Experimental Behavior of Mechanical In-Flange Connectors

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Abstract: Prestressed double-Tee beams are used in multi-level parking structures and in some states' bridges. Steel brackets, called in-flange connectors, are embedded in the flanges of the beams and are welded together on-site to facilitate system behavior. To date, placement, i.e. spacing, of these connectors has been based primarily on engineering judgment although some guidelines are available. This paper summarizes the results of seventy (70) monotonic and twenty-three (23) reversed-cyclic tests on double-Tee in-flange connectors in an effort to provide comprehensive information on the relative performance of several different available connectors. Test results are reported for six different connectors manufactured by two different companies using seven different test protocols described herein. Each connector was imbedded in a 965x914x102 mm (38x36x4 in) concrete slab designed to be representative of the overhanging portion of the flange of a concrete double-Tee beam. The connectors were also tested using a slow reversed-cyclic displacement control protocol and the ability of each connector to dissipate/absorb energy is also discussed, along with the relationship to serviceability-related concrete damage, i.e. cracking and spalling.

Keywords: Mechanical flange connectors, double-Tee beams, reversed-cyclic testing, energy dissipation, damage.

INTRODUCTION

Precast parking structures often use prestressed double-Tee beams to make up the floor and roof systems. The beams are placed side-by-side and can be overlain with a topping slab on site whose purpose is to facilitate composite system behavior for the entire slab. Another common method for developing composite and/or system action is the use of a mechanical flange connecting device without the topping slab. Flange connectors are discretely positioned along the flange edge over the length of the beam as shown by the arrows in the schematic of Fig. **1**.

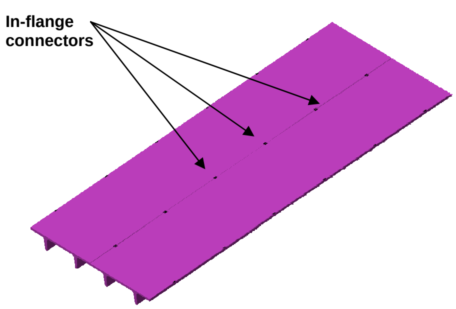

Fig. (1). Schematic showing the location of flange connectors on adjacent typical concrete double-Tee beams.

Structural damage to these has been noted following earthquakes, and a significant amount of serviceability issues have arisen at and around the mechanical flange connectors often caused by changes in humidity, settlement, and long-term deflection. There are different types of flange connectors available, most having the same basic design. Interestingly, there are no design specifications for spacing of double-Tee flange connectors; they are typically just spaced based on the design engineers' discretion or more often the manufacturers' suggestions. The existing database of connector performance is for a single manufacturer and consists of tens of test results under various test protocols. The results presented and discussed herein are intended to decrease this dearth in data and will approximately double the number of test results available to engineers (researchers and practitioners) and design code developers. The test protocols used in this study are slightly different from those previously used by other researchers which was the intent since there exists no standard for connector tests. One example of owner specification for mechanical connector spacing is the Texas Department of Transportation's suggested connector spacing of 1.6m (5 ft) for their double-Tee bridge construction, regardless of load or bridge details. This reinforces the notion that there is a need for published test results to provide information for non-proprietary code development related to mechanical flange connectors.

To date, few experimental studies investigating monotonic or cyclic behavior of in-flange connectors have been completed. Two of the initial such studies were by Venuti [1] and Aswad [2]. Venuti's results showed that there was less than 3% difference in ultimate strength for rebar connectors when the slabs were prestressed, thus consistent with not prestressing the slabs used in the present study. He also showed that the connector arm's angle of entry into the concrete slab had virtually no effect on the ultimate strength. More recent testing has been conducted by [3-6]. The majority of these latter tests were proprietary in nature, but helped to significantly further the understanding of the behavior of double-Tee flange connectors, hence they are mentioned here. This paper presents the results of 93 tests on six different types of in-flange connectors embedded in the side of 965x914x102mm (38x36x4in) concrete slabs in an effort to provide information on serviceability-related damage to the concrete near the connector. Eventually, studies such as the one presented here may help lead to damage-limiting design criteria for such connectors.

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OBJECTIVE AND SCOPE

There were six major objectives for this study:

(1) To provide a performance range for several different connector types subjected to monotonic and reversed-cyclic loading.

(2) Compare the performance of a 100mm (4in) thick monolithic slab with a connector to a 50mm (2in) slab topped several days later with another 50mm (2in) layer, both having the same type of connector. This specimen will be described in more detail later and is a prototype specimen specific to this study.

(3) Determine if the reversed-cyclic displacement protocol significantly affected test results through qualitative comparison of the hysteresis to previous studies on the same connector.

(4) Investigate the out-of-plane, i.e. vertical shear, behavior of the connectors to monotonic and reversed-cyclic loading.

(5) Examine the amount of energy the various connectors dissipated/absorbed during reversed cyclic testing in-plane (horizontal shear) and out-of-plane (vertical shear).

(6) Qualitatively compare the ability of the connectors to dissipate energy to the amount of damage sustained by the concrete slab surrounding the connector.

EXPERIMENTAL SETUP

In order to perform the tests described herein a test jig was fabricated which consisted of two large steel plates that clamp together by pretensioning 25mm (1in) diameter bolts. The boundary conditions, i.e. restraints and supports, are described in detail later in the test results section of this paper.

TEST SPECIMENS

Some previous studies have used rectangular concrete slabs in an attempt to model the behavior of double-Tee in-flange connectors. This is also the approach that was used in the present study. Fig. **2** shows the dimensions of the slab and also the location of the steel connector and reinforcing mesh within the slab; all dimensions are shown in mm. The connectors were approximately the same size, but steel type, angles, and some details varied. Table **1** provides a material description as well as some of the geometry for the various connectors. In addition to the descriptions and photographs provided in Table **1** the following differences should be noted:

(1) Connectors A-E had two 7.5mm (1/4in) raised surfaces approximately 25mm (1in) and 75mm (3in) from the end of each leg, which is shown in the photograph in Table **1**.

(2) Connectors A-E had a smooth transition, i.e. a twist, from the face of the flange to each leg, also shown in Table **1**.

(3) Connectors F and G had approximately a turn near the end of each leg, which can be seen in the lower figures of Table **1**.

