# Testing of JVI mini-V Flange Connectors for Precast Concrete Double-Tee Systems

Testing Program for JVI

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Contents:



Flange cracking in precast double-Tees is a problem that has been long recognized but little attended to over the years. The behavior may be attributed to poorly designed and poorly detailed connectors as well as poor quality control during installation and welding of adjacent connectors. While flange connector performance has been satisfactory in many applications, it is those instances of poor performance that are remembered by building owners, structural engineers and contractors. Recent seismic events have drawn particular attention to the importance of good connection schemes in preserving the structural integrity of precast concrete building systems. A review of available performance literature on flange connectors quickly made it evident to many engineering professionals that design of connections had been based on assumed behaviors, with little experimental proof of real capacities. This report describes the results of tests to provide a measure of capacities for a double-Tee flange connector (the mini-V) recently developed by JVI. During this pilot study 5 connector tests were conducted under monotonic and cyclic shear loading.

## Introduction:

It is common practice to use discretely spaced mechanical flange-to-flange connectors to join adjacent double-Tee beams together to form floors and roofs. The mechanical connectors may be used alone or a cast-in-place concrete topping may be combined with connectors. Toppings are seldom used without mechanical connectors and testing has shown that normal toppings are too brittle to be used alone for shear resistance in seismic conditions.

In the simplest situation, the purpose of mechanical flange connectors may be to provide for adjustment of differential camber in adjoining beams. Where heavy vertical loads exist, the connectors may be relied upon to distribute loading to adjacent beams through vertical shear in the connectors. Often, however, the connectors are also expected to transfer horizontal shear between beams to develop diaphragm action in floor or roof systems. The majority of existing connection schemes were developed in a trial and error manner, rather than through engineering design followed by experimental testing. Examples of such ad-hoc connectors attempting to resist a variety of loads continue to surface in buildings or parking garages where distress has caused owners to seek solutions to the existing poor behavior.

Damage and partial collapse of buildings, with double-Tee diaphragms that were poorly designed, during the 1994 Northridge California earthquake brought the problems in diaphragm design to the attention of design experts and building code officials. Subsequent analytical studies [1] have shown the potential weaknesses in existing floor diaphragm systems and have emphasized the importance of good diaphragm design.

Since diaphragms are used by designers to brace columns against buckling, to distribute lateral wind loads to resisting members, and to maintain integrity and distribute earthquake induced loads, their performance is crucial to good structural behavior and building serviceability. This combination of functions expected from connectors produces a wide set of qualities and capacities that good connectors should exhibit.

The ideal diaphragm flange-to-flange connector would possess a set of qualities to meet an entire series of demands from the various loadings that will occur.

1. *The connector should be flexible when tension parallel to the floor occurs* across a joint. Volume change in a concrete structure, particularly due to temperature, but also shrinkage and creep effects, will demand that joints move or open during the life of the structure and likely on a regular basis. If a connector cannot flex to accommodate this movement it will try to prevent it. Then it will develop large tension forces that may cause visible damage and deterioration in concrete surrounding the connector. If the connector can flex without causing damage in the concrete flange, large tension forces do not develop and a large tension strength capacity is not required. Joints will have to be treated with appropriate

caulking to accommodate the possible opening movement.

2. *A strong connector is needed to resist vertical shear forces*. The connector must be able to hold adjacent flanges together when a vertical load is applied on one member. Differential vertical displacement of adjacent flanges is unacceptable because it results in ridges or bumps in the floor and floor vibration. The connector should also be able to hold adjacent flanges in place to adjust for differential camber. To hold both flanges together with the same vertical displacements, the connector needs vertical shear strength and stiffness.

3. *A strong, stiff connector is needed to resist horizontal in-plane shear forces*. A floor or roof system can only be considered to be an acceptable diaphragm if the connections between double-Tees can transfer the required wind or earthquake induced shear forces. In addition to strength, stiffness is important or the diaphragm will be too flexible and assumed load distributions to resisting members will not occur or peripheral damage may develop in other components of the building.

4. *The connector must be able to perform well under a simultaneous combination of the loads* described above. All of the possible connector loads may exist simultaneously in a building structure.

5. *The connector should have strength and stiffness capacities that can be reliably predicted* and used in design. If the capacities cannot be reliably predicted then excessively large safety (or capacity reduction) factors need to be used in designing the structure's connection capacity.

