PCI/NSF Project

Evaluation Methodology for Precast Concrete Diaphragm Connectors Based on Structural Testing

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Abstract

This report presents an experimental approach for assessing the strength and deformation capacity of embedded connections used in conventional precast concrete panel systems. The approach encompasses both in-plane and out-of-plane loading on the connectors. A summary of the testing fixtures, testing procedures and data processing methods are provided. In addition a framework is developed for categorizing connectors based on their deformation capability. Three in-plane deformation ranges are identified: low deformation element, moderate deformation element and high deformation element. Lastly, a method for computing the design strength based on the test results is discussed.

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1. Introduction

Achieving the expected performance out of connection details is critical for the safety of precast concrete building and bridge systems. This document provides an approach for assessing the strength and deformation capacity of embedded connections used in conventional prefabricated concrete panel systems. In addition a series of performance levels are defined which can be used to categorize the connector based on the measured response. The document is limited to in-plane and out-of-plane demands on connections.

The ACI 318-08 building code requirements for structural concrete provide guidance on connections used to transfer forces between members. As noted in Chapter 16:

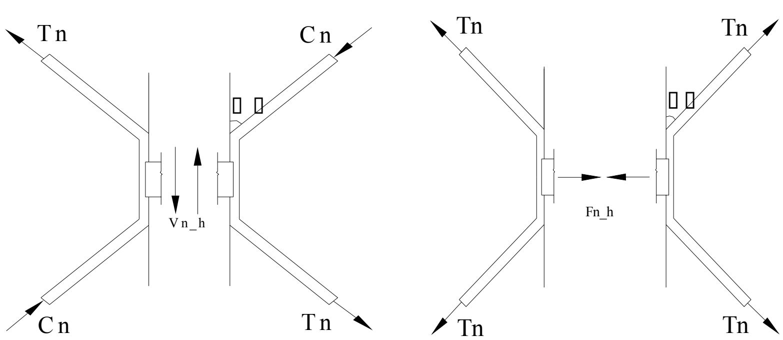
> Section 16.6.1.1 — The adequacy of connections to transfer forces between members shall be determined by analysis or by test.

The methods presented in this document provide a basis for which the adequacy of new and existing connections can be assessed through *testing*. A sample of some of the prevalent *analysis* methods are also described in the following section for completeness.

1.1. Simplified Analytical Approaches

Analytical methods have been developed and are provided in PCI design manuals. The approaches for determining the vertical shear, horizontal shear, and horizontal tension capacity of rebar based connections are commonly used in design of precast connections. This section provides background on the widely accepted analytical approach adopted by PCI.

Current formulation for in-plane strength determination of connection details is based on a general design criteria presented in the PCI design handbook (6th edition) Section 3.8.1.1. The assumption is made that the connection resists in-plane shear and tension through the tension and/or compression of the anchorage legs. The connectors with splayed legs are designed assuming that each anchor leg reaches yield in accordance with Figure 1-1. Based on this assumed mechanism the following analytical relationships are used for determining the nominal horizontal shear capacity, V_{n_h}, and the nominal horizontal tension capacity, F_{n_h}, of the connector:



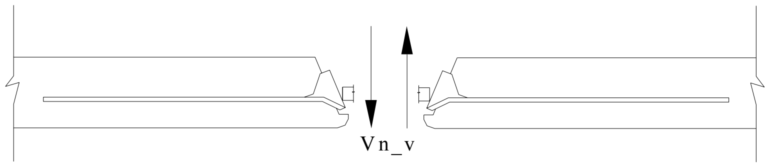
$$C_n = T_n = A_s f_y$$

$$V_{n\_h} = (C_n + T_n)\cos\theta$$

$$F_{n\_h} = 2T_n \sin\theta$$

Figure 1-1. Truss model of double-tee connection

The PCI Standard Connections Handbook (First edition) provides an analytical method for the determination of the nominal vertical shear capacity of the connection. It accounts for two possible failure modes: the first controlled by steel yielding, V_{n_v1}, and the second by concrete shear failure, V_{n_v2}. The analytical method is illustrated in Figure 1-2 and formulated as V_{n_v}.



$$V_{n\_v1} = 2A_s f_y$$

$$V_{n\_v2} = 2.8\sqrt{f_c'}A_{cr}$$

$$V_{n\_v} = Min(V_{n\_v1}, V_{n\_v2})$$

Figure 1-2. Vertical force model of double-tee connection

While the majority of connections are configured similar to the splayed connector discussed, the actual strength is dependent on the details of the connector, embedment, and welding techniques used to provide integrity between connectors. To properly assess the strength of a connection new analysis models can be developed for each connection. To validate these models an experimental verification is necessary. For these applications experimental evaluation criteria presented in this document can be applied to determine the appropriate capacity of the connection.

1.2. Desired Ductile Connection Design

The desired ductile mechanism cannot be formed unless each component of the connection is designed to maintain the load path without premature failure. A typical diaphragm connection consists of anchorage bars, faceplate, slug, and slug weld components. To ensure that ductile modes of failure occur a general rule can be followed. Design the connection to develop a predictable yield mechanism in the anchorage while protecting the other components through over-strength factors against premature failure. For example, designing the weld, slug and faceplate to have strength greater than the capacity of the anchorage will typically provide a ductile connection with a predictable strength. An acceptable hierarchy of strengths is illustrated in Figure 1-3.

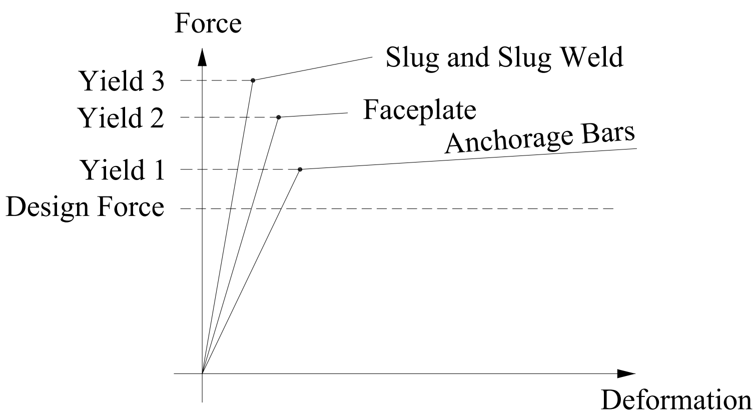


Figure 1-3: Design concept

2. Notation

Only symbols additional to those in ACI 318-08 are defined in this section.

Fv = Shear force resisted by connections [kip]

F_t	=	Axial force resisted by connections [kip]
F_v *Design*	=	Design shear force, it is suggested that the twice of weld shear force should not less than connection resistance force [kip]
F_t *Design*	=	Design axial force, it is suggested that the twice of weld axial force should not less than connection resistance force [kip]
ΔT	=	Tension deformation measured across the connectors [in]
ΔV	=	Shear deformation measured across the connectors [in]
C_n	=	Normal compression force [kip]
T_n	=	Normal tension force [kip]
V_{n_h}	=	Normal horizontal shear force [kip]
F_{n_h}	=	Normal horizontal tension force [kip]
V_{n_v}	=	Normal vertical force [kip]
V_{n_v1}	=	Normal vertical force limited by steel [kip]
V_{n_v2}	=	Normal vertical force limited by concrete [kip]
A_{cr}	=	Concrete Area of assumed failure surface

3. Definitions

Connector	-	One side of connection embedded in the panel.
Connection	-	The entire assembly including the connectors, slug, welds, etc….
Faceplate	-	The face of the connector. Typically the vertical portion of the connector exposed to allow welding to the adjacent connector.
HDE	-	High Deformability Element
LDE	-	Low Deformability Element
MDE	-	Medium Deformability Element
Slug	-	The plate or bar material used to connect one connector to the other. This can be part of the connector or a separate piece installed in the field.
Test Module	-	Laboratory specimen representing characteristics of a typical diaphragm connection for which acceptance is sought. See section 6.0.