(4) Connectors F and G had an angular transition from the face of the flange to each leg. This consisted of two angles to provide the change in orientation of the

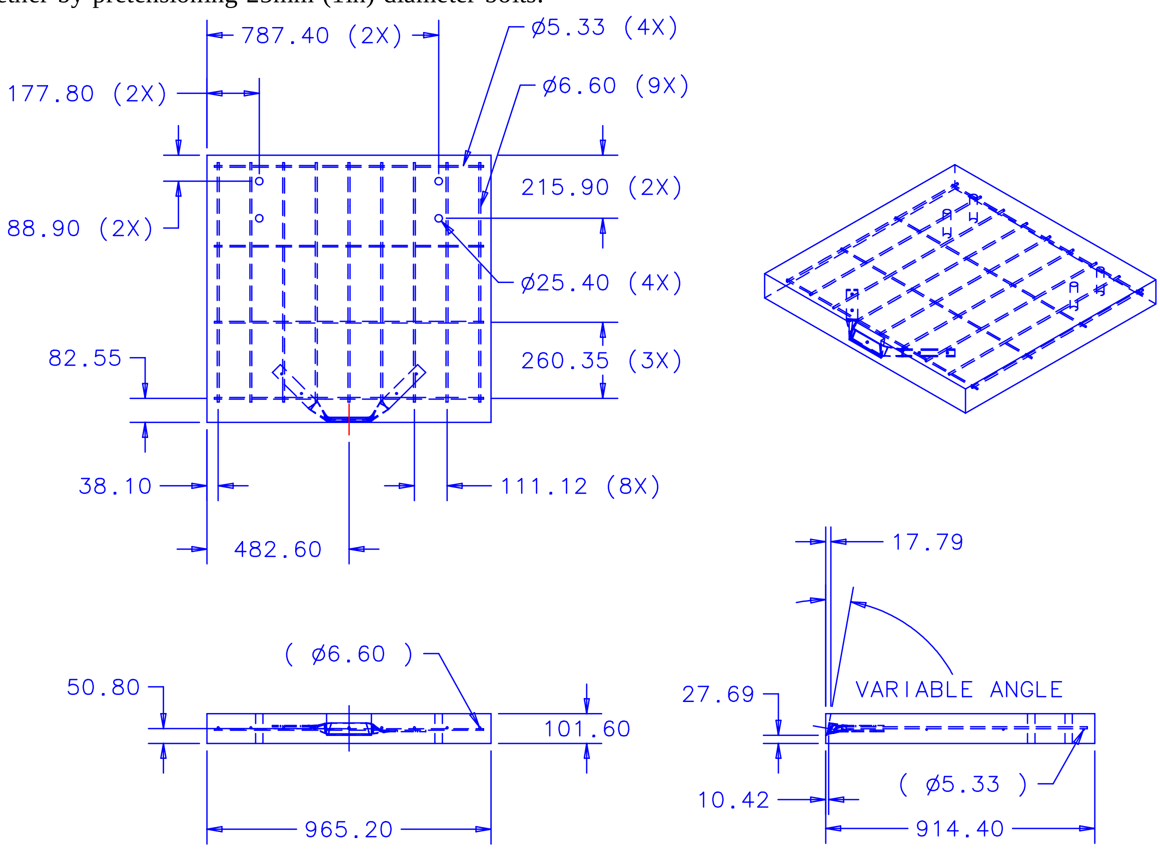
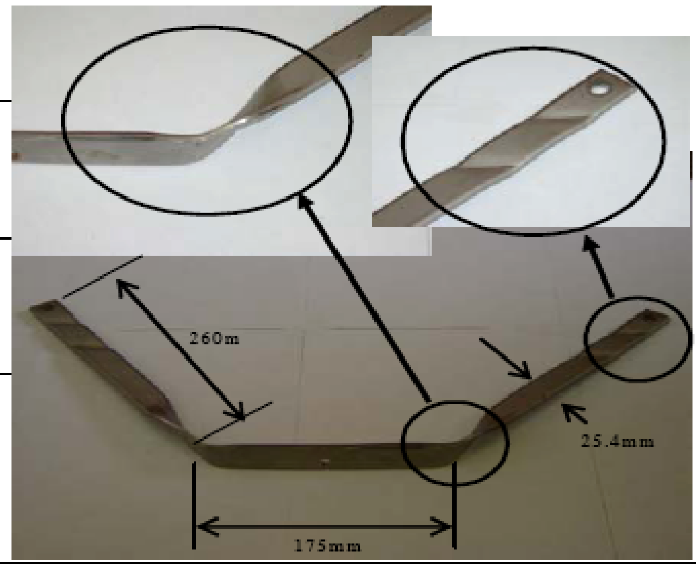
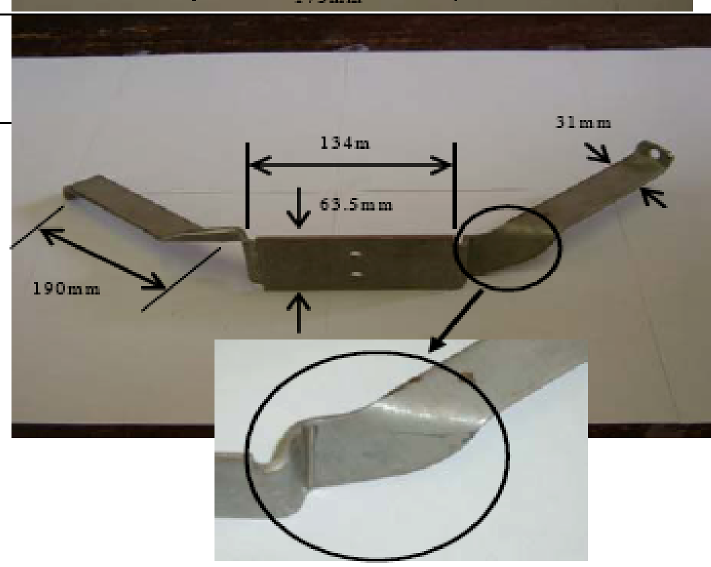


Fig. (2). Concrete slab/connector details including steel reinforcing mesh.

Table. 1. Material and Geometric Descriptions of the Connector Tests

MATERIAL DESCRIPTION		
STEEL TYPE	AISI 304 / UNS S-30400 (Stainless)	ASTM A-36 / ASME SA-36 (Mild)
CONDITION	ANNEALED	HOT ROLLED
TENSILE STRENGTH MPa(ksi)	585 (85)	400 to 550 (58 to 80)
YIELD STRENGTH MPa (ksi)	207 (30)	248 (36)
% ENLONGATION IN 50.8mm (2in)	40%	23%
CONNECTOR DESCRIPTION		
Connector A	AISI 304 steel connector with 25.4mm (1.0in) wide face-plate (shown)	
Connector B	AISI 304 steel connector with 25.4mm (1.0in) wide face-plate and offset legs (not shown)	
Connector D	ASTM A-36 connector with 35.3mm (1.3875in) wide face-plate (not shown)	
Connector E	ASTM A-36 connector with 35.3mm (1.3875in) wide face-plate and offset legs (not shown)	
Connector F	AISI 304 connector with 63.5mm (2.5in) wide face-plate and offset legs (shown)	
Connector G	AISI 304 connector with 50.8mm (2.0in) wide face-plate and offset legs (not shown)	

steel leg and is shown close-up in the photograph in Table **1**.

(5) Connector C is identical to connector A. The only difference between these connectors is that connector A was embedded in a monolithic slab while connector C was embedded in a 50mm (2in) slab and a 50mm (2in) slab was poured on top several days later. This type of separated pouring is specific to the present study and has not been investigated before. A secondary objective is to see if there is a significant increase in performance for this type of pour.