It is very important to understand that the "connector" in the above context refers to the concrete surrounding the connector embed, the physical embedded connector, and the welded slug between embeds in adjacent flanges. While the connector embed may be well designed for required stiffness and flexibility, if it is not welded properly or positioned correctly in the concrete it will not perform. One of the primary problems in achieving good connector performance is in obtaining slug welds that are correctly positioned on the connector and are of good quality.

## Objective and Scope:

The primary objective of the pilot tests described here was to measure the load-deformation behavior of a new flange-to-flange connector designed by JVI.

Accurate analytical prediction of connector performance in strength and stiffness is extremely difficult to obtain because of the non-homogeneous nature of concrete and local stress concentration effects at connections. True evaluation of connector capabilities can best be achieved through careful experimental testing with proper loadings and measurement of response.

A set of key tests were conducted on two variations of the JVI mini-V connector. The two versions of the connector were both bent metal straps, but consisted of normal carbon steel and type 304 stainless steel.

The tests included measuring performance under:

1. monotonically applied horizontal shear force, and
2. cyclically applied horizontal shear force.

The performance of the connectors were evaluated in each test by digitally measuring the applied load and the resulting deformation within the connector region of the connected system.

## Test Plan:

The mechanical connectors were individually embedded in concrete slabs that simulated conditions in 2” thick T-beam flanges with 2” thick topping. The connector tests were conducted at a special test station. Loads were applied by hydraulic jacks but the jack movement was actually controlled by specifying desired displacements. The deformations during the tests were monitored by a computer controlled system that automatically measured load and deformations digitally at constant time increments during the tests. The resulting data was stored and analyzed after the test was concluded

### Connectors

The connectors (Figure 1) were designed, fabricated, and supplied by JVI. Similar shapes were used for both carbon and stainless steel units. When placed in the 2" thick concrete slabs for testing (with 2” of added topping), the connectors were positioned with the flat welding face exposed at the edge of the flange. The connectors in the test specimens were often slightly mis-aligned, slanted from horizontal by as much as 1/8” as might be typical in production conditions.

*Figure 1:  Two views of the mini-V connector.*

Since the intention in design and usage should be to achieve a ductile response in the connector before failure, the connector should be welded to adjacent flanges with a slug and weld strong enough to develop the connector strength.  Use of small slugs and welds will reduce the reliability of the connection (due to field weld quality) and produce brittle failures since the failure is likely to occur in the field weld. The slug-to-connector weld was made with a 3/16" throat  by 4" long fillet weld using 6012 electrodes with the carbon steel and 308-16 with the stainless steel.  Welds of other types will create different connector performance.  The test results given here are only applicable to rectangular slugs welded to the face plate 3/16” down from the top of the plate. Weld locations at other positions on the face plate will produce significantly different results.

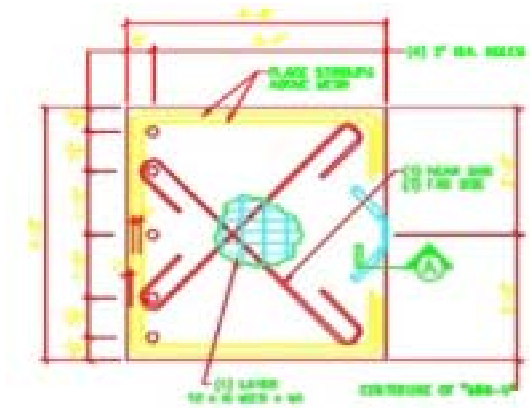
**Concrete slabs**

Each connector was cast at one edge of a 4' x 4' x 2" concrete slab intended to simulate a portion of a double-Tee beam flange.  The slabs were all cast at a commercial precasting plant and the concrete design strength was intended to be 5000 psi at 28 days. The topping was cast separately with a specified strength of 4000 psi at 28 days.  All of the slabs were built and reinforced to the same design specifications.  The slabs were reinforced throughout with 12" x 6" W2.5 x W4 welded wire fabric, the 6" spacing being measured along the flange edge and the W4 wire perpendicular to the edge. Since each slab represented a small portion of an actual flange, special reinforcing was required to resist the forces transferred by the connector, but this reinforcing was not placed in the vicinity of the connector itself.  Reinforcing for the slabs is shown in Figure 2. The welded wire fabric alone resisted any cracking that might have occurred above or below the connector.

**Test station**

A custom test station was fabricated in the lab specifically for conducting double-Tee flange connector tests.  An earlier pilot test series [2] used two 9' x 4' concrete slabs joined together by a flange connector.  Based on the results from those tests it was clear that only a single slab/connector was needed; a smaller and more efficient arrangement was designed.

