in-plane shear loading. The monotonically loaded specimens failed by pullout of the tension anchor leg. In the cyclically loaded connections, failure was by rupture of the steel strap at the compound bend adjacent to the face plate (see Fig. 8). Repeated reversed bending of the strap, with the initial residual stress due to shaping, made this the weak point in the connector.

DISCUSSION OF TEST RESULTS

A direct comparison of the strengths of the different types of connectors has limited value because connector spacings in the diaphragm can be altered for added overall capacity. Of course, a direct comparison of strength may be important in evaluating the cost of using more or fewer connectors. Here, a comparison of the ability of the connectors to maintain their strength, stiffness, and deformability under the various loading and restraint conditions is presented.

In Figs. 28 through 31, the maximum measured strength, stiffness, deformation capacity, and ductility of all connectors subjected to in-plane shear are compared. In these plots, the strength and stiffness are shown as the ratio between the measured values divided by the average of the corresponding specimens tested under monotonic in-plane shear alone (i.e., without lateral restraint, out-of-plane shear, or in-plane joint tension opening).

Strength

Fig. 28 and the data shown in Tables 2 through 7 show that under cyclic in-plane shear loading, the strength of the connectors was generally reduced, but only modestly with respect to that measured under monotonic loading. Much larger variations in strength, however, were caused by lateral restraint, usually increasing strength, and by out-of-plane shear or joint opening, often causing a decrease in strength.

The stud-welded DBA connectors showed the more drastic variation in strength, with a reduction of up to 60 percent in a given test. The structural tee and hairpin connectors also show important but less drastic variation in strength, between 20 and 40 percent of average, respectively. The bent wing connector showed minimal fluctuations in strength with maximum reduction of only about 10 percent from average; the M&A and Vector connectors had less variation from average. Variation in strength results for the Vector connector are slight except where the stainless connector was subjected to combined

Table 7. Summary of results: average values for Vector® connectors.

Load condition	Joint restraint	Out-of-plane shear (upward or downward) (kip)	Panel thickness (in.)	In-plane shear strength (kip): Bond-splitting crack load	In-plane shear strength (kip): Yield strength	In-plane shear strength (kip): Peak strength	Initial stiffness (kip/in.)	Δ_{max} (in.)	μ	Failure mode
Monotonic Shear	No	No	4	N/A	17.4	19.8	1152	0.14	7.4	Anchor rupture or pullout
Cyclic Shear	No	No	4	N/A	16.4	20.3	1212	0.08	4.4	Anchor rupture
Monotonic Shear^{SS}	No	No	4	N/A	17.1	22.8	1202	0.63	37	Anchor rupture
Cyclic Shear	No	No	4	N/A	13.8	18.6	960	0.08	5	Anchor rupture

Note: SS denotes stainless steel plate; 1 in. = 25.4 mm; 1 kip = 4.448 kN; N/A denotes not applicable.

