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EXPERIMENTAL OBSERVATIONS ON THE SEISMIC RESPONSE OF BEAM-COLUMN JOINTS IN REINFORCED CONCRETE DOUBLE-DECK BRIDGE STRUCTURES

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ABSTRACT

A research program to study the mechanisms controlling the seismic response of reinforced concrete beam-column joints in ductile double-deck bridge structures is being conducted at the University of California at Berkeley. Two one-third scale beam-column joint test specimens were built according to current design criteria and tested in the laboratory. The two test specimens differed in the capacity of the members framing into the joints. This difference in capacity resulted in a difference in maximum demands on the beam-column joint at the member interfaces. The scope of the tests was to determine the response of the joints when subjected to uni- and bidirectional loading cycles and compare the response to different levels of maximum joint shear stress demand. The test specimens were able to sustain the design forces and deformations without significant distress in the joint. Most of the plastic deformation and damage concentrated at the column ends. Analysis of the response of the two specimens will be used in the development of joint force transfer mechanisms.

KEY WORDS

seismic design, beam-column joints, compression strut mechanism, truss mechanism, joint shear stress

INTRODUCTION

The principal mechanisms that are widely accepted to model joint behavior in the design of building structures are the compression strut mechanism and the truss mechanism. In the compression strut mechanism, the concrete transfers the compression forces across the joint while member longitudinal reinforcement in tension is anchored in this compression force field, as shown in Fig. 1. In the truss mechanism, the tension and compression forces are transferred by the interplay of joint reinforcement and core concrete, as shown in Fig. 2. In joints where yielding occurs at the beam ends and the columns remain elastic, both models require additional horizontal joint reinforcement to confine the concrete or resist joint shear. The distributed column reinforcement, being elastic, is usually relied on to act as vertical joint reinforcement. In bridge construction it is more common to design a structural system where yielding is anticipated in the columns rather than the beams. In this case it is unclear whether the distributed column reinforcement can be relied on to act as vertical joint reinforcement. The objective of the test program described in this paper is to investigate this issue.

Two one-third scale beam-column joints with different levels of joint shear stress demands were built according to current design criteria and tested in the laboratory. The difference between the two specimen lies in the difference in capacity of the members framing into the joint. The nominal strength of the columns and beams of the second specimen are greater than those of the first. This results in a higher joint shear stress demand imposed on the beam-column joint of the second specimen.

The joint design was according to the recommendations of the 1985 report of ACI-ASCE Committee 352 [ACI-ASCE, 85]. These recommendations are given for the design of beam-column joints in building frames, where yielding is expected at the beam ends. Because yielding is expected at the column ends in the test specimens, the applicability of these design recommendations is assessed in the test program. The joint shear stress demand imposed on the beam-column joint of the first test specimen equals the capacity prescribed by the ACI-ASCE Committee 352 recommendations for this type of joint. The demand imposed on the beam-column joint of the second specimen exceeds the prescribed capacity. The tested beam-column joints, reinforced only with a horizontal spiral and no vertical ties, were able to carry the imposed shear demands without significant distress.

The specimen design, instrumentation system, testing set-up, and test results for the first test specimen have been discussed in more detail elsewhere in the literature by Mazzoni [Mazzoni *et al.* 1995].

ACI-ASCE COMMITTEE 352 RECOMMENDATIONS

ACI-ASCE Committee 352 recommendations provide guidelines on the design of member proportions and reinforcement details in the beam-column joints of reinforced concrete moment resisting building frames. The design criteria for the joint specify the amount of transverse reinforcement necessary to confine the joint core so it can resist the design shear stresses. The recommendations published in 1985 hypothesize that joint shear is carried primarily by a diagonal compression strut from one corner of the joint to the opposite corner. Joint transverse reinforcement serves the purpose of confining the diagonal compression strut so that it can retain strength to target inelastic deformation levels.