EXPERIMENTAL RESULTS

The "cracking load" and "spalling deflection" are also provided in an effort to characterize the performance of the connectors with regards to serviceability. For example, in several cases a four to five kilogram piece of concrete fell from the specimen. Although the connector was often still anchored at its ends and reasonably intact, in the eyes of the building owner, this is clearly not felt to be adequate performance of their structure. Cracking was identified *via* timed videotape which was correlated with the load and deflection data for each test. Cracking was defined in this study as identifiable to the naked eye from approximately 1.5m away. Spalling was also identified in this manner when some portion of concrete released, fell, or was clearly separated from the specimen.

All measurements reported and discussed in this paper were measured using the LVDT in the actuator. This LVDT was re-calibrated just prior to beginning this test program.

Monotonic Horizontal Shear Tests

Recall that for several of the connectors there was some horizontal asymmetry, i.e. the connector leg was above the reinforcing mesh on one side and below the mesh on the other side. Therefore, each of the connector types were tested by pulling to the left and pulling to the right, designated horizontal shear left and horizontal shear right, respectively. The actuator is bolted directly to a steel plate that was welded to the flat face of the steel connector. The test protocol for the both of these tests was also a linear displacement control test procedure. The actuator was displaced a total of 38.1mm (1.5in) in 180 seconds, producing a constant deformation rate of 0.212 mm/sec (0.00833 in/sec). Fig. **3** presents the load displacement results for a typical connector for shear right and shear left, with a schematic in the corner showing the boundary conditions during this type of test.

Table **2** provides the interested reader with values for all the connectors tested in horizontal shear and includes the concrete strength, cracking load, deflection at ultimate load, ultimate load, and deflection at first spalling. These values are discussed in more detail later.

Monotonic Vertical Shear Tests

The test protocol for the vertical shear up and vertical shear down was also a linear displacement control procedure. The actuator was displaced a total of 25.4mm (1 in) in 120 seconds. The load-deformation plots for a typical vertical shear up and vertical shear down test is presented in Fig. **4**. As with the other test types, within that plot is a schematic showing the boundary conditions for the specimens tested in

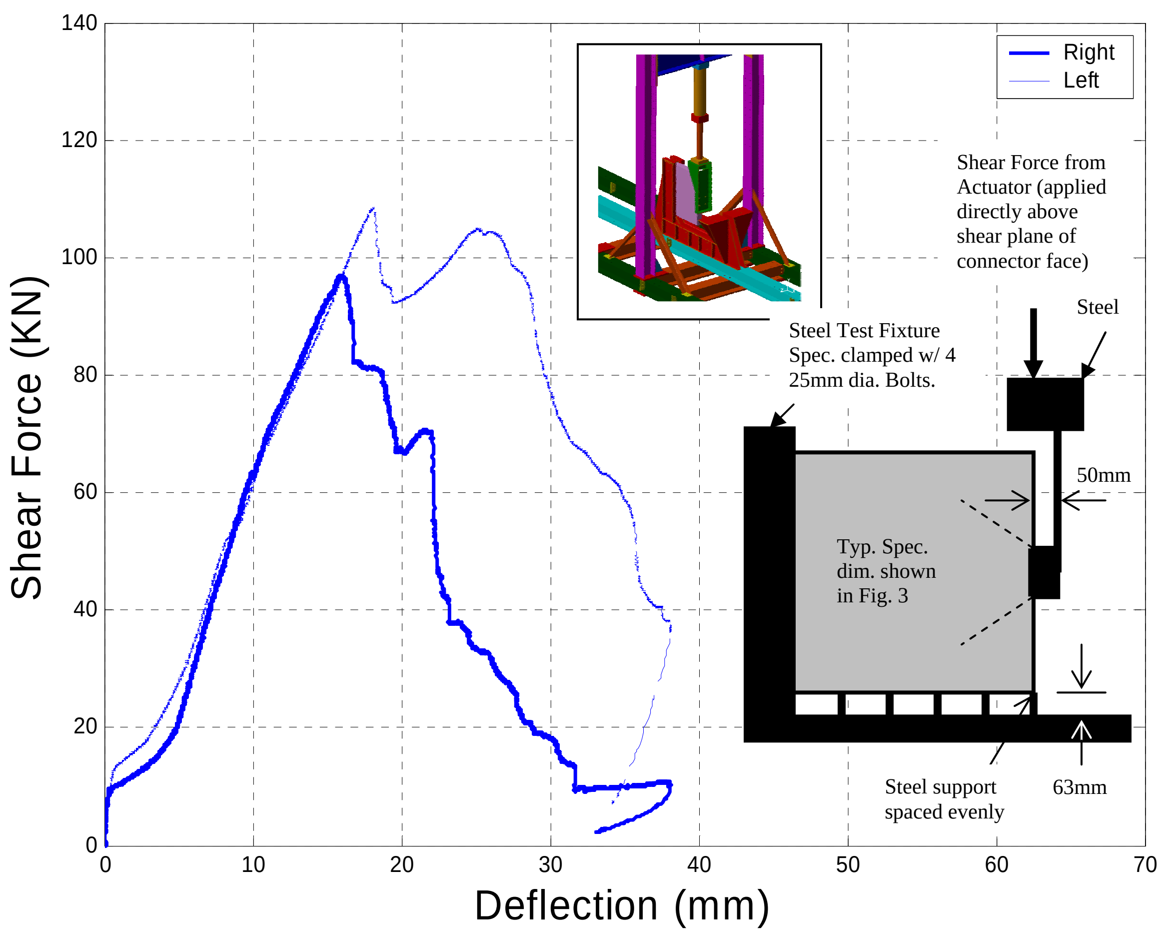


Fig. (3). Load-deflection relationship for a typical horizontal shear test. Schematic showing tests setup with boundary conditions for horizontal shear tests.

vertical shear. Additional graphical results and detailed descriptions of all tests can be found in a comprehensive report by van de Lindt and de Melo e Silva [7].

Monotonic Tension Tests

The ability of an in-flange connector to accommodate tensile load is important during traffic loading, earthquakes, wind, changes in humidity, and settlement. This test protocol consisted of displacing the actuator a total of 38.1mm (1.5in) in 180 seconds and was felt to be slow enough to be considered a near static test, as with the shear tests previously described. It should be noted that most specimens were significantly damaged before the test was completed at the final displacement of 38.1mm (1.5 in). Fig. **5** presents the load-deflection results for a typical tension test as well as details related to the boundary conditions, e.g. supports, during the test.

Shapes were approximately the same regardless of connector type and, again, all graphical results for each connector can be found in van de Lindt and de Melo e Silva (2003). The tabular summary of the results is also presented under test type "tension pull" in Table **2**.

REVERSED-CYCLIC TEST RESULTS

In order to quantify the performance of the connectors under load reversals, a series of reversed cyclic tests were performed on each of the connectors previously described. With this in mind, each of the connectors was tested in duplicate using the reversed-cyclic displacement protocol presented in Fig. **6**. As indicated on the figure, two full cycles at each displacement level were used. Of course, there are numerous reversed-cyclic test protocols in existence, however it was felt that two however it was felt that two cycles at each level would provide sufficient information regarding stiffness, strength degradation, and energy dissipation, for the objectives of this study. Due to many similarities in hysteresis shape among the various connectors, as well as to space limitations, only several of the reversed-cyclic test results are presented graphically. As mentioned previously, graphical results for all the connectors tested can be found in [7]. Table **3** presents additional results for all the

Reversed-Cyclic Horizontal Shear Tests

As noted above, selected hysteresis plots are presented. Connector A had a significant asymmetry between the left and right direction as can be seen by its hysteresis in Fig. **7**.