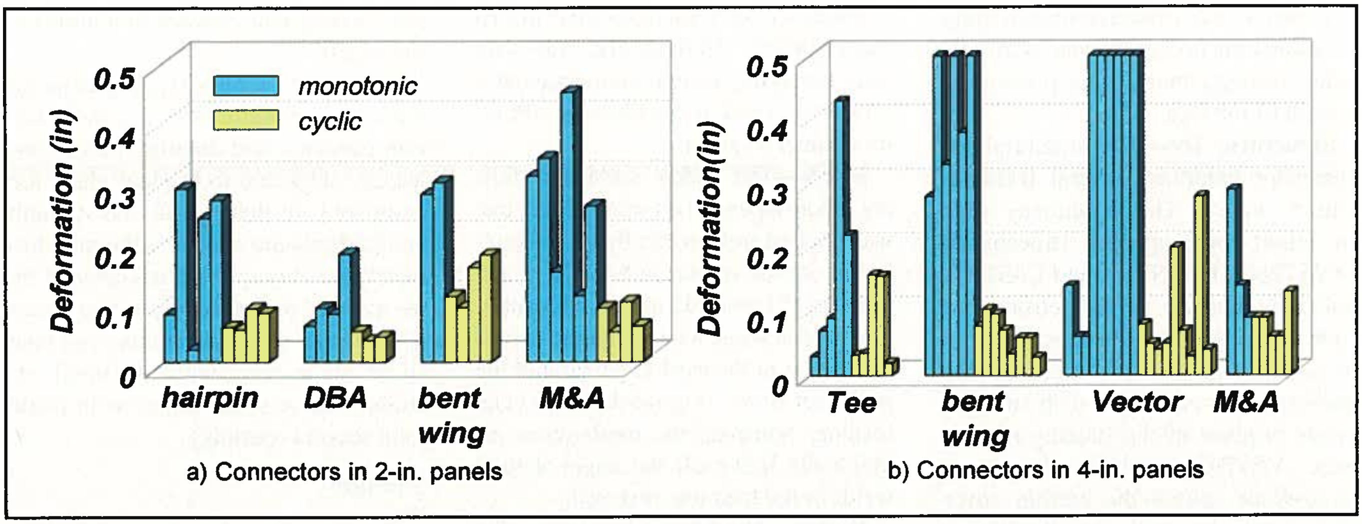

Fig. 30. Deformation capacity: (a) Connectors in 2 in. flange panels; and (b) Connectors in 4 in. flange panels. Note: 1 in. = 25.4 mm.

cyclic shear with initial opening where ratios of 67 and 78 percent of average were found.

Stiffness

The initial in-plane shear stiffness of the connectors showed large variations, irrespective of the connector type (see Fig. 29). Stiffness against in-plane shear is particularly sensitive to pre-opening of the joint (see Tables 2 through 5) because the face plate is pulled away from the concrete, reducing bearing and friction resistance. An in-plane shear stiffness variation from average of nearly 50 percent was observed for some specimens such as the structural tee, the bent wing, and the Vector connector in the 4 in. (102 mm) thick panels.

A 90 percent in-plane shear stiffness reduction for the stud-welded DBA connector was measured when a joint opening of $^1/_8$ in. (3 mm) was applied. As the DBA connector face plate is pulled away from the panel edge upon pre-opening (with in-plane tension) of the joint, the concrete bearing area alongside the connector face plate is reduced. As a result, the remaining in-plane shear stiffness of the DBA connector is provided mainly by bending of the anchors, with little or no contribution from plate bearing against the concrete.

The drastic change in the stiffness of the connectors due to multi-axial loading is a concern in design because of the uncertainty that exists in predicting the amount of in-plane opening or out-of-plane shear that may be acting in a joint. Based on the test results reported here, it seems prudent to compute the stiffness of the floor diaphragm using a plausible range of values to determine the effect stiffness has on prediction of diaphragm deformation.

Deformability and Ductility

Connector toughness is the ability of a connector to maintain its resistance through deformations beyond yielding. Toughness may be critical in allowing connected members to survive extreme loading conditions, in energy absorption, and in allowing load redistribution between connectors after initial yielding.

Current engineering knowledge and understanding of diaphragm behavior under extreme loads is insufficient to define the toughness necessary in a good connector. Though current research at