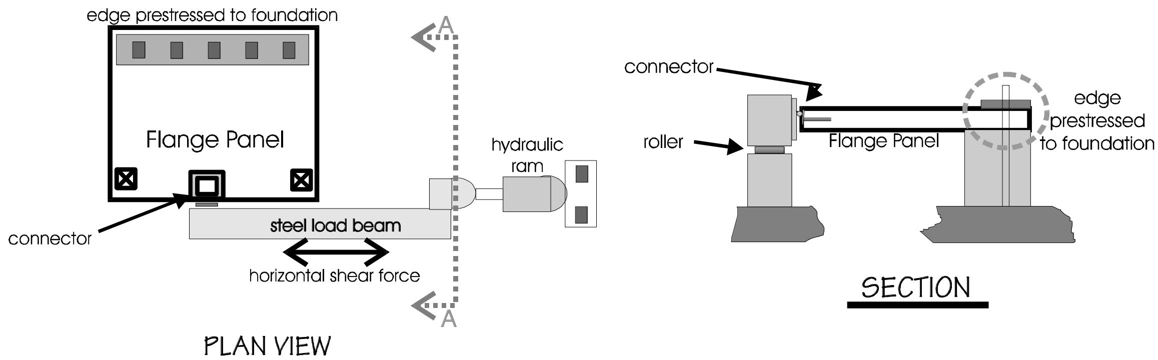
*Figure 2: Sketch showing reinforcing in the test slabs. Nor supplementary reinforcing was placed in the vicinity of the connector embed.*



The slab, with connector, is fixed at the edge opposite to the connector by clamping it down to an anchor block as shown schematically in Figure 3. The remainder of the slab then cantilevers out toward the flange edge simulating the cantilever of the flange from the web in an actual double-Tee.

Loads are applied to the connector through a steel loading beam and only one connector/slab unit is included in a test. The steel loading beam provides horizontal shear loads from a hydraulic jack that is computer operated in either load or displacement control. The horizontal shear loads can be applied in push/pull form allowing cyclical loading of the connector.

*Figure 3. Schematic of test station.*




## Instrumentation

Applied loads were measured digitally through load cells attached between the hydraulic jacks and the loading beam. Deformations in the flange sub-assemblies were measured using linear variable differential transformers (LVDT). Data from load and deformation measurements were automatically collected by a computer controlled data acquisition system and digitally recorded.

Two LVDT's were attached to the concrete slab to either side of the connector and measured the horizontal movement of the load beam relative to the slab. The relative displacement is considered as the deformation that is taking place in the connector and slab joint. Since only one slab/connector sub assembly is included here, the deformation in an actual flange-to-flange joint would be twice the amount measured in the tests and the sub-assembly joint stiffness is the stiffness of the test specimen. The strength in the test specimen and a full flange-to-flange joint would be identical.

**Test schedule**

Five tests were conducted on flange sub-assemblies. The sequence of tests is listed in Table 1. The monotonic horizontal shear test was performed by slowly increasing the amount of displacement applied to the specimen until a failure occurred. A different sequence was employed in the specimens subjected to cyclic horizontal shear loads. Based on a previous monotonic test, the load and displacement at first yield were estimated using a method defined for the Precast Seismic Structural Systems (PRESSS) research program [3]. Load cycles were generally applied in groups of three to displacements levels that were expected to be multiples of the yield displacement level.

**Table1:**

| Connector Type: | Test type | P yield (kips) | Displ. at yield (inches) | P ultimate (kips) | Ult. Displ. (inches) | Stiffness (kips/inch) |
|---|---|---|---|---|---|---|
| Stainless steel MiniV | Monotonic Horz. Shear | 10.5 | 0.042 | 13.9 | >0.65 | 333 |
| | Cyclic Horz. Shear | 10.2 | 0.031 | 10.9 | >0.25 | 337 |
| | Cyclic Horz. Shear | 9.4 | 0.043 | 12.1 | >0.8 | 310 |
| | Cyclic Horz. Shear | 8.5 | 0.020 | 9.5 | 0.48 | 469 |
| Black steel Mini V | Cyclic Horz. Shear | 9.3 | 0.021 | 10.4 | 0.48 | 491 |
| *Note: For a connection between 2 double-Tees the displacements above should be doubled, stiffness halved.* | | | | | | |

## Connector Test Results:

Five connectors were tested, two different materials (plain and stainless steel) were used. One test was conducted on the plain carbon steel connector in a 2" thick topped slab and four tests on stainless steel connectors in topped 2" slabs. The tests used horizontal shear loading. Table 1 summarizes the results from all of the tests.