Some spalling occurred on the top of the slab, but none on the bottom. Toward the end of the test, following the point at which the load reached ultimate, one leg of the connector fractured. Connector G, whose hysteresis is presented in Fig. **8**, behaved similarly to the same type of connectors tested using a different reversed-cyclic protocol by [5] even though boundary conditions and loading protocol were not identical. One noticeable difference was the displacement at which ultimate was reached. The displacement at ultimate was slightly greater for the present tests than that of the pre-

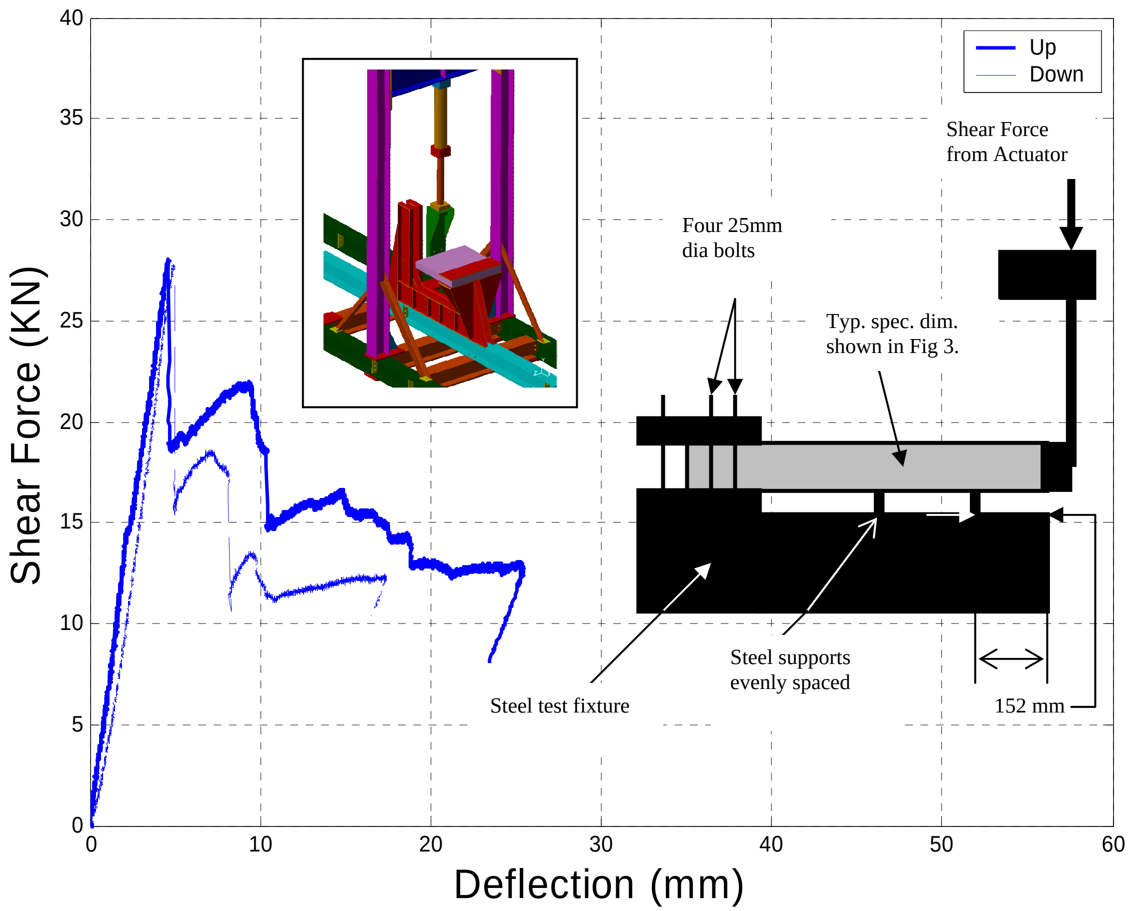


Fig. (4). Load-deflection relationship for typical vertical shear test. Schematic showing tests setup with boundary conditions for vertical shear tests.

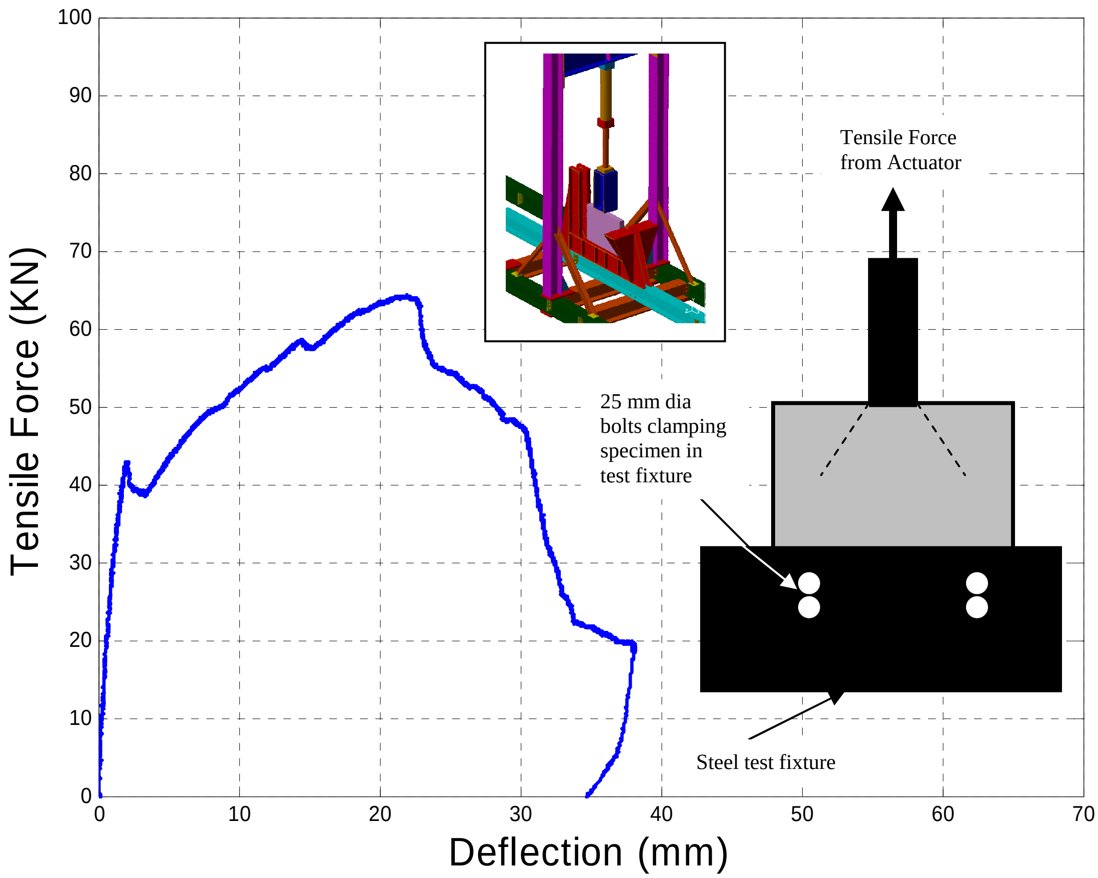


Fig. (5). Load-deflection relationship for tension tests. Schematic showing test setup with boundary conditions.

vious study. Interestingly, the monotonic results for the same type of connector were very similar for the two studies. This connector appeared to provide excellent energy dissipation with hysteresis loops that did not pinch as much as many of the other connectors.