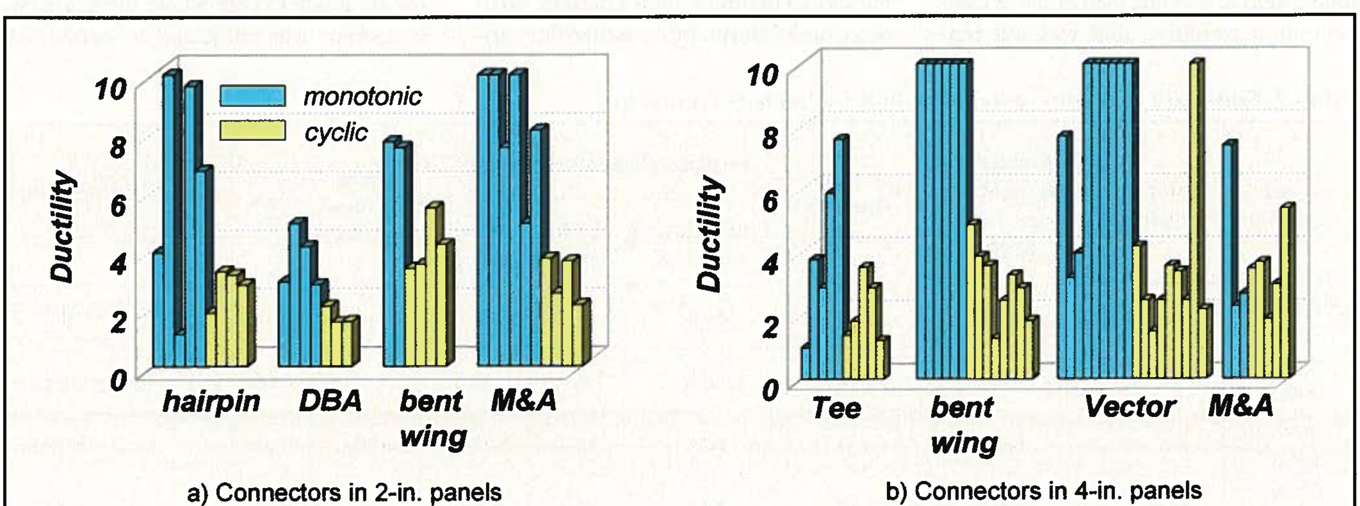

Fig. 31. Ductility: (a) Connectors in 2 in. flange panels; and (b) Connectors in 4 in. flange panels. Note: 1 in. = 25.4 mm.

Lehigh University and the University of Arizona is aimed at improving the understanding of inelastic behavior of precast diaphragms, designers may approach diaphragm design with an intention to maintain elasticity. Yet, even though floor diaphragms may be designed to remain elastic under extreme loads, toughness remains a desirable property if unexpected inelastic deformation occurs.

Figs. 30 and 31 show the in-plane shear deformation capacity, Δ_{max} (see Fig. 12), and displacement ductility of all connectors. For clarity, the plots have been terminated at a maximum shear deformation of 0.5 in. (12.7 mm) and a maximum ductility of 10, even though some connectors exceeded these values during testing (see Tables 2 through 5).

The data show significant variations in both deformation capacity and ductility depending on loading and restraint conditions. No general trends could be identified, except that the in-plane shear deformability and ductility was often drastically reduced under cyclic loading.

The large variation in the in-plane shear deformability and ductility exhibited by the connectors is of concern in design, as the connectors resisting in-plane shear may not have toughness when combined with simultaneous out-plane vertical shear or in-plane tension joint opening.

Under monotonic in-plane shear, the bent wing connector in both 2 and 4 in. (51 and 102 mm) flanges stands out as the most reliable for deformability and ductility of all of the connectors tested in this study. This connector showed dependable strength and stiffness and had very good deformability and ductility, especially when embedded in 4 in. (102 mm) thick flanges.

With cyclic loading, the bent wing connector also showed dependable strength, but its stiffness showed large fluctuations. Its deformability and ductility decreased significantly, mainly because of the highly pinched hysteresis loops exhibited under cyclic loading (see Figs. 18 and 19).

The Vector connector, however, demonstrated superior energy dissipation with reduced pinching of the hysteresis loops under cyclic in-plane shear. If the strap's anchor legs were tied into the concrete with a crossbar, as in the bent wing connector, the undesirable anchorage failures with monotonic in-plane shear could be avoided and better behavior could be obtained.