For joints with anticipated large inelastic deformation demands in adjacent beam components, the nominal joint shear strength, V_n, is expressed as:

$$V_v = \gamma \sqrt{f'_c}\,(psi)\; b_j h \tag{1}$$

where f'_c is the specified compressive strength of the concrete in the joint; b_j is the effective joint width; and h is the thickness of the column in the direction of load being considered. Because it does not have horizontal structural members framing into all four sides, the lower level beam-column joint in a double-deck bridge structure is considered an exterior joint. For this case, γ=15. The committee report recommends that the design should ensure flexural hinging in the beams rather than in the columns, as is typical in the design of moment resisting building frames.

The joint reinforcement prescribed by the committee consists of a horizontal spiral, or horizontal rectangular hoops and cross-ties. In consideration of vertical joint shear stresses, the committee report specifies that column reinforcement shall be distributed uniformly around the joint. It is assumed that the column longitudinal reinforcement, expected to remain elastic, will provide any needed restraint against vertical dilation of concrete in the joint.

DESIGN OF TEST SET-UP

In a typical double-deck bridge structure, the lower deck is rigidly framed into the columns, whereas the upper deck and footing are pin-connected to the columns. During seismic excitation, the longitudinal inertial forces are resisted by framing action between the lower deck and the columns. A longitudinal edge beam stiffens the deck at this level to improve this framing action. Transverse inertial forces are resisted by framing action between the lower deck bent cap and the column.

In the laboratory, the two lateral load resisting systems, longitudinal and transverse, are studied both independently and simultaneously. The model tested in the laboratory is shown in Fig. 3. The lateral loads are applied by actuators attached to the top column. Even though the inertial loads in the prototype are distributed between both levels, the specimen in the laboratory is subjected to lateral loads at the top level only so that both columns are subjected to similar loads. The loading actuators are shown in Fig. 4. The actuators are installed perpendicular to each other and at an angle to the specimen so that bidirectional loads can be imposed from a single reaction frame. Displacement cycles of increasing amplitude are imposed by these actuators.

The dead load of the structure is simulated by an unbonded prestressing rod passing through the column center. The prestressing load is the same for both columns.

DESIGN OF TEST SPECIMEN

The experimental program involves testing of two physical models of a joint of a double-deck bridge structure. The joint configuration is depicted in Fig. 3. The specimens are designated Specimen-1 and Specimen-2.

As described previously, the design approach for bridge structures commonly strives to have flexural plastic hinges in the column, with effectively elastic response in all other components. For Specimen-1, the objective is to achieve a relatively high nominal joint shear stress of 15 $\sqrt{f'_c}$ (psi). The quantity of column longitudinal reinforcement is set so as to achieve this target joint shear stress when the column reaches its plastic moment strength. The columns are designed to have the same strength so that the worst loading conditions are imposed on the joint.

The objective in the design of Specimen-2 was to achieve a higher nominal joint shear stress of 20 $\sqrt{f'_c}$ (psi). The same design procedure as that used for Specimen-1 was used for Specimen-2. The joint configuration and member dimensions were kept the same as those of Specimen-1. The reinforcement details for the two specimens were also kept the same by replacing the #5 bars used for the longitudinal reinforcement of Specimen-1 with #6 bars in Specimen-2. The amount of transverse reinforcement in the joint is identical for the two specimens since it is based on the recommendations of ACI-ASCE Committee 352. The amount of transverse reinforcement was changed only in the transverse cap beam due to the increased shear demand. Reinforcement details are shown in Fig. 5-7.

TEST PROGRAM AND RESULTS

Loading History

The test specimen was subjected to both unidirectional and bidirectional loading. The loading scheme is shown in Fig. 8. The last two unidirectional loading cycles are performed to determine the damaging effects of bidirectional loading on the specimen response. This loading scheme is applied to each specimen at cycles of displacement amplitudes increasing from 0.1 inch to 12 inch.

Observed Behavior -- Specimen-1

The first hairline cracks appeared at the bottom column-joint interface at displacements of 0.1 inch. The number and size of the cracks increased with increasing displacement amplitudes. Typical bending cracks were distributed along the column height near the joint.

At displacement cycles of 3 inch it was visually noticeable that most of the deformation concentrated in the top story. As the displacement amplitude increased, the damage concentrated in the top column plastic hinge where significant increase in distress was evident. Even though the cover concrete had previously spalled over a significant length, no further distress was noticed in the bottom column plastic hinge zone.