Reversed-Cyclic Vertical Shear Tests

The displacement control protocol for the reversed-cyclic vertical shear tests consisted of two positive and two negative cycles at increasing displacements, similar to the reversed-cyclic horizontal test protocol. Of particular interest during reversed cyclic tests was a connectors' ability to dissipate a significant amount of energy prior to spalling. Unfortunately, much of this energy can be dissipated in the form of cracks and friction, which often causes significant service related damage to the concrete, and eventually damage to the reinforcement as a result of moisture seepage.

Connector A reached an ultimate load of 44.5kN (10 kips) during the vertical shear reversed cyclic-testing as shown in the hysteresis plot of Fig. **9**. Spalling for this specimen occurred on about the 10th cycle and consisted of a 3 to 5 kg piece of concrete. Damage was somewhat typical for the specimens tested in vertical reversed-cyclic shear discussed below and some additional test details and damage description can be found in Table **3**.

Recall that connector C was the specimen that was topped with a 50mm (2 in) slab several days after the initial pour. The connector was positioned near the top of the initial pour. This connector performed very well as apparent in Fig. **10**.

Note the highly asymmetric hysteresis, a difference between the positive and negative ultimate capacities of almost 50%. A detail of the test specimens is that one leg of the connector is set under the steel reinforcing mesh and the other connector leg on top. This was the setup for the connectors indicated as having offset legs in Table **1** and a picture of this just prior to pouring is shown in Fig. **11**. It should be noted that there was a slight difference in concrete compressive strength for the bottom and top layer. This hysteresis shape was asymmetric, but was consistent from specimen to specimen.

Energy Dissipation

The ability of a connector to dissipate energy during reversed-cyclic loading is an important measure of its ability to perform adequately during an earthquake. Unfortunately, deformation goes hand-in-hand with energy dissipation, hence service-related damage often is associated with good energy dissipation. Ideally, one would like a connector with a high stiffness and reasonable ductility. This would provide maximum energy dissipation during load reversals with a minimal level of damage to the concrete in which the

Table 2. Results Summary for Seventy (70) Monotonic Connector Tests

Test Type	Specimen Type & Number	Concrete[1] Strengh MPa (ksi)	Cracking Load kN (k)	Ultimate Deflection Mm (in)	Ultimate Load kN (k)	Spalling Deflection mm (in)
VERTICAL SHEAR UP	A-03	59.2 (8.59)	24.9 (5.59)	17.8 (0.7)	54.9 (12.3)	NA[4]
	A-12	59.2 (8.59)	26.9 (6.05)	10.7 (0.42)	40.6 (9.12)	NA[4]
	B-01	57.9 (8.40)	23.3 (5.23)	12.7 (0.5)	34.0 (7.65)	NA[4]
	B-08	57.9 (8.40)	17.3 (3.88)	7.4 (0.29)	26.3 (5.91)	NA[4]
	C-08	55(8)/64(9.3)[3]	58.3 (13.1)	22.4 (0.88)	60.5 (13.6)	24.6 (0.97)
	C-09	55(8)/64(9.3)[3]	39.2 (8.8)	16.8 (0.66)	52.1 (11.7)	NA[4]
	D-08	66.5 (9.7)	27.1 (6.1)	10.9 (0.43)	34.3 (7.7)	18.8 (0.74)
	D-09	66.5 (9.7)	16.0 (3.6)	4.6 (0.18)	28.0 (6.3)	14.2 (0.56)
	E-09	66.5 (9.7)	34.7 (7.8)	13.2 (0.52)	37.8 (8.5)	16.8 (0.66)
	E-05	66.5 (9.7)	36.5 (8.2)	13.5 (0.53)	36.9 (8.3)	15.2 (0.6)
	F-03	57.9 (8.40)	23.6 (5.3)	9.4 (0.37)	37.4 (8.4)	21.3 (0.84)
	F-12	57.9 (8.40)	NA[4]	4.6 (0.18)	28.0 (6.3)	NA[4]
	G-05	59.2 (8.59)	31.6 (7.1)	8.1 (0.32)	37.4 (8.4)	9.9 (0.39)
	G-12	59.2 (8.59)	20.9 (4.7)	9.4 (0.37)	28.9 (6.5)	14.7 (0.58)
VERTICAL SHEAR DOWN	A-13	59.2 (8.59)	35.2 (7.9)	11.9 (0.47)	35.2 (7.9)	20.1 (0.79)
	A-14	59.2 (8.59)	16.9 (3.8)	10.4 (0.41)	41.8 (9.4)	NA[4]
	B-13	57.9 (8.40)	39.2 (8.8)	10.9 (0.43)	40.0 (9.0)	18.5 (0.73)
	B-14	57.9 (8.40)	15.6 (3.5)	9.4 (0.37)	40.9 (9.2)	NA[4]
	C-07	55(8)/64(9.3)[3]	10.7 (2.4)	6.4 (0.25)	17.4 (3.9)	NA[4]
	C-06	55(8)/64(9.3)[3]	17.8 (4.0)	5.1 (0.2)	24.5 (5.5)	14.2 (0.56)
	D-07	66.5 (9.7)	31.2 (7.0)	8.9 (0.35)	37.4 (8.4)	11.9 (0.47)
	D-06	66.5 (9.7)	30.3 (6.8)	14.0 (0.55)	42.3 (9.5)	18.5 (0.73)
	E-13	66.5 (9.7)	37.8 (8.5)	6.35 (0.25)	40.0 (9.0)	14.5 (0.57)
	E-14	66.5 (9.7)	15.1 (3.4)	10.4 (0.41)	39.6 (8.9)	10.9 (0.43)
	F-06	57.9 (8.40)	---[2]	---[2]	---[2]	---[2]
	F-07	57.9 (8.40)	24.0 (5.4)	4.83 (0.19)	27.6 (6.2)	9.4 (0.37)
	G-07	59.2 (8.59)	---[2]	---[2]	---[2]	---[2]
	G-14	59.2 (8.59)	30.7 (6.9)	9.1 (0.36)	32.9 (7.4)	16.3 (0.64)
HORIZONTAL SHEAR LEFT	A-05	59.2 (8.59)	86.3 (19.4)	20.3 (0.8)	92.1 (20.7)	NA[4]
	A-11	59.2 (8.59)	87.7 (19.7)	15.5 (0.61)	100.1 (22.5)	15.5 (0.61)
	B-11	57.9 (8.40)	100.1 (22.5)	25.4 (1.0)	103.2 (23.2)	NA[4]
	B-12	57.9 (8.40)	103.2 (23.2)	31.5 (1.24)	118.8 (26.7)	NA[4]
	C-01	55(8)/64(9.3)[3]	NA[4]	14.5 (0.57)	88.11 (19.8)	NA[4]
	C-10	55(8)/64(9.3)[3]	NA[4]	18.8 (0.74)	92.1 (20.7)	NA[4]
	D-04	66.5 (9.7)	80.5 (18.1)	16.8 (0.66)	84.6 (19.0)	22.4 (0.88)
	D-01	66.5 (9.7)	93.0 (20.9)	15.0 (0.59)	93.0 (20.9)	16.5 (0.65)
	E-10	66.5 (9.7)	95.2 (21.4)	18.3 (0.72)	95.7 (21.5)	25.4 (1.0)
	E-08	66.5 (9.7)	89.4 (20.1)	19.6 (0.77)	101.5 (22.8)	24.1 (0.95)
	F-05	57.9 (8.40)	75.2 (16.9)	11.2 (0.44)	78.3 (17.6)	NA[4]
	F-09	57.9 (8.40)	35.1 (7.9)	19.1 (0.75)	99.7 (22.4)	13.2 (0.52)
	G-01	59.2 (8.59)	99.7 (22.4)	27.9 (1.1)	110.8 (24.9)	NA[4]
	G-10	59.2 (8.59)	95.2 (21.4)	18.0 (0.71)	108.1 (24.3)	NA[4]