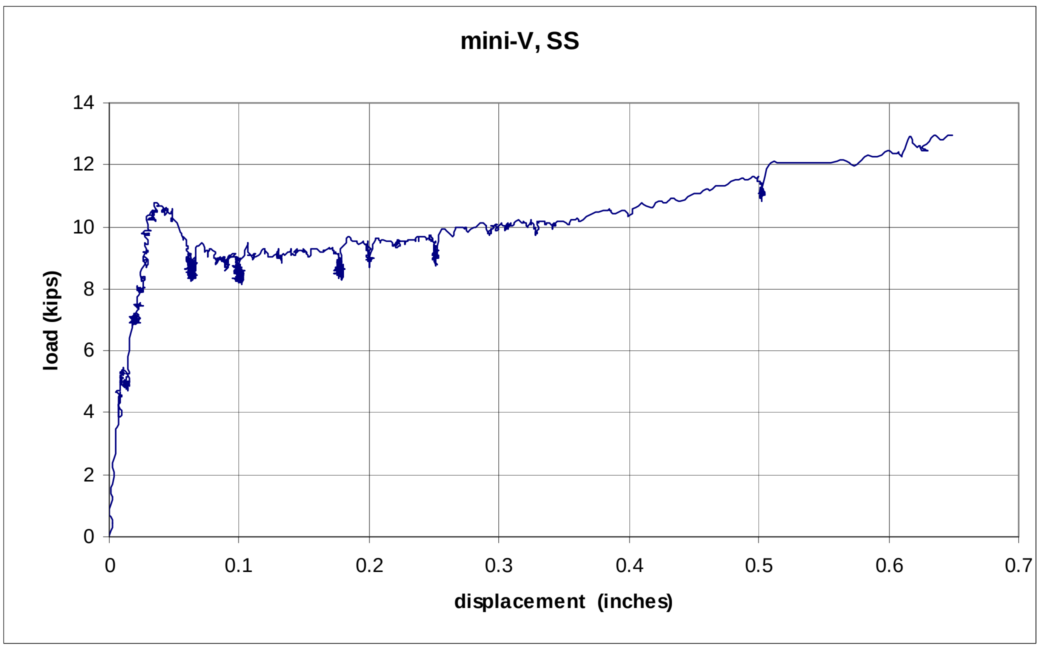
The yield capacity and displacements listed in Table 1 were estimated based on the methods suggested by the PRESSS research program [3] unless a clear yield point was obvious in the experimental data. The initial stiffness was calculated from the resistance capacity and displacement at 75% of the peak capacity.

## Results from Test Series on Pilot Hand Crafted Connectors:

*Test 1 – Stainless steel connector with monotonic horizontal shear:*

A failure did not actually occur in the first test. As the load peaked at 13.9 kips the anchorage of the test slab at its supported end began to slip. The remaining tests were conducted with additional capacity in the anchorage to allow the connector to reach its capacity. The data still provides a valid measure of the performance of the connector in shear since the peak deformation of 1.2” (double the test measurement) is greater than would be encountered in practice. The measured stiffness was 333 kips/inch of deformation. A plot of the load-deformation characteristics is shown in Figure 4.