SUMMARY OF FINDINGS

In this study, various types of double-tee connectors were tested to failure under different load conditions. All connectors were subjected to horizontal in-plane shear under monotonic or reversed cyclic loading conditions. The effects of in-plane joint opening restraint and of a pre-existing tension opening of the joint on the in-plane shear performance were also considered. Out-of-plane vertical shear was simultaneously applied with in-plane shear in some tests. Based on the test results, the following observations are made:

1. Mechanical Characteristics of the Connectors Under Monotonic Shear

Of the connectors tested in 2 in. (51 mm) thick flange panels, the hairpin had the largest in-plane shear stiffness and strength [approximately 20 kip (89 kN)]. This connector displayed a moderate displacement ductility of about four (4) or larger.

The stud-welded DBA connector showed large variability and reached in-plane shear strengths of only 12 kip (53 kN). Its strength depended strongly on the quality of the stud weld between the DBA and the steel plate. This connector had an in-plane shear deformability similar to that of the hairpin but, again, it was strongly dependent on the quality of the weld. With an initial in-plane joint opening of $^1/_8$ in. (3.2 mm), the stiffness of the DBA connector was reduced to about one-tenth of its value without an opening and its strength was reduced to one-half.

The bent wing connector displayed excellent response characteristics under in-plane shear with a strength and stiffness lower than those of the hairpin, but with a larger ductility [a displacement ductility of seven (7) or more]. Unlike the severe concrete damage observed for the hairpin and the stud-welded DBA connectors, very little concrete cracking or damage was observed for the bent wing connector at ultimate shear load during any of the tests.

Among the connectors tested in 4 in. (102 mm) thick flange panels, the structural tee had both high in-plane shear stiffness and strength. The connector, however, lacked ductility. Damage to the double-tee flange at ultimate in-plane shear load was severe and included large diagonal cracks as well as spalling of the concrete directly above the stem of the structural tee. The M&A connector had a peak in-plane shear strength and stiffness very close to those of the structural tee.

The response of the bent wing connector in a 4 in. (102 mm) thick flange panel was nearly identical to that observed in the 2 in. (51 mm) panels in terms of in-plane shear stiffness and strength, but with even larger displacement ductilities. The Vector connector performed in a manner that was very similar to the bent wing.

2. Concrete Cracks Caused by Welding

All specimens using a hairpin connector showed hairline cracks after welding of the slug. This result is attributed to the small concrete cover of the reinforcing bar anchors [about $^3/_4$ in. (19 mm) or less] immediately behind the connector plate. Such a small cover could not sustain the thermal expansion and contraction generated upon welding of the slug, thus causing cracking of the concrete in the connector region.

In contrast, the panels with stud-welded DBA, bent wing, Vector, or M&A connectors had a larger depth of concrete between anchor legs and surface and did not show cracking upon welding of the slug. The 4 in. (102 mm) thick flange panels with increased anchor cover showed no cracking due to welding of the slug, irrespective of the type of connector used.

3. Serviceability

Bond-splitting cracks were usually observed along the embedded anchor bars or legs at an in-plane shear load of 7 to 10 kip (31 to 44.5 kN) for the hairpin and stud-welded DBA connectors in 2 in. (51 mm) thick panels. Similar cracking occurred at about

20 kip (89 kN) for the structural tee connector in a 4 in. (102 mm) panel. These loads correspond to approximately 60 to 70 percent of the ultimate in-plane shear strength of the connectors. The bent wing and Vector connectors did not exhibit bond-splitting cracks along the legs and showed only minor cracking up to the yield strength.

4. Anchorage of Embedments

Most connectors failed by fracture of the embedded leg, the face angle, or the weld under both monotonic and cyclic in-plane shear loading. Exceptions to this result include the premature fracture of the stud weld between the DBAs and the plate and the pullout of the Vector anchor leg.

Failure of the structural tee connector was limited by the shear capacity of the concrete flange. As a result, the adequacy of the provided development length for the bars used with the structural tee connector could not be fully evaluated. Continuity of the flange mesh in the M&A connector provided full anchorage in this system with resulting failures being in rupture of the angle or the mesh wire to angle weld.