(Table 2) contd…..

Test Type	Specimen Type & Number	Concrete[1] Strengh MPa (ksi)	Cracking Load kN (k)	Ultimate Deflection Mm (in)	Ultimate Load kN (k)	Spalling Deflection mm (in)
HORIZONTAL SHEAR RIGHT	A-01	59.2 (8.59)	95.2 (21.4)	35.3 (1.39)	98.8 (22.2)	NA[4]
	A-02	59.2 (8.59)	NR	26.2 (1.03)	103.2 (23.2)	NA[4]
	B-04	57.9 (8.40)	78.3 (17.6)	15.5 (0.61)	89.9 (20.2)	NA[4]
	B-05	57.9 (8.40)	NA[4]	14.0 (0.55)	96.6 (21.7)	NA[4]
	C-02	55(8)/64(9.3)[3]	71.6 (16.1)	20.1 (0.79)	80.5 (18.1)	34.5 (1.36)
	C-05	55(8)/64(9.3)[3]	64.5 (14.5)	34.3 (1.35)	106.8 (24.0)	NA[4]
	D-02	66.5 (9.7)	NA[4]	16.0 (0.63)	86.3 (19.4)	SF[5]
	D-10	66.5 (9.7)	NA[4]	19.1 (0.75)	87.2 (19.6)	SF[5]
	E-04	66.5 (9.7)	---[2]	17.3 (0.68)	103.7 (23.3)	---[2]
	E-02	66.5 (9.7)	88.1 (19.8)	18.3 (0.72)	103.7 (23.3)	---[2]
	F-04	57.9 (8.40)	56.1 (12.6)	22.4 (0.88)	71.6 (16.1)	NA[4]
	F-01	57.9 (8.40)	70.3 (15.8)	17.5 (0.69)	88.1 (19.8)	NA[4]
	G-11	59.2 (8.59)	69.9 (15.7)	11.9 (0.47)	82.3 (18.5)	NA[4]
	G-02	59.2 (8.59)	94.8 (21.3)	16.0 (0.63)	97.0 (21.8)	31.5 (1.24)
TENSION PULL	A-04	59.2 (8.59)	27.6 (6.2)	6.6 (0.26)	44.9 (10.1)	NA[4]
	A-09	59.2 (8.59)	53.0 (11.9)	7.9 (0.31)	62.7 (14.1)	19.8 (0.78)
	B-03	57.9 (8.40)	26.3 (5.91)	16.3 (0.64)	36.7 (8.25)	18.0 (0.71)
	B-02	57.9 (8.40)	NA[2]	14.7 (0.58)	58.3 (13.1)	NA[4]
	C-12	55(8)/64(9.3)[3]	55.2 (12.4)	23.9 (0.94)	65.0 (14.6)	25.4 (1.0)
	C-11	55(8)/64(9.3)[3]	---[2]	34.5 (1.36)	62.7 (14.1)	22.4 (0.88)
	D-05	66.5 (9.7)	SF[5]	6.1 (0.24)	36.5 (8.21)	SF[5]
	D-03	66.5 (9.7)	---[2]	9.7 (0.38)	46.3 (10.4)	22.9 (0.9)
	E-11	66.5 (9.7)	43.3 (9.74)	8.9 (0.35)	46.7 (10.5)	32.0 (1.26)
	E-03	66.5 (9.7)	31.3 (7.04)	6.6 (0.26)	43.9 (9.87)	23.1 (0.91)
	F-02	57.9 (8.40)	9.9 (2.23)	13.2 (0.52)	23.6 (5.31)	20.6 (0.81)
	F-11	57.9 (8.40)	29.1 (6.53)	18.0 (0.71)	57.9 (13)	18.0 (0.71)
	G-08	59.2 (8.59)	38.9 (8.76)	34.3 (1.35)	71.2 (16.0)	21.3 (0.84)
	G-04	59.2 (8.59)	39.7 (8.92)	22.1 (0.87)	64.1 (14.4)	32.5 (1.28)

[1]Concrete compressive strength is reported as the average of three 4 x 8in cylinders for each pour - tested at approximately equal intervals throughout the connector tests.
[2]An equipment error of some type occurred during testing, e.g. video tape problem occurred so cracking load could not be identified.
[3]Since the *Connector C* slabs consisted of pours several days apart, both concrete compressive strengths are provided.
[4]This value was not applicable (NA) because it did not occur during the test.
[5]Steel connector failed (SF) prior to concrete cracking or spalling. This occurred frequently for the connectors made from ASTM A-36 steel.

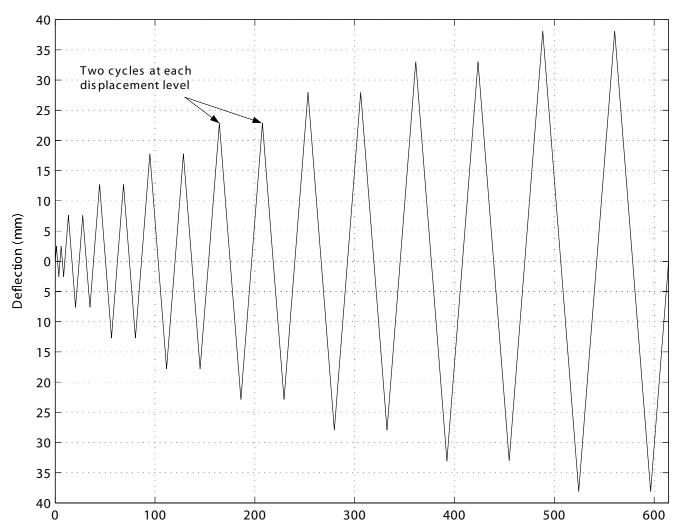


Fig. (6). Horizontal reversed-cyclic test protocol used in this study connectors tested using this protocol. Note that the boundary conditions were identical to those presented in the monotonic tests.