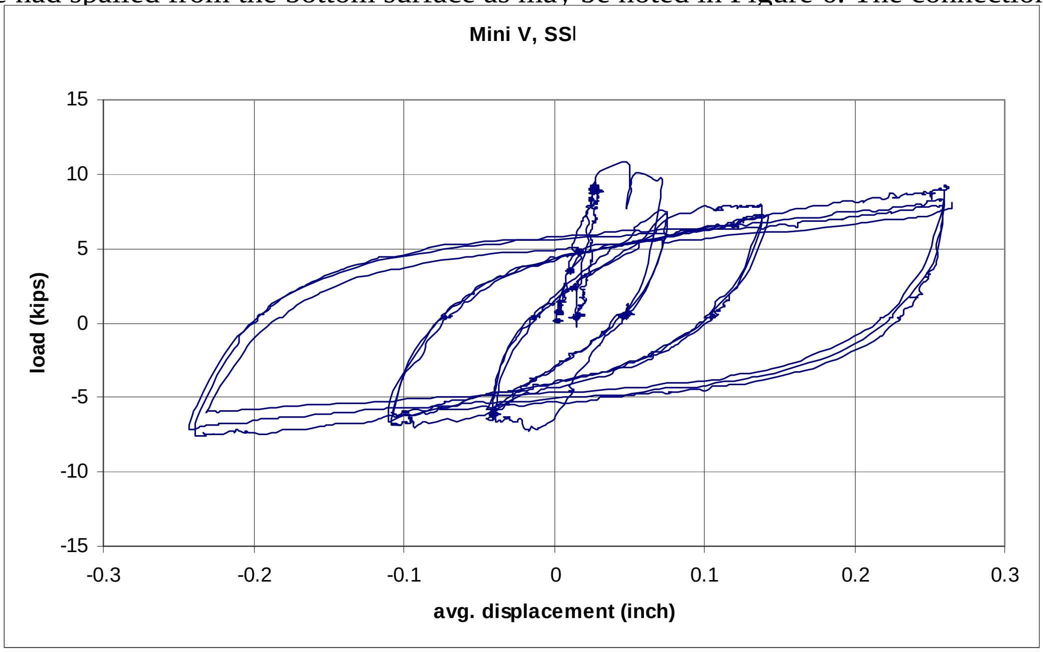
*Figure 4. Plot of applied shear load and connector deformation, stainless connector.*




*Tests 2 thru 4 - Stainless steel connector with cyclic horizontal shear:*

The performance of the first cyclic test (test #2) is shown in Figure 5. The connector was initially loaded to 75% of the expected yield capacity and unloaded. Subsequent cycles of load were applied at various multiples of the expected yield displacement level (which was based on the initial monotonic shear test). The first indication of concrete distress appeared with a small amount (3/4" by 1" x 1/8" deep) of concrete spalling at the edge of the flange directly adjacent to where the connector anchor leg was bent into the concrete. This type of deterioration would not be visible in normal structures and would not affect the integrity of the joint since the spall was 2-3/4" below the top surface of the topped flange. At the end of the cycles of 0.07" displacement there was a permanent gap of nearly 1/8" between the connector face plate and the flange concrete due to initial yielding of the connector at the bend location of the embedded legs. The connector easily reached a displacement capacity of +/- ¼". No damage was visible at the top surface of the flange, but a large area of concrete had spalled from the bottom surface as may be noted in Figure 6. The connection

*Figure 5. Plot of the horizontal shear and displacement for test 2: cyclic stainless.*