At displacement cycles of 5 inches, diagonal cracks began to appear in the beams near the beam-joint interface. As the displacement amplitude increased, the cracks began to fan out toward the mid-depth, as shown in Fig. 9. At a displacement amplitude of 8 inches, during the longitudinal and transverse cycles after the bidirectional loading, the first signs of buckling of the top column longitudinal reinforcement were noticed.

During the first longitudinal cycle at a displacement of 12 inch, the top column longitudinal reinforcement buckled on the compression side. Some bars buckled in the plane of the spiral, some inward, and some outward. In the second longitudinal cycle at this displacement, one of the bars that buckled in the previous cycle, the one at the extreme tension fiber, fractured. Additional bars buckled and fractured during the excursion in the transverse direction at this displacement level. All of the buckled and fractured bars were in the top column plastic hinge zone.

Observed Behavior -- Specimen-2

The first cracks began to appear at the column-joint interface at displacements of 0.25 inch. At a displacement amplitude of 0.5 inch, additional bending cracks were evident along the column height.

At displacement cycles of 3 inches most of the damage began to concentrate in the bottom column plastic hinge zone. At this displacement amplitude shear cracks appeared in both the columns and the beams -- the amount of shear reinforcement in all the members but the transverse cap beam is the same for both Specimen-1 and Specimen-2.

The diagonal cracks that were seen at the vertical beam-joint interfaces of Specimen-1 were also observed at the same locations in Specimen-2 at displacement amplitudes of 3 inches. At a displacement level of 8 inches inclined cracks began to appear on the face of the stub. At higher displacement levels these cracks were evenly distributed on the stub face. Similar cracks were not observed in Specimen-1.

Because of clearance limitations in the test apparatus, it was necessary at this stage of the test to offset the specimen by 2 inches in the east direction and before beginning the 10 inch cycles. During longitudinal loading at this amplitude, a number of bars in the bottom column buckled. The test was terminated before any bars were able to fracture due to the limited capacity of the gravity loading system.

Hysteretic Response -- Specimen-1

The response of the two stories of Specimen-1 to the different loading conditions is depicted in Fig. 10-13. These figures display the relations between shear and drift of each story during unidirectional loading in the transverse and longitudinal directions. As seen from the response curves, nonlinear structural response of both stories resulting from yield of column longitudinal reinforcement started at a story drift of approximately 1.5%. The maximum drifts reached are characteristic of the loading direction and the story under consideration. However, all stories were capable of reaching lateral drifts of 5% without loss of strength.

Comparison of the hysteretic response curves shows that most of the deformation was concentrated in the top story. This concentration of deformation led to a concentration of damage in the top column plastic hinge. Failure of the specimen was due to buckling and fracturing of the column longitudinal reinforcement in this section. This behavior is most evident in the response curves of the transverse loading: 80% of the deformation in the last excursion occurred in the top story. Even though it did not sustain any damage beyond spalling of the cover concrete, the critical section of the bottom column did reach its nominal capacity and was capable of dissipating hysteretic energy, as seen in Fig. 12 & 13.

The specimen was subjected to three unidirectional loading cycles in each direction. The third cycle occurred after the specimen was subjected to two cycles of bidirectional loading. The response curves show that the capacity reached during this third cycle was lower than that reached during the first two cycles. This behavior apparently is a result of the significant damage incurred on the specimen by bidirectional loading.

Hysteretic Response -- Specimen-2

The hysteretic response curves for Specimen-2 are shown if Fig. 14-17. The increase in column capacity due to the increase in bar size from the first specimen to the second is evident in the resulting increase in the maximum lateral load resisted. This increase in lateral load capacity, however, does not result in a significant change in the ultimate displacement capacity. Even though the column longitudinal reinforcement did not fracture at this maximum displacement, imminent failure can be attributed to the significant reduction in strength at the higher displacement levels.