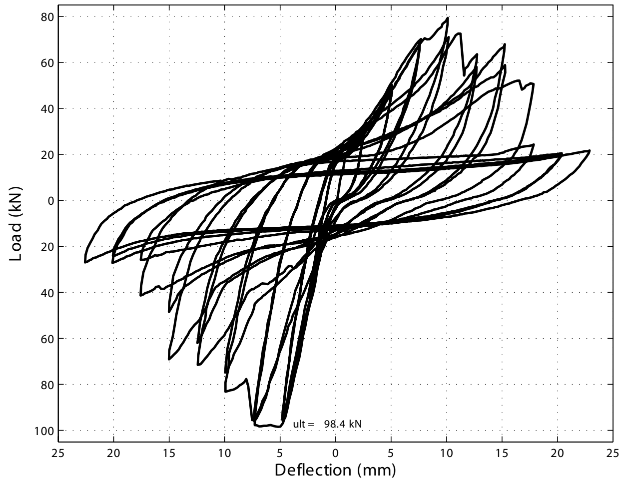


Fig. (7). Hysteresis for Connector A in reversed cyclic horizontal shear.

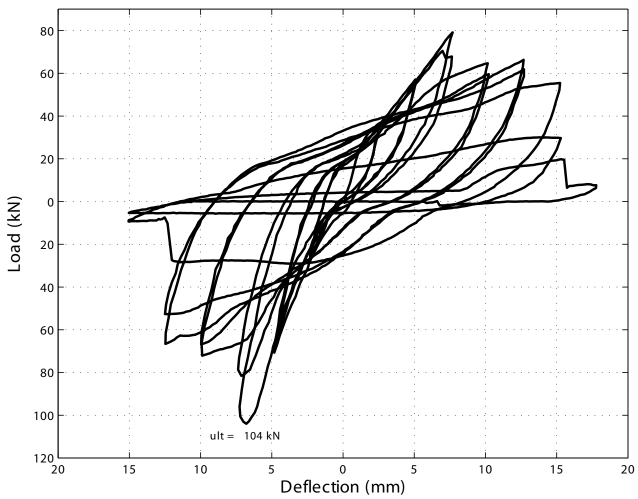


Fig. (8). Hysteresis for Connector G in reversed cyclic horizontal shear.

connector is embedded. The energy that is absorbed/dissipated by the connector can be calculated as where dE is the incremental energy (area of hysteresis) and t is time.

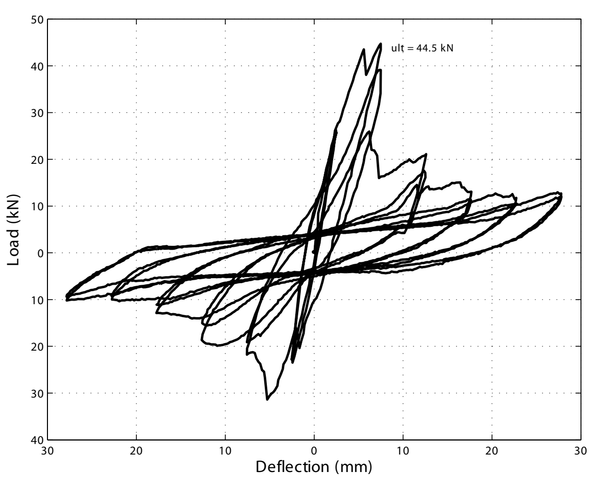


Fig. (9). Hysteresis for Connector A in reversed cyclic vertical shear.

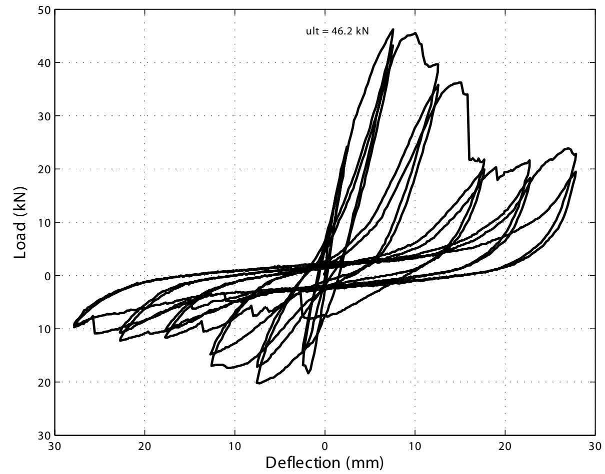


Fig. (10). Hysteresis for Connector C in reversed cyclic vertical shear.

Fig. (11). Picture showing orientation with respect to steel reinforcing mesh for connectors with offset legs (as indicated in Table **1**).

Connector C was able to dissipate the most energy. At low levels of displacement, most of the connectors absorbed approximately the same amount of energy, i.e. within approximately 25% of one another. Using the integral expression described above it was found that connector C was also able to dissipate a significant amount of energy during the reversed-cyclic vertical shear tests. Perhaps the most important result is the inverse relationship of energy dissipation to sustained service damage. The photographs in Fig. **12** shows the sustained damage to four connectors following the reversed cyclic tests. The connectors that dissipated the most energy are shown in Figs. **12b** and **12d** and appear to have sustained a great deal more damage to the concrete slab than the connectors that dissipated the least amount of energy (Figs. **12a** and **12c**). This pattern was typical for all the specimens tested during this study. Photographs and detailed damage descriptions of all specimens are available in a comprehensive report by [7]. Although the steel connector itself is almost destroyed in Fig. **12a**, the slab only released a few kilograms of concrete and has a large half-moon crack. This is a moderate level of damage considering the steel connector was displaced horizontally over 37.5mm (1.5in) on the

Table 3. Results Summary for Twenty-Three (23) Reversed-Cyclic Tests

Test Type	Specimen Type and Number	Ultimate Load kN (k)	Damage/Test Description – Notes[1]
Horizontal Shear	A-10	98.4 (22.1)	Significant asymmetry between right and left direction observed. Some minor spalling observed and one leg of the connector fractured near end of test.
	B-09	105.9 (23.8)	No spalling during this test.
	C-04	103.2 (23.2)	Following ultimate, a large decrease in load capacity was observed on the next cycle but not on subsequent cycles.
	D-11	90.7 (20.4)	Very low ductility, fractured at approximately 10mm (0.4 in). Very little damage occurred to the slab because of the low displacement level.
	D-12	89.0 (20.0)	No damage to concrete. Both sides of connector fractured at the same time.
	E-01	101.9 (22.9)	Showed about 50% more deformation capacity than the other mild steel connector, D. Only one side of the connector fractured.
	E-12	91.2 (20.5)	Significant damage was done to the concrete slab. One side of connector fractured.
	F-08	110.0 (24.7)	Hysteresis very similar to tests conducted by Oliva (2000) using a different reversed-cyclic protocol, Both sides of connector fractured near end of test.
	F-10	71.6 (16.1)	Significant spalling occurred. Enough to allow one side of connector to completely release from concrete.
	G-03	104.5 (23.5)	Behaved very similar to connectors tested by Oliva (2002) using a different reversed-cyclic protocol. Both sides of connector fractured near end of test.
	G-09	103.2 (23.2)	Same as specimen G-03.
Vertical Shear	A-06	40.0 (9.0)	Strength degradation very significant immediately following ultimate. Spalling on top and bottom.
	A-07	44.5 (10.0)	Spalling of a 2 to 3 kg concrete piece.
	B-06	39.1 (8.8)	Spalling occurred at approximately 15mm of displacement on the bottom only.
	C-13	46.2 (10.4)	Only very slight spalling occurred.
	C-14	51.6 (11.6)	Spalling of a 2 to 3 kg concrete piece.
	D-13	40.9 (9.2)	One side of connector fractured. Major spalling occurred on bottom at two different times during the test.
	D-14	36.5 (8.2)	Connector plate between actuator and connector fractured. This occurred following ultimate, but before spalling occurred.
	E-06	40.0 (9.0)	Very large piece of concrete fell from bottom of specimen, approximately 10 to 15 kg. Spalling also occurred on top of specimen.
	E-07	36.0 (8.1)	Spalling on top and bottom of specimen, approximately 5 kg.
	F-13	44.9 (10.1)	Ultimate occurred on upward cycle. Spalling on top and bottom.
	F-14	42.3 (9.5)	Very large piece of concrete spalled from the slab edge where the connector was located. The connector was still anchored.
	G-13	48.5 (10.9)	Face of slab where connector located damaged significantly. Connector still anchored as with connector F.