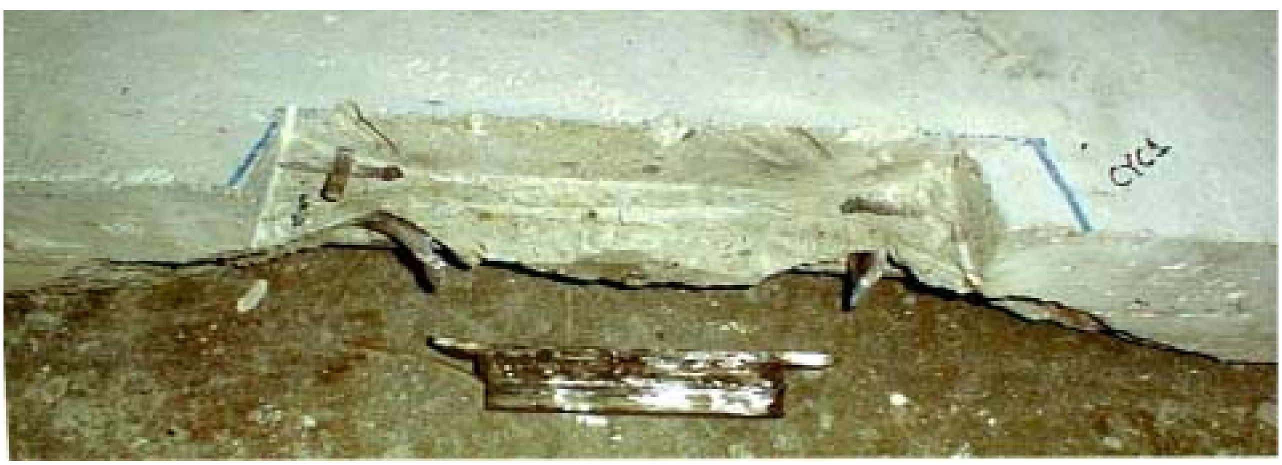
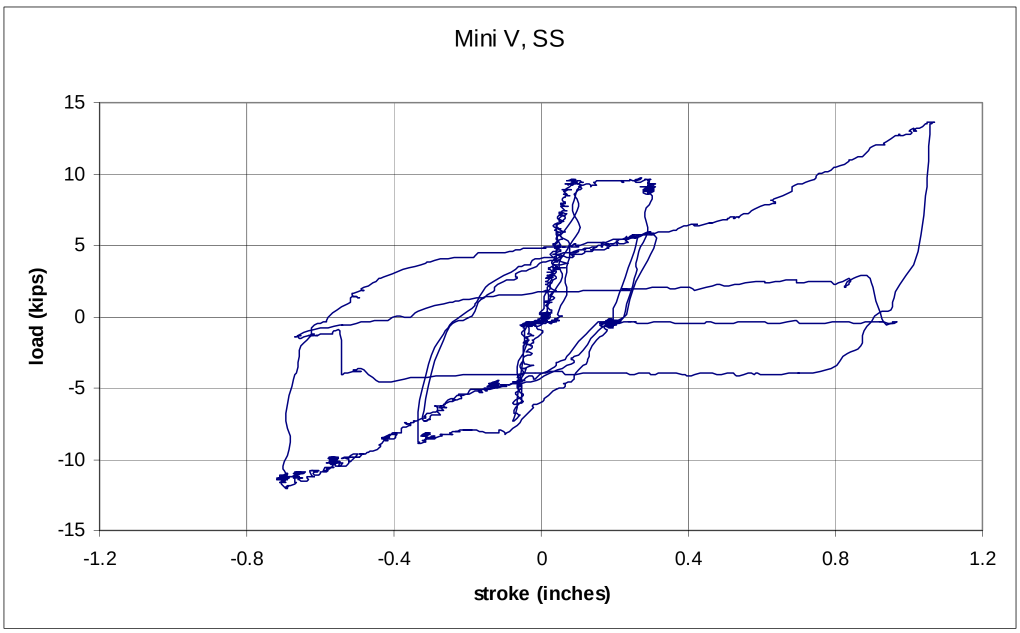



*Figure 6. Condition of concrete near connector is shown after completion of cyclic test #1. Note that the topping concrete above the connector was sawn away to provide welding access to the connector*

failed with the rupture of the connector at the bend where the face plate meets the anchor leg as shown in the photo.

The behavior recorded during the second cyclic test is plotted in Figure 7. The intended loading history was not accomplished during this test due to problems that developed with the LVDT instrumentation. The plot displays the movement of the loading beam and the applied load. After the first cycles of load at +/- 0.05" the LVDT's failed and load was mistakenly applied to +/- 0.25" followed by a final cycle at –0.7" and +1.1" when the connector ruptured. The failure occurred at the bend between the face plate and the embedded leg as shown in Figure 8. The concrete cracking and gap opening behind the face plated developed as in the first cyclic test.

*Figure 7. Plot of the horizontal shear and shear displacement for test 3: cyclic stainless.*