While it is evident in the curves that most of the inelastic deformation of Specimen-1 was concentrated in the top story, the deformation distribution between the two stories of Specimen-2 varies depending on the loading condition. In the longitudinal direction, the deformation is evenly distributed between the two stories, with a slightly larger deformation in the bottom story. The axial load variation in the bottom column, due to overturing effects during transverse loading, determines whether the nominal capacity of this column is higher or lower than that of the top column. When the bottom column is weaker than the top column, most of the deformation concentrates in the bottom story. When the bottom column is stronger than the top one, the deformation concentrates in the top story.

CONCLUSIONS

Test results have shown that a joint with moderate quantity of reinforcement has the capacity to sustain large joint shear stress demands without significant distress. The second test has shown that a beam-column joint in the configuration considered in this case is able to sustain demands greater than the capacity prescribed in the past.

ACKNOWLEDGEMENTS

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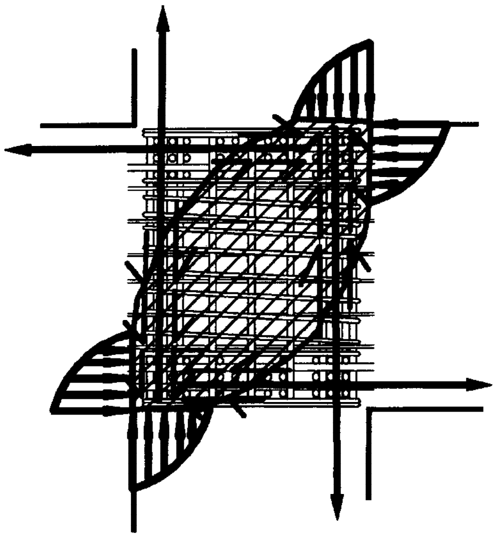