4. Scope

The intent of this document is to provide both testing procedures and a framework that establishes the specific performance categories for in-plane and out-of-plane loading of precast concrete diaphragm connectors. Both deformation and force based criteria are developed.

To be consistent with the performance based design methods currently under development by University of Arizona, acceptance criteria are based on prequalification of deformation capacity. Terminology of low deformability element (LDE), medium deformability element (MDE) and high deformability element (HDE) will be used to categorize the connections.

For traditional force based diaphragm design, a procedure for experimentally determining the capacity of connectors is also presented.

5. Testing Agency

It is important that testing be carried out by a recognized independent testing agency. The testing and reporting must be supervised by a professional engineer familiar with the proposed design procedure and experienced in testing and seismic structural design. Possible agencies include but are not limited to University, Government, and Private testing labs with more than 5 years of experience in structural testing of concrete components and/or systems.

6. Test Modules

A minimum of one test module shall be evaluated for each strength characteristic of interest. At a minimum one out-of-plane shear test module, one in-plane shear test module, and one in-plane tension test module shall be evaluated.

Modules shall have a scale large enough to fully represent the complexities and behavior of the real materials and of the load transfer mechanisms in a full-scale system.

Since the test module represents only a small portion of a precast panel, confining effects provided by the whole panel is lost and the panel may be subjected to premature cracking. Additional reinforcement may be used to prevent premature failure of the test module. The additional reinforcement shall not be placed in a way that would alter the performance of the connector. Example reinforcing strategies for 2ft x 4ft and 4ft square panels are illustrated in Figure 6-1

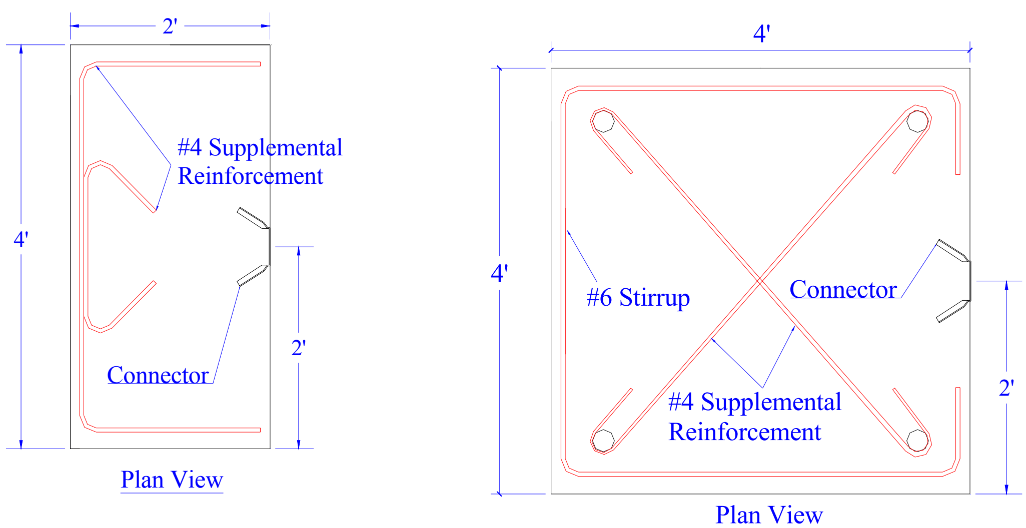


Figure 6-1 Supplemental reinforcement layout and construction details

7. Test Procedures

In this section, the procedures of in-plane tests and out-of-plane tests are presented. In-plane testing procedures are developed using both displacement and force based loading protocols while the out-of-plane test are based on forced based loading protocols.

7.1. In-Plane Test Setup

A multi-directional test fixture shall be used to allow for the simultaneous control of shear, axial, and bending deformations at the panel joint. The fixture shall utilize up to three actuators, two in axial displacement and one in shear displacement (Figure 7-1, Figure 7-2).

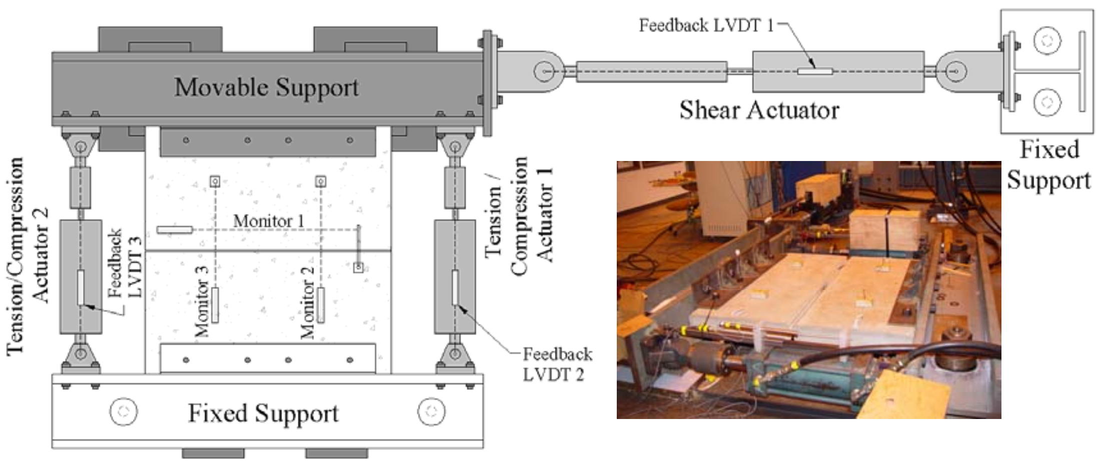


Figure 7-1. Multi-directional test fixture plane view

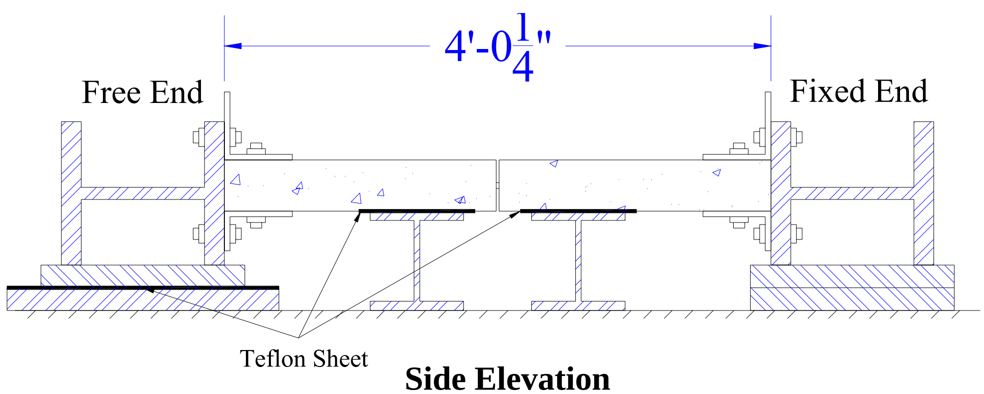


Figure 7-2. Multi-directional test fixture elevation view

Demand shall be applied through independent displacement control of each of the three hydraulic actuators. The test specimen shall be connected to a restraint beam on either end of the panel. One beam shall be fastened to the lab floor, providing a fixed end, while the other beam rests on a pair of low friction (i.e., Teflon coated) steel plates, providing mobility with minimal frictional forces.

Independent control of the three actuators allows for application of shear, axial and bending deformations. Vertical movement of the panel shall be restricted by Teflon coated bearing pads under the center of each panel. This eliminates sag of the test specimen due to self-weight, while still allowing for free, near frictionless travel in the horizontal plane of motion.

Joint deformation shall be measured directly on the precast panel using a series of displacement transducers. Shear deformation shall be determined from measurements taken at the location of the connection. Axial deformation shall be averaged from two transducers on either side of the connection. A possible arrangement of displacement transducers is illustrated in Figure 7-1.