[1]As mentioned in the text, full damage descriptions and pictures of the specimens following testing are available in [7].

last cycle. However, in Fig. **12b**, the steel reinforcing mesh is partially exposed and the connector is visible within the slab. A large amount of concrete spalled during this test. The mild steel connector, connector D, only resulted in some moderate damage to the concrete slab as shown in Fig. **12c**. Recall that the vertical reversed-cyclic protocol called for a final displacement cycle of almost 28mm (1.1in). In contrast to Fig. **12c** is the damage sustained by the slab as a result of the vertical reversed cyclic test on connector C. Although this specimen dissipated the most energy, one can see from Fig. **12d** that the slab is significantly damaged; although it is still held together by the mesh reinforcement. Overall, for the twenty-three (23) specimens tested under reversed-cyclic displacement protocol, the pattern was similar.

Fig. (12a). Concrete slab damage sustained as a result of testing Connector B in horizontal reversed cyclic shear.

Fig. (12b). Concrete slab damage sustained as a result of testing Connector C in horizontal reversed cyclic shear.

Fig. (12c). Concrete slab damage sustained as a result of testing Connector D in vertical reversed cyclic shear.

Fig. (12d). Concrete slab damage sustained as a result of testing Connector C in vertical reversed cyclic shear.

DISCUSSION

As previously mentioned, the steel connectors were tested such that ultimate capacity was reached but not necessarily fracture of the connector. The role played by the concrete during loading of the specimen was primarily to provide anchorage for the connector, including linkage with the steel reinforcing mesh. It is hoped that this study (and future connector test results) will provide a sound basis for the development of damage-limiting specifications for mechanical flange connectors.

Cracking

Cracking occurred during all the vertical monotonic shear tests, and during most of the horizontal monotonic shear tests. The major exceptions were both connector C specimens tested in horizontal shear to the left, which did not crack. Cracking loads for vertical shear were fairly consistent with the exception of the connector C specimens, which had a mean cracking-load exceeding 44 kN (10 kips).

The most important test to identify cracking is perhaps in-plane tensile loading of the connector. Here the highest load needed to cause cracking, usually of the side at the flange, i.e. slab edge, was exhibited by the connector C specimen. Most of the connectors had cracking loads ranging from 26 to 53 kN (6 to 12 kips).

For the reversed-cyclic tests almost all specimens cracked. The only specimens that exhibited only slight cracking were the mild steel connectors tested in horizontal reversed-cyclic shear. The crack patterns varied depending on test type and specimen type. For example, during the tensile tests connectors F and G tended to cause a very wide crack with spider-web like cracks propagating outward whereas the connectors A and B tended to cause several cracks on the flange edge and into the top of the flange.

Ultimate Capacity: Monotonic versus Reversed-Cyclic

The ultimate capacities recorded during the reversed-cyclic horizontal shear tests did not tend to be significantly lower than during the monotonic test. This may have been because the first cycle was a displacement cycle to 2.5 mm

(0.1 in) and the second to 5 mm (0.2 in), not providing enough cycles to observe this reduction in strength due to cycling. However, the test protocol used in this study did provide the desired information on post ultimate strength and stiffness reduction due to cycling, as well as provide information on energy dissipation versus observed damage.

OVERALL DAMAGE

Damage to the concrete slabs for the specimens tested in vertical shear was fairly consistent regardless of the connector tested. Less spalling occurred during the vertical shear up test. During the horizontal shear tests significant spalling occurred in the local area of the connector. Connectors F and G tended to cause more spider-web-like cracking at longer distances away from the connector.

Damage (including spalling) during the reversed-cyclic vertical tests was very significant for all specimens. The size and severity of the spalling ranged but occurred consistently. The reversed-cyclic horizontal tests also caused significant damage to many of the concrete slabs. The only specimens that were not significantly damaged were those with mild steel connectors. The mild steel connectors tended to fracture before damage was done to the slab.

SUMMARY AND CONCLUSIONS

The results of seventy (70) monotonic and twenty-three (23) reversed-cycle tests on mechanical in-flange connectors commonly used in precast construction were summarized and some discussion provided with reference to a detailed report (due to space limitations). The primary focus for the monotonic tests was on serviceability issues such as cracking load and whether or not there was significant spalling, as well as ultimate load with corresponding displacement. For the reversed-cyclic tests the focus was the hysteretic behavior, ability to dissipate energy, and the qualitative relationship between energy dissipation and damage to the concrete.

Based on the work presented herein and the research study as a whole, the following conclusions are made:

1). Behavior of the various connectors under monotonic tensile load is approximately the same up to 20 kN load.

2). In tension, the ductility of the stainless steel connector was approximately 1.5 to 2 times greater than that of the A-36 mild steel connectors. Although the connectors were only tested to a displacement of 25mm many were still carrying load and had not fractured. The stainless steel was significantly more ductile in horizontal and vertical shear as well.

3). The slab poured in two 50 mm (2 in) layers several days apart seemed to dissipate the most energy during reversed-cyclic load and performed very well during all tests. The concrete compressive strengths were in the same range as the monolithic pours for each layer used in the two successive pours. However, cost effectiveness of such a procedure was not part of this study and are likely to be prohibitive.

4). As expected, the ability of the connectors to dissipate energy during reversed-cyclic loading was at the expense of damage to the concrete slab. Observation during these tests confirmed the direct relationship between energy dissipation and concrete damage.

5). Out-of-plane vertical loading caused significantly more damage to the concrete than in-plane (horizontal shear) loading during monotonic and particularly reversed-cyclic tests. From a damage standpoint, vertical forces and/or motions should be the focus of additional study, such as shear transfer of a moving vehicle over a joint.

6). Perhaps most significantly, it can be concluded that ultimate strength behavior among the connectors is similar enough to facilitate the development of design guidelines independent of the connector used.

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