5. Influence of Loading Condition

Concurrent Out-of-Plane and In-Plane Shear—Under concurrent in-plane shear and vertical shear of 1.5 kip (6.7 kN) acting either upward or downward, the response of the connectors before the onset of nonlinear behavior was similar to that observed under in-plane shear alone. The data show, however, that the ultimate strength and deformability of the connectors can be reduced in the presence of an out-of-plane load. An out-of-plane load was particularly detrimental to the stiffness, strength, and deformability of the structural tee connector.

The bent wing connector showed stiffness reductions, but its strength and deformability did not appear to be influenced by out-of-plane shear. Out-of-plane shear had no detectable effect on the in-plane shear strength of the M&A connector, but it did lower its deformation capacity. The Vector connector was not tested with concurrent vertical and in-plane shear.

Cyclic In-Plane Shear Load—Under cyclic load, most connectors showed no or only modest reductions of in-plane shear strength when compared with their counterparts subjected to a monotonic load. The in-plane shear deformation capacity and ductility of all connectors, however, showed significant reductions under cyclic load that varied depending on the type of connector, restraint against in-plane joint opening, amount of the applied out-of-plane shear, and in-plane joint opening.

Restraint Against Joint Opening—In-plane joint opening restraint had a favorable effect on the in-plane shear response of the connectors. Typically, the connectors' in-plane shear strength and deformation capacity increased when the joint was restrained against opening.

Joint Opening—An initial in-plane opening of the joint caused a reduction in the in-plane shear stiffness and strength of the connectors. Reductions of in-plane shear stiffness between 10 and 60 percent were observed depending on the connector type and the initial joint opening imposed on the specimens [either $^1/_8$ or $^1/_4$ in. (3.2 or 6.4 mm)—but double that amount in a real diaphragm with deformation in two connectors]. Reductions in strength were not as severe, on the order of 10 percent.

The deformation capacity of the units with an initial joint opening showed only modest reductions under monotonic loading. When subjected to cyclic in-plane loading, however, an initial joint opening resulted in large reductions of the in-plane shear deformability and ductility of the connectors, particularly when a $^1/_4$ in. (6.4 mm) opening was imposed [equal to $^1/_2$ in. (12.7 mm) in a joint with two connectors].

Foam Inserts—In some specimens, a $^1/_4$ in. (6.4 mm) thick piece of foam material was provided along the edges of the connector's face plate to prevent direct bearing against the concrete (hairpin, DBA, and structural tee connectors). This was done to avoid concrete spalling induced by bearing stress during the early stages of in-plane shear loading and to improve the shear ductility of the connectors.

The use of foam inserts prevented early spalling of the concrete in the connector region, as intended, but resulted in a measurable reduction of the in-plane shear stiffness of the connectors (about 10 to 20 percent reduction).

The in-plane shear strength remained about the same, but the ductility was reduced due to the increase in the deformation at yield as a result of the increase in the connector's initial elastic flexibility. For the structural tee connector, however, the deformation capacity increased by about 30 percent when an isolation foam pad was provided. In contrast, the isolation foam did not affect the deformation capacity of the hairpin or the stud-welded DBA connectors.

6. Overall Performance

Amongst all the connectors tested in this study, the bent wing connector was the most dependable in terms of in-plane shear strength and deformation capacity in both 2 and 4 in. (51 and 102 mm) thick flanges. Although not the strongest, it offers moderate in-plane shear strength [about 12 kip (53 kN)] and an excellent cyclic response with minimal cracking under service load levels.

The Vector connector could be equally dependable if it were anchored using a cross wire similar to the bent wing, preventing failure due to anchor leg pullout observed in the monotonic in-plane shear tests. Higher strengths could be achieved in the bent wing and Vector connectors by using thicker straps or higher strength materials than those used for the specimens tested in this study.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of this investigation and especially the response characteristics noted in the preceding "Summary of Findings," the following conclusions and recommendations can be made.