7.2. Reference Deformation

To properly represent typical hysteretic response of seismic demands connections shall be evaluated under cyclically increasing demands. The cyclic demand shall be applied relative to the yield of the connection to ensure that an appropriate number of elastic and inelastic cycles are applied. To accomplish this, a reference deformation relative to the yield of the connector shall be determined.

The reference deformation shall be determined experimentally or analytically.

Experimental determination of the reference deformation shall be based on a monotonic test of the connection test module. The reference deformation represents the effective yield deformation of the connector. It shall be computed by taking the intercept of a horizontal line at the max load and a secant stiffness line at 75% of the max load (Figure 7-3 inset).

Analytical determination of the reference deformation is allowed for connections where the yield deformation can be computed based on well established engineering concepts.

7.3. In-plane Displacement Based Protocols

The panels may be evaluated under in-plane pure shear, pure tension, and combinations of shear with tension. Tests shall be conducted under displacement control at quasi-static rates (< 0.05in/sec) or through force control. Unless noted, all panels shall be tested until the specimen capacity approaches zero.

Under earthquake demands a floor diaphragm system is subjected to a spectrum of relative motions. Five displacement protocols shall be used to assess the performance of connectors subjected to these possible motions. They include:

1. Monotonic In-plane Shear
2. Cyclic In-plane Shear
3. Monotonic In-plane Tension
4. Cyclic In-plane Tension and Compression
5. Monotonic In-plane Shear with Proportional Tension

7.3.1 Monotonic In-plane Shear

The monotonic shear tests shall be conducted to evaluate the connector response under pure shear deformation. The original panel separation of approximately ¼ -in. is maintained through the test by extending the axial actuators in proportion to the applied shear deformation. The test represents the joint condition where the panels are shearing without tensile opening. The test thus provides an estimate of average connector yield, peak strength, and the deformation capacity. Monotonic shear protocol consists of three cycles to 0.01-in. to estimate initial stiffness and verify equipment operation. Afterwards, the specimens shall be loaded monotonically to failure (Figure 7-3).

7.3.2 Cyclic In-plane Shear

Cyclic shear tests provide an insight on degradation of shear properties (i.e., stiffness and ultimate strength) under loading reversals. The loading protocol is in accordance with PRESSS program recommendations [Priestley 1992]. Three preliminary cycles to 0.01-in. shall be conducted to evaluate control and acquisition accuracy. The remaining protocol consists of groups of three symmetric shear cycles at increasing deformation levels. Each level is based on a percentage of a reference deformation computed from the preceding monotonic test.

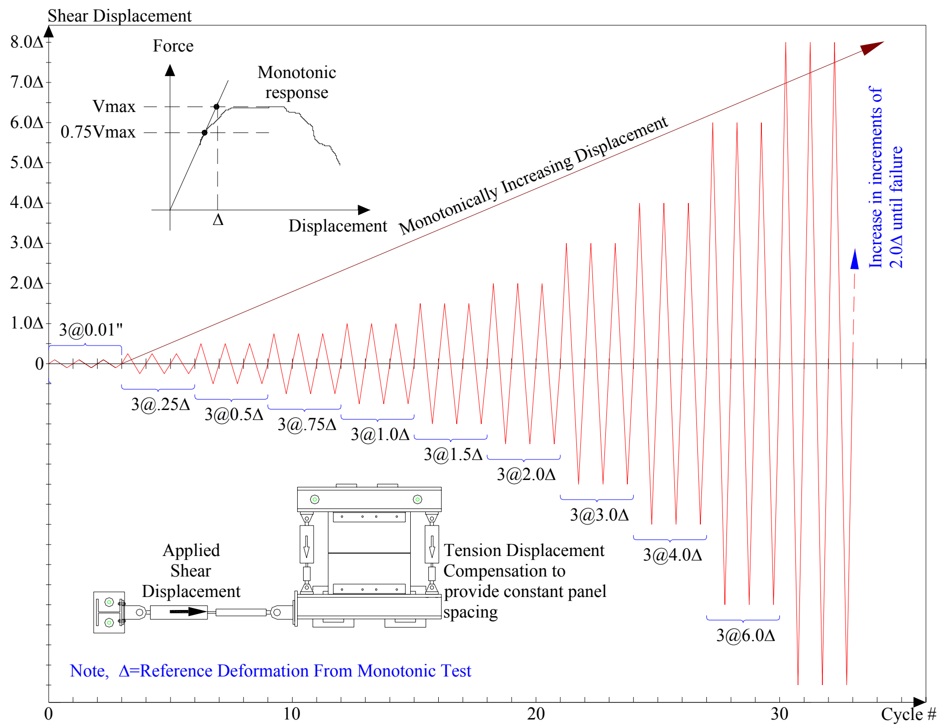


Figure 7-3. Shear loading protocol

7.3.3 Monotonic In-plane Tension

In current diaphragm design, the flexural diaphragm tensile forces are assumed to be resisted by the chord reinforcements. The contribution of shear connectors to flexural resistance is commonly neglected. Previous research has shown that in many cases web connectors provide high tension stiffness. To quantify the relative tensile contribution of the web connectors and chord connectors, monotonic tension tests shall be conducted. The loading protocol consists of three tension/compression deformations to 0.01-in. followed by application of an increasing tension deformation till failure (Figure 7-4). The test shall be paused at each 0.1-in. to allow for observations.

7.3.4 Cyclic In-plane Tension / Compression

Previous research indicates that connector compression stiffness can be in excess of ten times the tension stiffness [Pincheria 1998]. In order to make a comprehensive evaluation of the difference between tension and compression behavior of chord connectors, cyclic tension/compression loading shall be applied. The cyclic protocol consists of three cycles of tension and compression displacement at increasing levels of tension displacement. Each compression half cycle consists of a displacement to 0.01-in. Four elastic displacement levels are applied. The inelastic levels increase at a rate in accordance with a protocol developed for the PRESSS program [Priestley 1992]. The loading protocol is illustrated in Figure 7-4.

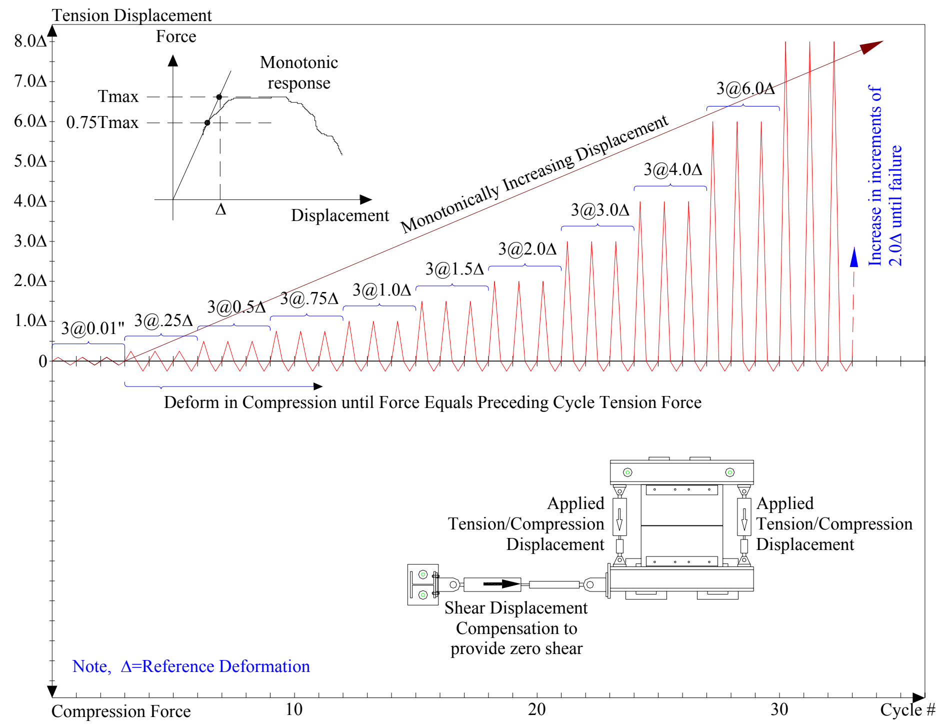


Figure 7-4. Tension/Compression protocol

7.3.5 Monotonic In-plane Shear with Proportional Tension

A combination of shear and tensile deformation ratio of 2.0 and 0.5 shall be used for web and chord connections, respectively. The monotonic shear with tension test consists of three cycles of 0.01 inch in shear and a proportional tension/compression deformation (Figure 7-5). The shear and tension deformations are increased proportionally using the chosen constant shear-to-tension deformation ratio. The test shall be paused at each 0.1 inch of shear deformation for observations.