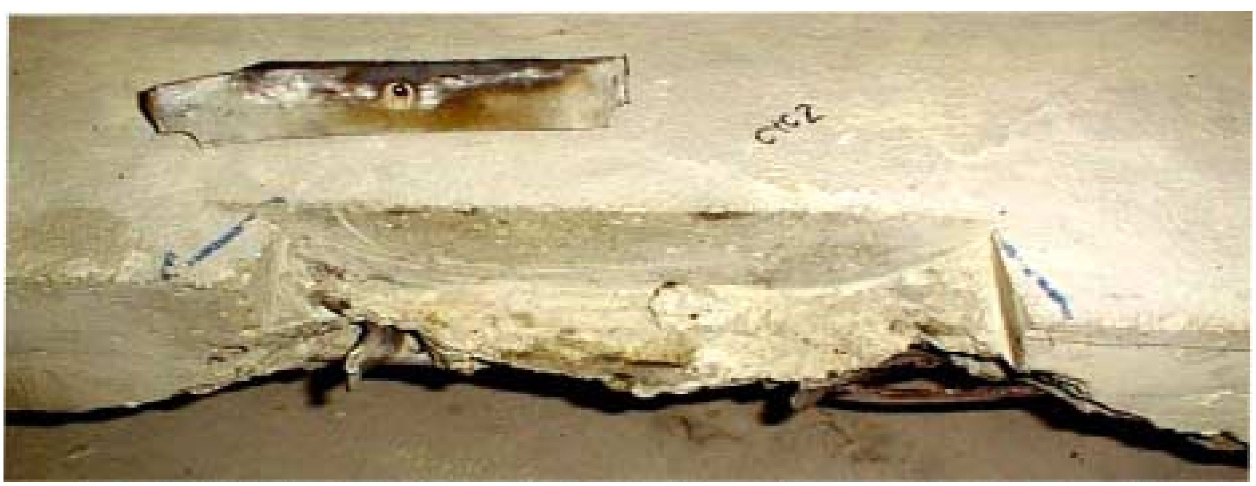
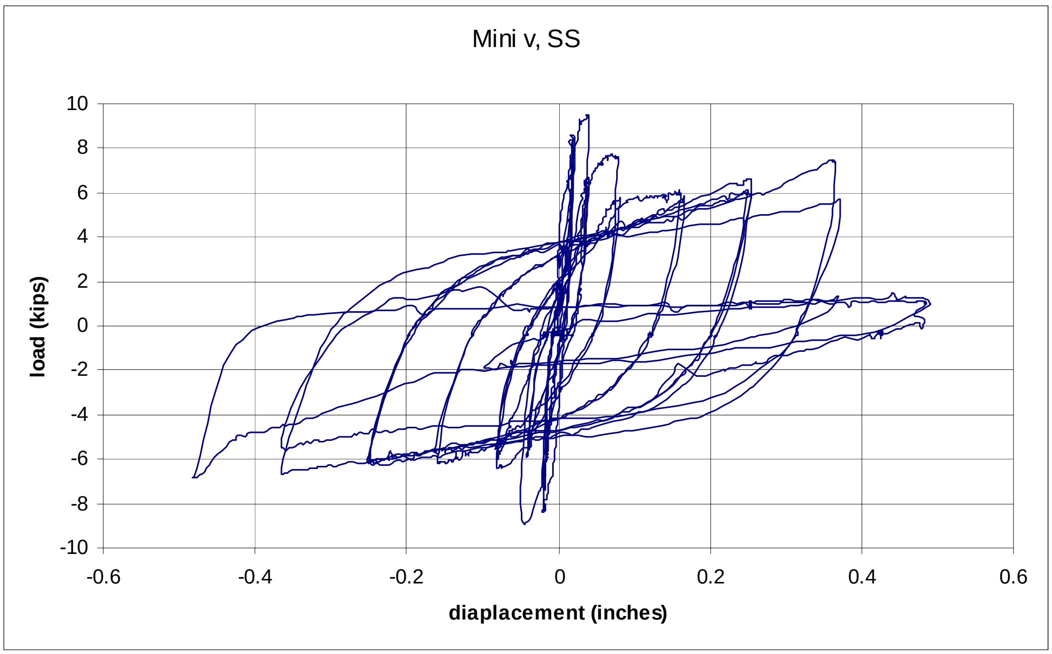




*Figure 8. Condition of concrete surrounding the connector after completion of cyclic test #2.*

Since the second test did not achieve the loading sequence objectives that were desired, a third cyclic test (test #4) was run with horizontal shear. The load and connector displacement results are plotted in Figure 9. This test was successful with cycles applied at 0.02", 0.04", 0.08", 0.16", 0.24", 0.36" and 0.48" displacement levels. The first sign of concrete distress appeared with a 45$^{o}$ crack developing at the edge of the flange from the point where the anchor leg of the connector entered the concrete and extending downward to the bottom surface. The crack was not visible from below the flange. This initial crack appeared at displacement cycles of 0.02" (expected yield displacement). Actual yield displacement was subsequently measured as 0.02" from the test data. During the second cycle of displacement at 0.08" (4 x yield) the concrete above the connector spalled to the level of the bottom of the topping. At 0.16" of displacement spalling occurred below the connector and into the bottom surface of the flange. One anchor leg of the connector ruptured during the second cycle of displacement at 0.36". The ruptured again occurred at the bend adjacent to the face plate.