1. Cyclic in-plane shear loading causes a reduction of shear deformation capacity and ductility in normal mechanical connectors compared to the values expected from monotonic testing. Monotonic tests should not be used to evaluate connector re-

sponse to cyclic loads (such as those caused by seismic motion).

2. Under concurrent in-plane and out-of-plane (vertical) shear (acting either upward or downward), the response of connectors before the onset of nonlinear behavior is similar to that observed under in-plane shear alone. This result suggests, for example, that the wheel load in a parking structure would not affect the in-plane, elastic response of the diaphragm. However, the ultimate strength and deformability can be reduced by the presence of an out-of-plane load.
3. An initial in-plane opening of the joint, such as that created by volume change, causes a reduction in the in-plane shear stiffness and strength of connectors. The deformation capacity of the units with an initial joint opening will have modest reductions under monotonic loading. When subjected to cyclic in-plane loading, however, an initial joint opening can cause large reductions of the in-plane shear deformability and ductility of connectors.
4. Flange connectors with vertical face plates, parallel to the flange edge, are strongly preferred over slanted face plates. The vertical face plate prevents the erector from dropping a slug into the space between face plates until it jams into place, as occurs with slanted face plates. This forces the slug to be held in place at a specific location for welding. The deformability, ductility, and strength of a connector are all affected by the location of the weld slug relative to the location of the anchor legs on the face plate. The weld slug location was controlled in the tests described here, and should be controlled by the design engineer in double-tee diaphragms in practice.
5. A rectangular weld slug should be used. Experience has shown that a good fillet weld between a rectangular slug and a vertical face plate is much more likely to be obtained in the field than a good bevel-groove weld with a round bar slug.
6. For design purposes, it is recommended that 70 percent of the in-plane shear ultimate strength of the hairpin, stud-welded DBA, and structural tee connectors be used as a service load limit for these three connectors in 2 in. (51 mm) thick flanges. Higher loading caused cracking in the concrete flange. Such cracks are not only visually unacceptable to owners, but also allow the ingress of contaminated moisture into the concrete, which could eventually cause corrosion of the anchors. The bent wing and Vector connectors did not develop similar cracks in the concrete flange.
7. It can be concluded that the embedment length provided for the reinforcing bars, legs, and DBAs was generally adequate to develop the full strength of the anchors in both 2 and 4 in. (51 and 102 mm) flanges. For the bent wing connector, the cross wire provided as mechanical anchorage proved to be adequate to develop yielding and fracture of the connector near the face plate. If the Vector connector had a similar cross wire, the observed pullout failures would probably have been averted.
8. Engineers would be ill-advised to attempt to predict connector in-plane shear behavior through calculations. Connector performance was clearly shown to be quite variable as affected by numerous conditions such as: concurrent loading, joint restraint against opening, volume change deformations, concrete cover over anchor legs, and placement and welding of the connecting steel slug. Until our understanding of these mechanisms is further developed, design should be based on connector capacities proven through testing.

FUTURE WORK

This and past studies have shown that the in-plane shear strength and deformation capacity of connectors can be drastically reduced under combined in-plane shear and in-plane tension compared with those capacities under in-plane shear alone. Additional test data are still needed to better characterize the behavior of selected connectors under concurrent in-plane shear and in-plane tension (or compression).

Under certain load conditions in this study, the response of only one test specimen for a connector was measured. Thus, additional test replicates of connectors under these loading conditions are needed to corroborate the experimental results found in this study.

Future research programs should include tests of full or sub-assemblies of double-tee floor diaphragms with mechanical connectors subjected to a variety of load conditions. Such tests can provide information on the response of a floor system, such as load redistribution, that cannot be captured by testing individual connectors. The results of such tests would also serve as a basis for validating the development of linear and nonlinear numerical simulation procedures for precast floor diaphragms.