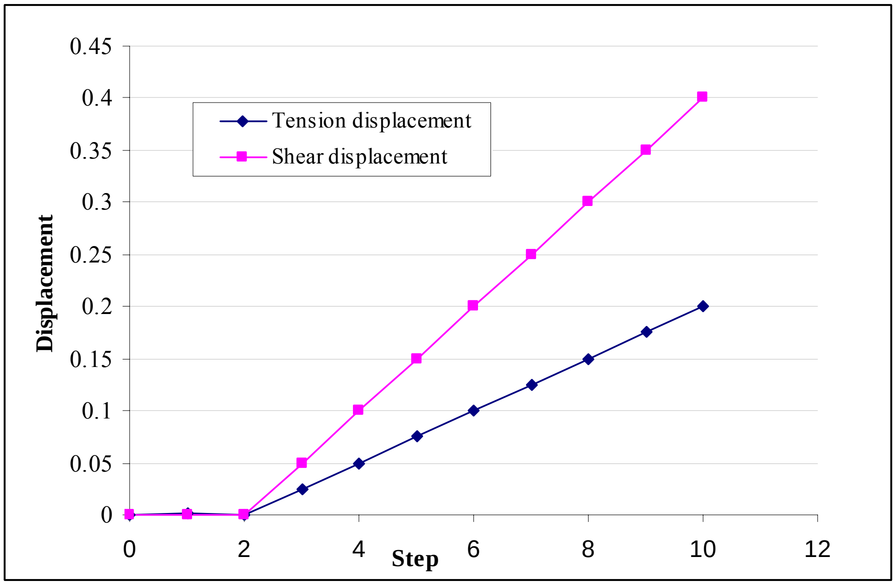


Figure 7-5. Monotonic shear with tension (@ DV/DT=2)

7.4. In-Plane Enhanced Displacement Based Control Loading Protocol

Two enhanced displacement based control protocols shall be used to evaluate the connections under in-plane shear. These protocols are developed to examine the shear performance of connections under varying amounts of axial force. These test protocols provide information that can be used to model the shear resistance of connections at various locations in the floor diaphragm. This includes regions of high compression, tension or areas where zero axial load is present.

All tests shall be conducted at quasi-static rates in a mixed displacement and force control. The control shall be achieved using an inner an outer control loop. The outer loop conforms to the deformation based shear protocols discussed in Figure 7-3. Each displacement step shall be divided into small sub-steps of approximately 0.001in. Each sub-step shall be applied in the inner loop. The inner loop is controlled in a mixed load and displacement manner. After the application of each inner loop shear sub-step, the force in the axial actuators shall be measured. If the sum of the forces is greater than the target axial load, the actuators shall be extended an equal amount until the axial force equals the target. If the force is less, the actuators shall be retracted, and if they are equal to the target no additional axial steps are necessary. An error tolerance of 500lb to 1000 lbs shall be used for acceptance. Following this procedure the next sub-step shall be applied and the axial inner loop shall be repeated. This shall be continued until the full outer shear step is applied. The next shear step would be applied and the process would be repeated.

Two load control protocols are used. They include:

1) Monotonic and Cyclic Shear Deformation with a Target Axial Load of 0 kips;
2) Cyclic Shear Deformation with a Target Axial Load of 10 kips;

For example, the procedure of applying shear deformation with the 0 axial loads is as follows:

1) Apply Shear Deformation Step to Shear Actuator;
2) Read Force in Tension Actuators 1 and 2, F1 & F2;
3) Compute Total Force, Ft = F1 + F2;
   IF Ft > 0, Extend actuator 1 and 2 until Ft = 0
   IF Ft < 0, Retract actuator 1 and 2 until Ft = 0
4) Go to Step 1 Until Target Shear Displacement Reached.

7.5. Out-of -plane Test Setup

The out-of-plane tests shall be performed to quantify the behavior of the conenctor when the panels are subjected to out-of-plane loads. A self-reacting frame is used as shown in Figure 7-6 and Figure 7-7. The test frame is fabricated from wide flange and channel sections.

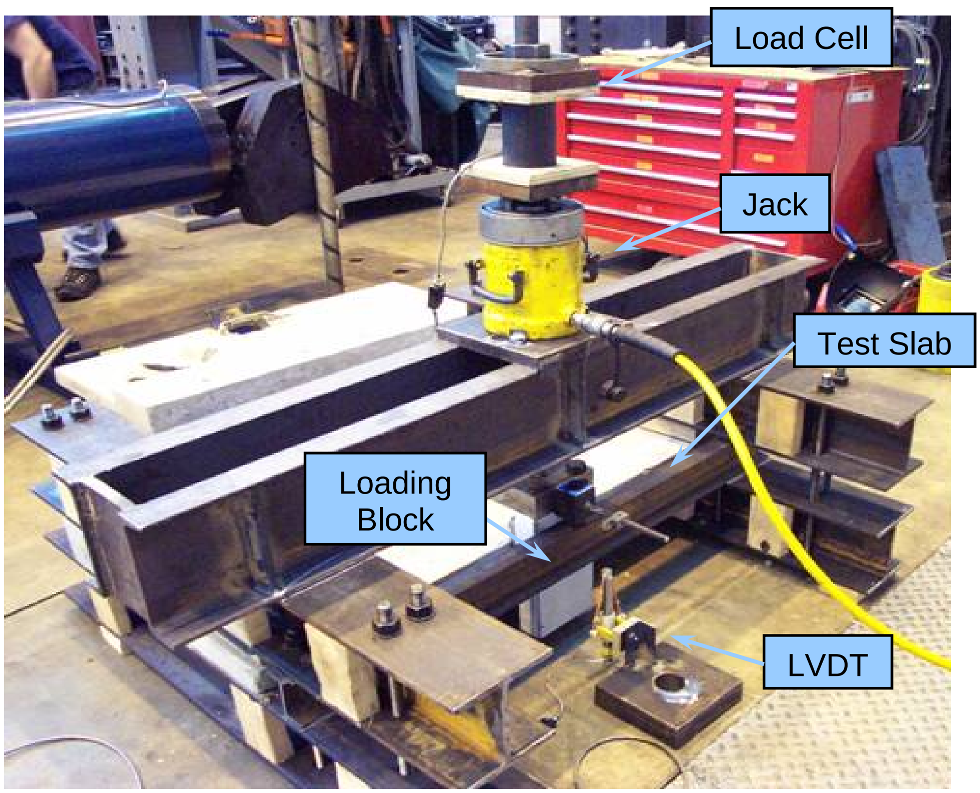


Figure 7-6 Out-of-plane test setup

Figure 7-7. Side view of wood blocks and shims securing reduced thickness region

7.6. Out-of-plane Loading Protocol

The out-of-plane monotonic shear tests shall be conducted to evaluate the connector response under vertical pure shear deformation. The test represents the joint condition where the panels are shearing vertically. The test thus provides an estimate of average connector yield, peak strength, and the deformation capacity. Monotonic shear protocol consists of three cycles to 0.01-in. to estimate initial stiffness and verify equipment operation. Afterwards, the specimens should be loaded monotonically to failure (Figure 7-3).