Fig. 1 Compression Strut Mechanism

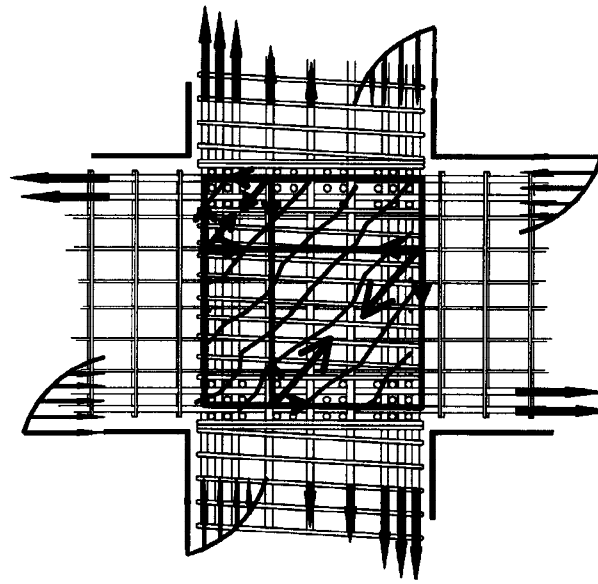

Fig. 2 Truss Mechanism

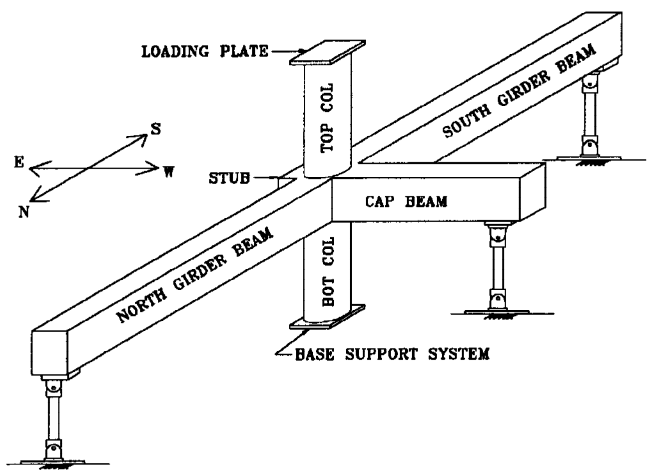

Fig. 3 Test Setup

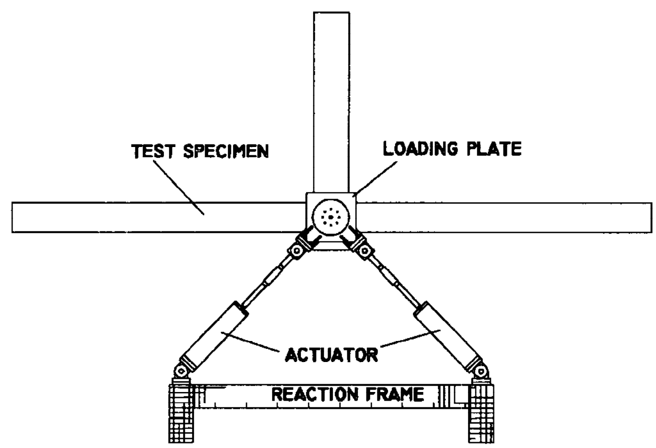

Fig. 4 Lateral Load System

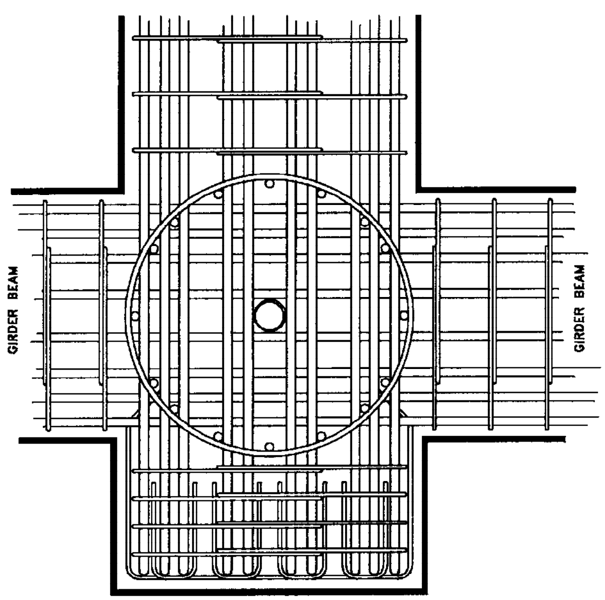

Fig. 5 Joint Detail -- Plan View

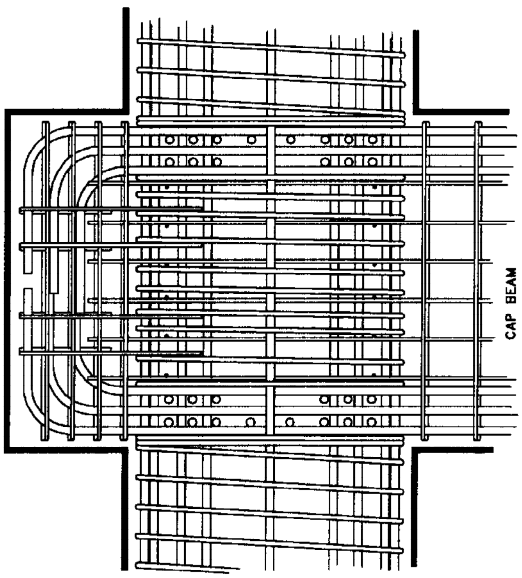

Fig. 6 Joint Detail -- South Elevation