ACKNOWLEDGMENTS

The research reported in this paper was funded by the National Science Foundation (NSF) through Grant CMS-9412906. Initial work on this study was accomplished through the support of the Precast/Prestressed Concrete Institute through a Daniel P. Jenny Fellowship. Their support is gratefully acknowledged. The following companies also participated in funding portions of this study: JVI Inc., Ivy Steel and Wire, and Tindall Corporation.

Special thanks are extended to a consortia of PCI producers who contributed additional funding through the PCI Research and Development Council including: Atlanta Structural Concrete, Blakeslee Prestress Inc., Ferreri Concrete Structures, Metromont Prestress Co., Spancrete Industries, Spancrete Midwest Inc., and the Tindall Corporation.

The technical input provided by the industry advisory panel to the project is also gratefully acknowledged. Special recognition is due to the PCI JOURNAL reviewers who provided numer-

ous valuable comments, recommendations, and suggestions for presenting the acquired data clearly.

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 the entities mentioned here.

REFERENCES

1. Pincheira, J. A.; Oliva, M. G.; and Kusumo-Rahardjo, F. I., "Tests on Double-Tee Flange Connectors Subjected to Monotonic and Cyclic Loading," PCI JOURNAL, V. 43, No. 3, May-June 1998, pp. 82-96.
2. Zheng, W., "Analytical Method for Assessment of Seismic Shear Capacity Demand for Untopped Precast Double-Tee Diaphragms Joined by Mechanical Connectors," Ph.D. Thesis, The University of Wisconsin, Madison, WI, 2001, 313 pp.
3. *PCI Design Handbook – Precast and Prestressed Concrete,* Fourth Edition, Precast/Prestressed Concrete Institute, Chicago, IL, 1992.
4. Seo, C., "Behavior and Design of Shear Leg for Shear Connectors Between Double Tee Members," MS Report, University of Wisconsin, Madison, WI, May 1997, 93 pp.
5. Priestley, M. J. N., "The U.S.-PRESSS Program Progress Report," Third Meeting of the U.S.-Japan Joint Technical Coordinating Committee of Precast Seismic Structural Systems (JTCC-PRESSS), San Diego, CA, November 18-20, 1992.
6. Oliva, M. G., "Testing of JVI Flange Connectors for Precast Concrete Double-Tee Systems," Structural and Materials Test Laboratory Report, University of Wisconsin, Madison, WI, June 2000, 39 pp.
7. Oliva, M. G., "Pilot Testing of JVI V-2 Flange Connectors for Precast Concrete Double-Tee Systems," Structural and Materials Test Laboratory Report, University of Wisconsin, Madison, WI, June 2002, 12 pp.
8. Oliva, M. G., "Testing of the Shear-Loc Flange Connector for Precast Double-Tee Systems," Structural and Materials Test Laboratory Report, University of Wisconsin, Madison, WI, May 1999, 26 pp.
9. Oliva, M. G., "Tests on Tindall Bent Plate Flange Connector," Structures and Materials Test Laboratory Report, University of Wisconsin, Madison, WI, June 1997, 7 pp.
10. Strigel, R. M.; Pincheira, J. A.; and Oliva, M. G., "Reliability of 3/8 in. Stud-Welded Deformed Bar Anchors Subjected to Tensile Loads," PCI JOURNAL, V. 45, No. 6, November-December 2000, pp. 72-82.

APPENDIX—NOTATION

V_{max} = maximum measured load (see Fig. 12). Under cyclic loading, this value corresponds to the average of the values measured in both directions of loading.

V_y = equivalent yield strength (see Fig. 12)

Δ_y = yield displacement (see Fig. 12)

Δ^m_{max} = displacement corresponding to a strength loss of 20 percent of the maximum applied load under monotonic loading (see Fig. 12a)

Δ^c_{max} = displacement corresponding to a strength loss of 20 percent of the maximum applied load measured after three consecutive cycles at the same displacement amplitude (see Fig. 12b)

μ = Δ_{max}/Δ_y = displacement ductility