Load shall be applied through a loading block to the slab. The loading block shall be pulled up via a threaded rod passing through a hydraulic jack and load cell. The loading mechanism sits on the channels that span across the beam sections. Rotation of the loading block as it is pulled shall be restrained by a channel on the opposite side of the block as the slab. Friction between the loading block and channel shall be minimized by a Teflon sheet affixed to the loading block. A hydraulic jack shall be used to slowly increase the pressure and the resulting uplift force to the slab plate. The vertical displacement at the slug connection shall be measured using an LVDT. The load and displacement shall be recorded continuously.

7.7. Testing Observations and Acquisition of Data

Data shall be recorded from the test such that a quantitative, as opposed to qualitative, interpretation can be made of the performance of the test module. A continuous record shall be made of the force versus deformation. For in-plane tests the axial and shear force and deformations should be recorded. For out-of-plane test the vertical force and deformation should be recorded. For static testing data should be recorded at a minimum rate of 1.0 cycle/second.

Photographs shall be taken to illustrate the condition of the test module at the initiation and completion of testing as well as points through the testing history. Ideally photos should be taken at the end of each group of cycles. Photos taken at points of interest, such as cracking, yield, ultimate load and post-test, are adequate for most evaluations.

8. Test Report

The test report must be sufficiently complete and self-contained for a qualified expert to be satisfied that the tests have been designed and carried out in accordance with these criteria, and the results satisfy the intent of these provisions.

The test report shall contain sufficient evidence for an independent evaluation of the performance of the test module. As a minimum, all of the following information shall be provided:

8.1. A description of the theory used to predict test module strength and deformation.

8.2. Details of test module design and construction, including engineering drawings.

8.3. Specified materials properties used for design, and actual material properties obtained by testing.

8.4. Description of test setup, including diagrams and photographs.

8.5. Description of instrumentation, location, and purpose.

8.6. Description and graphical presentation of applied loading protocol.

8.7. Material properties of the concrete measured in accordance with ASTM C39. The average of a minimum of three tests shall be used. The compression tests shall be conducted within 7 days of the connection tests or shall be interpolated from compression tests conducted before and after the connection test series.

8.8. Material properties of the connector, slug, and weld metal based on material testing or mill certification. At a minimum the yield and tensile stress and the ultimate strain shall be reported.

8.9. Description of observed performance, including photographic documentation, of test module condition at key loading cycles.

8.10. Graphical presentation of force versus deformation response.

8.11. A detailed and simplified backbone of the load-deformation response.

8.12. Yield and peak strength and deformation capacity and connection category in shear and tension.

8.13. Test data, report data, name of testing agency, report author(s), supervising professional engineer, and test sponsor.

9. Backbone Approximation

The experimentally measured performance shall be categorized in accordance with the procedures outlined in ASCE/SEI 41-06 *Seismic Rehabilitation of Existing Buildings*. Each connection shall be classified as deformation-controlled (ductile) or force-controlled (non-ductile). This assessment shall be determined based on the backbone curve of the response.

For all the experimental data, a smooth "backbone" curve shall be drawn through each point of peak displacement during the first cycle of each increment of loading (or deformation) as indicated in ASCE/SEI41-06. This method provides a higher estimate of load than the previously used method outlined in FEMA356, in which the "backbone" curve is defined by drawing through the intersection of the first cycle curve for all the (i)th deformation step with the second cycle curve of (i-1)th deformation step. The difference between the two methods is illustrated in Figure 9-1.

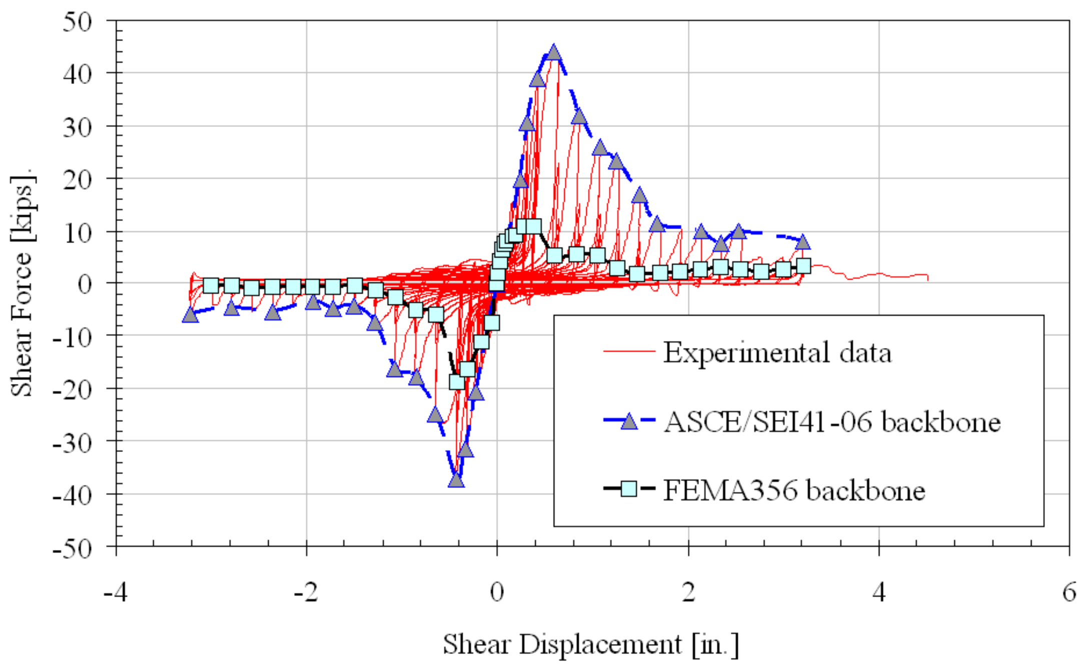


Figure 9-1. Backbone curve

The backbone shall be approximated by a series of linear segments drawn to form a multi-segmented curve. The curve shall be simplified to conform to one of the types indicated in Figure 9-2

As depicted in Figure 9-2, the type 1 and type 2 curve are representative of ductile behavior where there is an elastic range (point 0 to point 1) followed by a plastic range (point 1 to point 3 on the curve). The type 3 curve is representative of a brittle or non-ductile behavior where there is an elastic range (point 0 to point 1) followed by loss of strength.

Deformation controlled elements shall conform to Type 1 or Type 2 response with $e > 2g$. All other responses shall be classified as force-controlled.

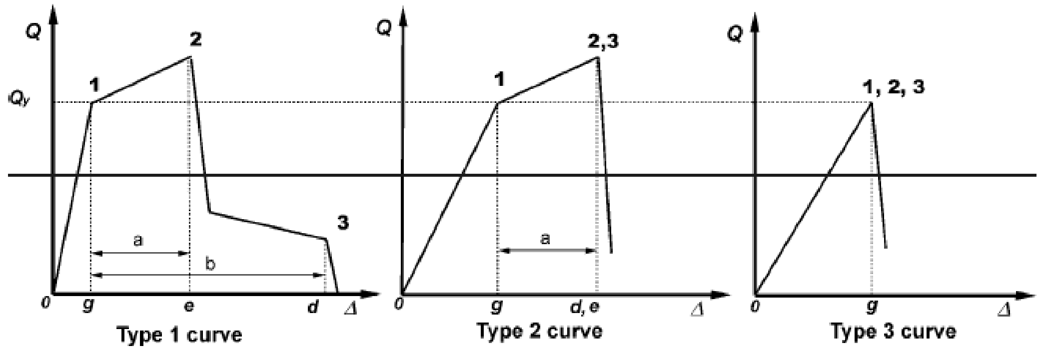


Figure 9-2. Component Force versus Deformation Curves [ASCE/SEI41-06]

10. Test Results

The deformation capacity and the load carrying capacity of the tested connectors shall be determined in accordance with the procedures outlined in this section.

10.1. Deformation Capacity

The yield and peak values shall be determined for each test. The strength and deformation at each level shall be determined from the simplified backbone curves developed in accordance with Section 9. The yield shall correspond to point 1 as indicated in Figure 9-2. The peak load and deformation shall correspond to point 2 as indicated in Figure 9-2.