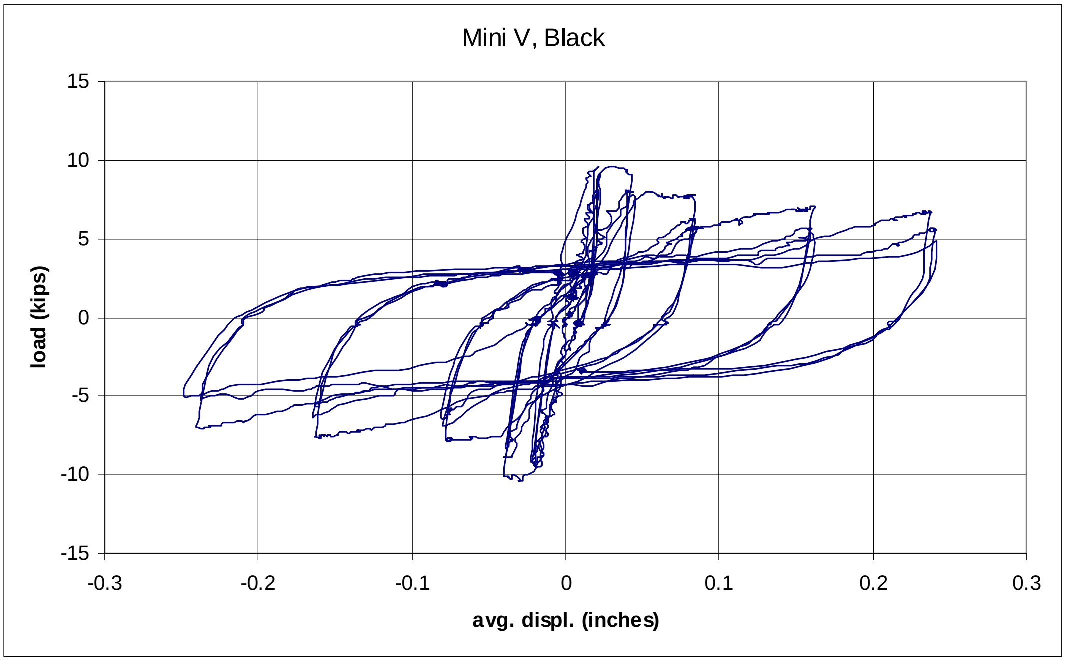
*Figure 9. Plot of the horizontal shear and shear displacement for test 4: cyclic stainless steel.*



*Test 5 - Plain steel connector with cyclic horizontal shear load:*

A test similar to the previous series on stainless steel material was conducted on a flange connected with a black steel connector (test #5). The test results are plotted in Figure 10. Cycles of load were attempted at the same displacement levels as used in test #4 but the connector failed on the first cycle to 0.36”. After the first cycle of displacement at +/- 0.02” a gap of slightly less than 1/16” had permanently developed behind the connector face plate. At 0.04” of displacement small cracks appeared in the concrete at the flange edge just adjacent to where the connector legs entered the concrete. Those cracks led to spalling of the concrete at the flange edge with displacements of 0.08”. The spalling extended to the bottom flange surface by the last cycle at 0.08”. One leg of the connector broke, at the bend adjacent to the face plate, as the displacements were increased from 0.24” to 0.36”.

*Figure 10. Plot of the horizontal shear and shear displacement for test 5: cyclic black steel.*



## Summary:

These pilot tests proved that the new mini-V connector can be successfully used as a lightweight connector for connecting 2” thick double-Tee flanges. The connector was successful in that none of the tests failed in either an anchor pullout or weld rupture. Moreover, the connector exhibits reliable load capacity after yielding and a desirable ductility that should help insure survival of resisting ability after extreme loading. Further testing is necessary to provide full guidelines for design of connections using this connector – including vertical shear capacity, axial flexibility, and behavior under combined loading conditions. While the test specimens were 2” thick flanges with 2” of added topping, the topping was not continuous across the flange joint and the capacities measured were provided by the mechanical connectors alone.

The results described here, particularly regarding the ductility and avoidance of weld failures, are strongly dependent on the placement and type of weld used between the connector and the steel slug. The steel slug was placed 3/16” below the top of the face plate in all of the tests. A 3/16” by 4” fillet weld was used between the slug and the connector to provide over-strength in the weld. Placing the slug at a different elevation will likely have a strong effect on the connector ductility and flexibility. Using a smaller weld may change the failure mechanism from a ductile and reliable failure in the steel strap of the connector to a brittle and unreliable weld failure. Since reliability of flange field welds made in various weather conditions is in general not as high as desired, this type of failure should be avoided.

## References: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3. Priestley, N., 1992, “The US-PRESSS Program Progress Report”, 3rd Meeting of the US-Japan Joint Technical Coordinating Committee on Precast Structural Systems (JTCC-PRESSS, San Diego, CA, Nov. 18-20, 1992