If the connector is deformation-controlled (i.e., e > 2g), then the mean deformation and force values shall be used. If the connector is force-controlled then the yield and peak values shall be based on the mean value minus one standard deviation.

The connectors shall be classified as a Low Deformability Element (LDE), a Moderate Deformability Element (MDE), or a High Deformability Element (HDE) based on their deformation capacity in tension and shear. The peak deformation (measured at point 2) shall be used to classify the deformability category of the connector. The categorization is based on the critical values indicated in Table 10-1. The category ranges were determined from finite element analysis of a database of diaphragm systems under a range of earthquake demands.

Table 10-1 Deformation category range		
Deformability Category	Tension deformation, ΔT [in]	Shear deformation, ΔV [in]
LDE	$0.00 < \Delta T \leq 0.15$	$0.00 < \Delta V \leq 0.30$
MDE	$0.15 < \Delta T \leq 0.50$	$0.30 < \Delta V \leq 0.70$
HDE	$\Delta T > 0.50$	$\Delta V > 0.70$

10.2. Force Capacity

To provide the design force for the typical connector used in the precast concrete diaphragm system, these methods can be followed:

10.3.1- Three tests of each type are required with none of the results varying more than 10 percent from the average of the three, unless the lowest test value is used.

10.3.2- The average result based on a minimum of six tests may be used regardless of the variations.

10.3.3- The results of two tests may be used when the higher value does not exceed the lower value by more than 5 percent and the lower value is used with the required factors of safety.

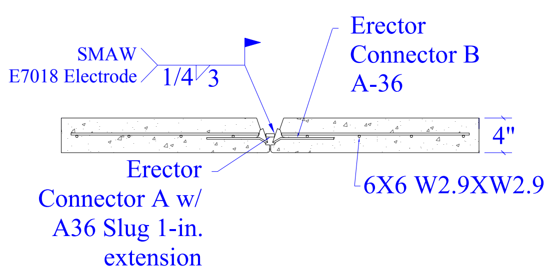
Note: Where tests are not conducted to failure, the highest load achieved for each test shall be assumed as ultimate.

11. Sample Report

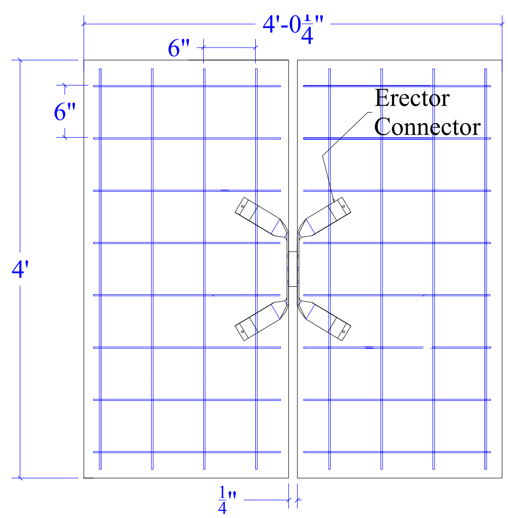
11.1. Test Subassembly Details

A series of example tests are presented in the section. This work is funded by Meadow Burke Co. and is conducted at the ATLSS Center at Lehigh University.

The subassembly is developed assuming that the connectors are spaced at 4 feet and embedded in a double tee panel with a 2ft distance from the DT web to the free flange face. The test specimens are fabricated from two panels 2ft wide and 4ft long (Figure 11-1). The panels are connected to form a 4ft square subassembly. Welded wire reinforcement (WWR) is included in each panel to meet ACI temperature and shrinkage reinforcement requirements. In addition to the WWR conventional reinforcement is used to maintain integrity during testing. The bars are placed at the periphery of the panel to minimize influence on the connector response. The supplemental reinforcement is illustrated in Figure 11-2.



SIDE ELEVATION



TOP PLAN

Figure 11-1. Specimen details – 4” uniform thickness

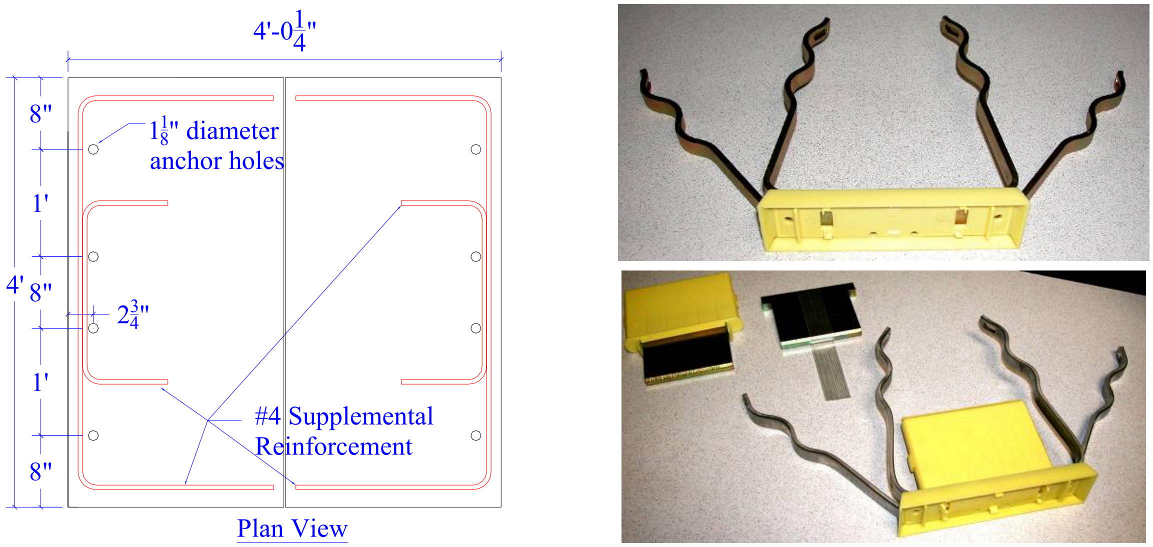


Figure 11-2. Supplemental reinforcement layout and construction details

11.2. Test Results

The in-plane shear behavior of the Meadow Burke connector is investigated in this example test series. In this series, four cyclic shear tests (as indicated in Figure 7-3) were performed (tests CV_AA-1 through CV_AA-4). The behavior of the four specimens is similar. Figure 11-3 shows the spalling failure specimen CV_AA-1 at a load of 9.2 kips (displacement of 2.3 inches). This type of failure is typical of all specimens. Figure 11-4 through Figure 11-7 contain the load versus displacement relationship for the four specimens. Finally Table 11-1 contains the peak load and displacements for all specimens.

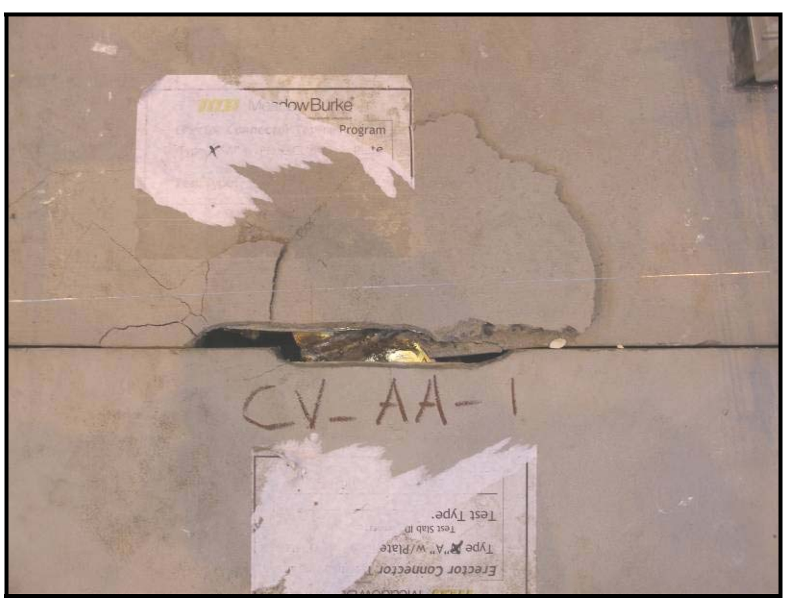


Figure 11-3. Specimen CV_AA-1 - spalling of top surface of panel at a load of 9.2 kip

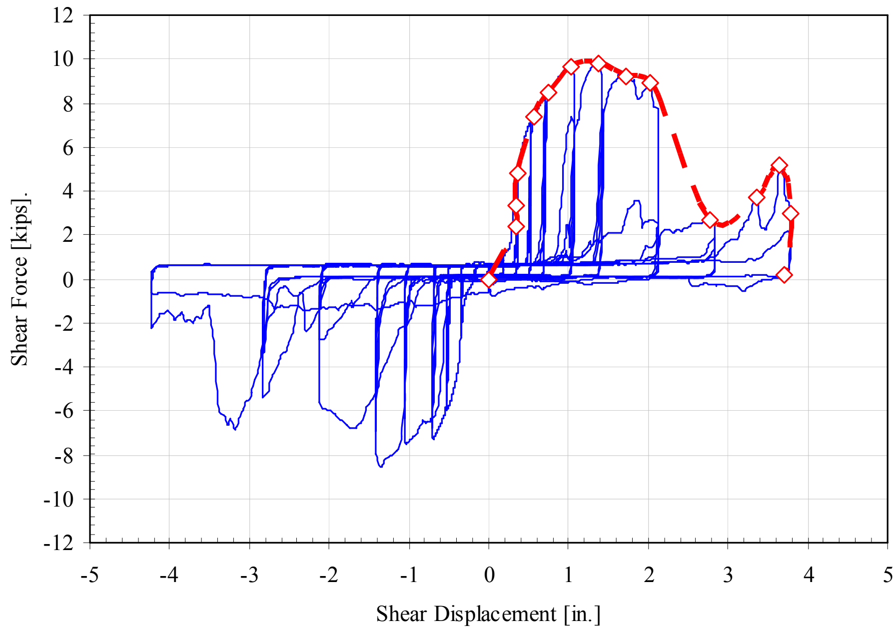


Figure 11-4. Cyclic shear test CV_AA-1 - shear load versus shear displacement

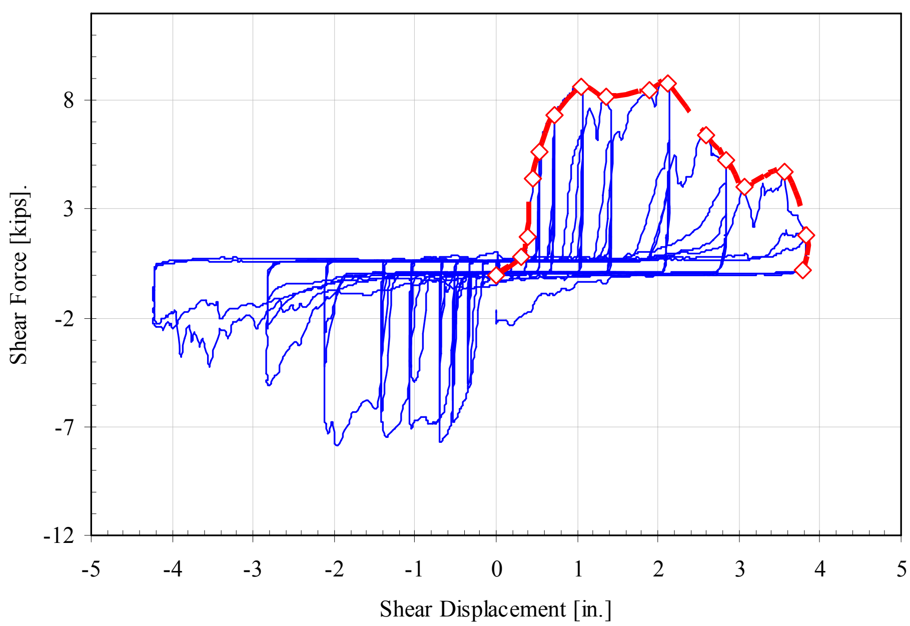


Figure 11-5. Cyclic shear test CV_AA-2 - shear load versus shear displacement

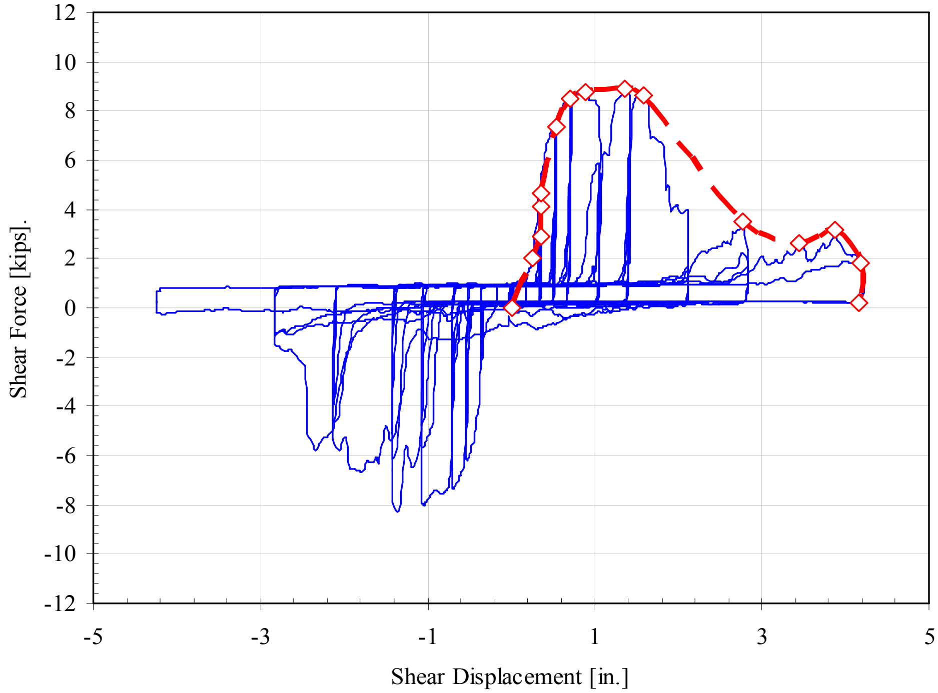


Figure 11-6. Cyclic shear test CV_AA-3 - shear load versus shear displacement

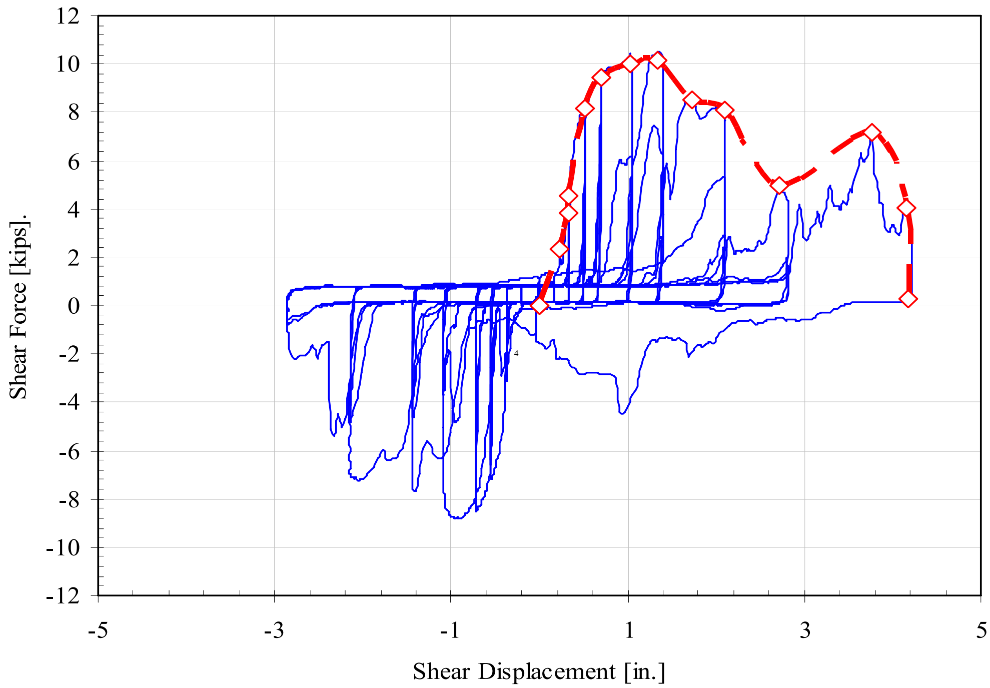


Figure 11-7. Cyclic shear test CV_AA-4 - shear load versus shear displacement

11.3. Test Result Analysis

Follow the definition of the backbone curve in ASCE/SEI41-06, all the backbone curves of this series tests are indicated in Figure 11-8.

As indicated in Figure 11-8, the backbone curves derived for the four experimental tests are compared and an average multi-linear representation of the subassembly behavior is indicated in Figure 11-9.

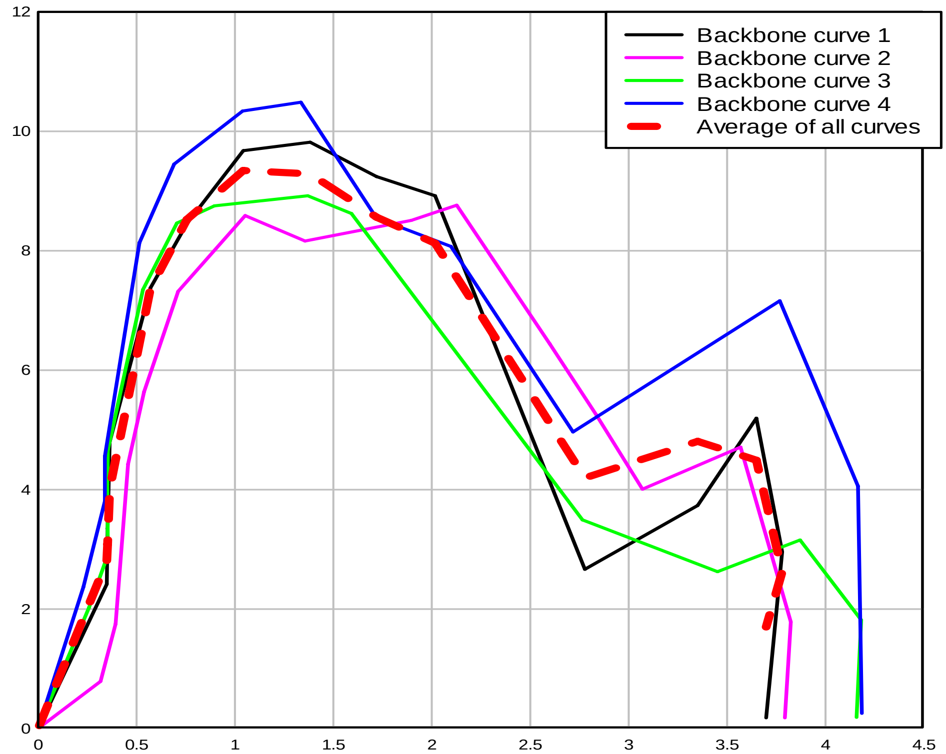


Figure 11-8. Backbone Curves for all experimental tests and average backbone curve

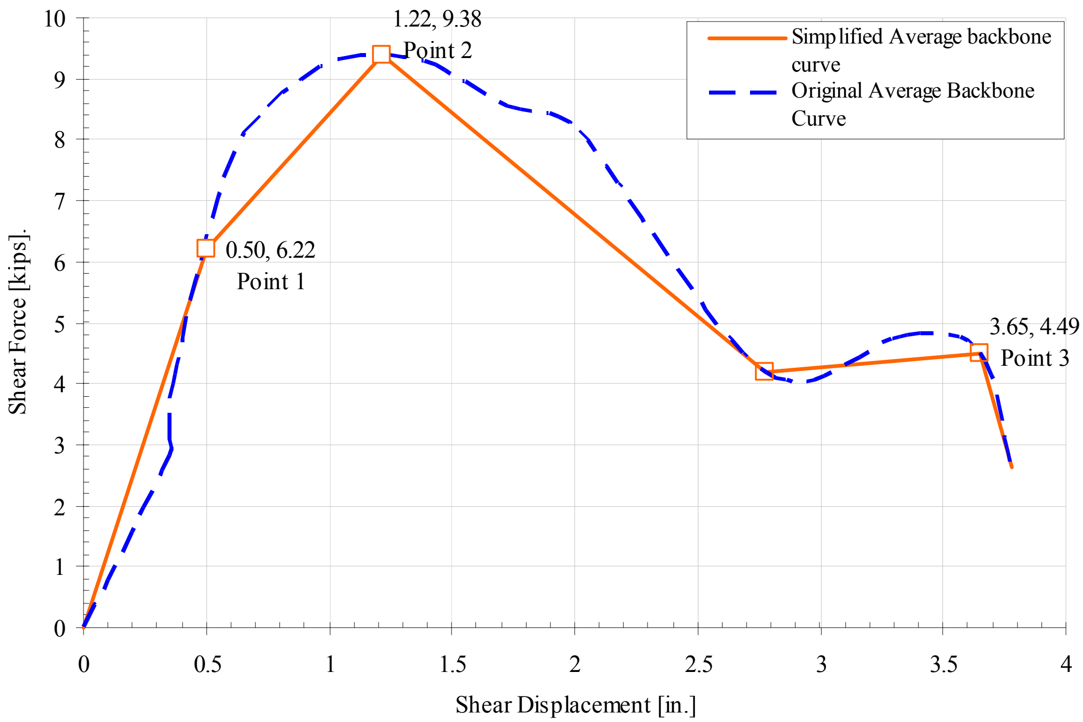


Figure 11-9. Simplified multi-linear average backbone curve for the example test series

The multi-linear curve depicted in Figure 11-9 complies with a type 1 curve as illustrated in Figure 9-2, so the corresponding critical point 1, 2 and 3 are as indicated in the Figure 11-9, it is the deformation-controlled action since $1.22(e) > 2*0.5(g)$, the stiffness is the slope of the first segment on the curve, the yield strength is around 6.22 kip; the peak load is around 9.38kip, and the deformation corresponding to the peak load is around 1.22in. According to the definition of deformability category defined in section 10.2, the example connector under cyclic shear load belongs to High deformability element (HDE).

Also, as indicated in Table 11-1, average peak loads and corresponding deformations for each test are presented.

Table 11-1 Maximum Load and Displacement

Specimen	Max Load (kips)	Δ @ Max Load (in)
CV AA-1	9.8	1.3
CV AA-2	8.9	2.1
CV AA-3	8.9	1.4
CV AA-4	10.1	1.3
Average =	*9.4*	

Follow the procedure of design force defined in section 9.3, since 4 tests are conducted, which satisfied with the minimum requirement of 3 tests, and the deviations for each test are:

$$Sd_1 = \left| \frac{9.8 - 9.4}{9.4} \right| = 4.2\% < 10\%$$

$$Sd_2 = \left| \frac{8.9 - 9.4}{9.4} \right| = 5.3\% < 10\%$$

$$Sd_3 = \left| \frac{8.9 - 9.4}{9.4} \right| = 5.3\% < 10\%$$

$$Sd_4 = \left| \frac{10.1 - 9.4}{9.4} \right| = 7.4\% < 10\%$$

All of them are less than 10 percent, so the item 9.3.1 should be applied to develop the design force using the average value. The calculated design force and deformation category are indicated in Table 11-2.

Table 11-2 Design Force & Deformation Category

Design Load	9.4 kips
Deformation Category	HDE

As depicted in this section, the deformability and design force of typical connector can be evaluated following this acceptance criteria.

12. Reference

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