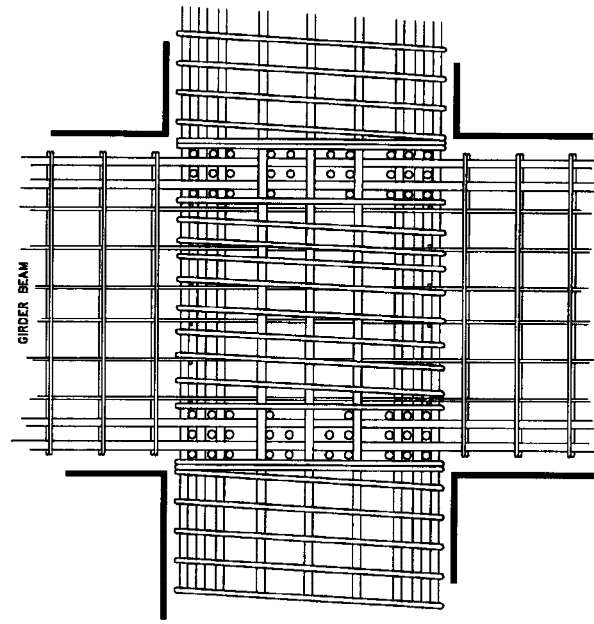

Fig. 7 Joint Detail -- East Elevation

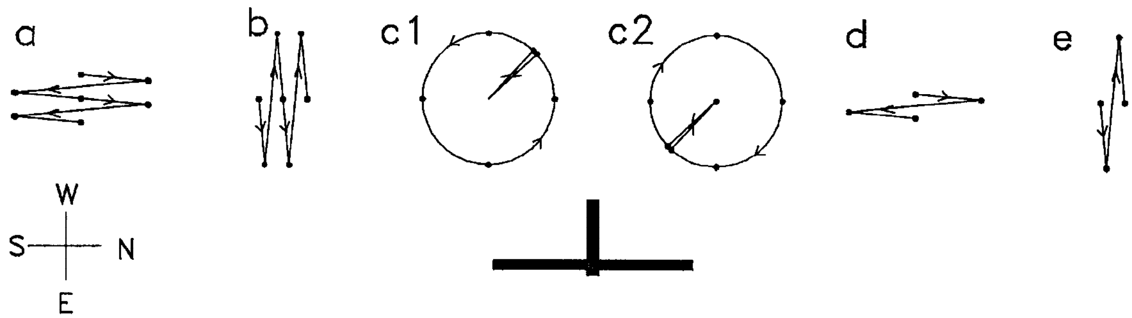

Fig. 8 Loading Scheme

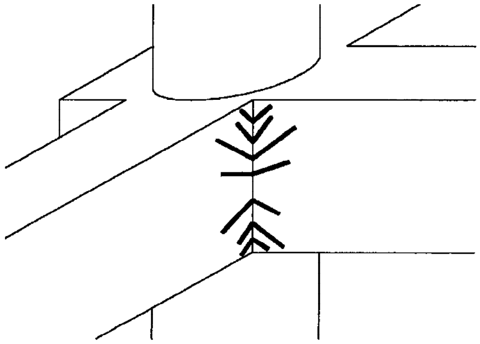

Fig. 9 Beam-End Cracks

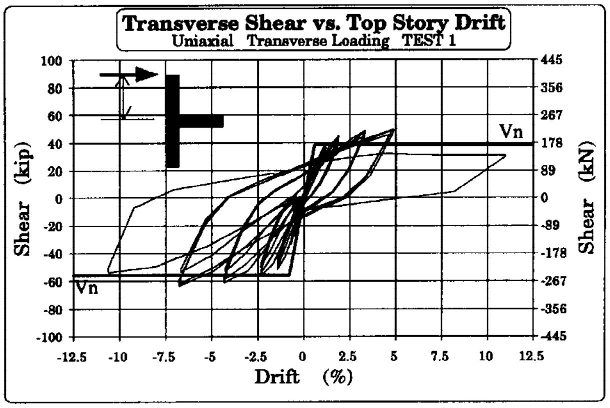

Fig. 10 Transverse Shear versus Top Story Drift TEST-1

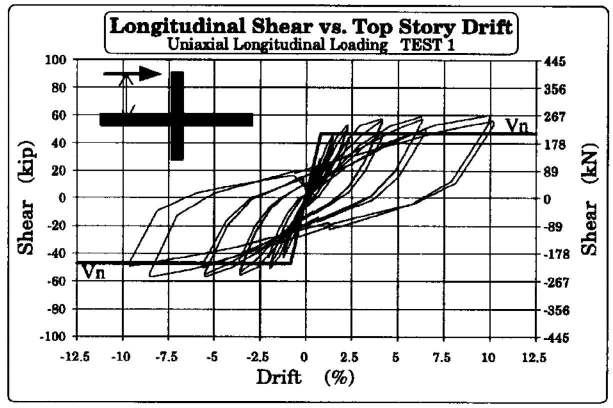

Fig. 11 Longitudinal Shear versus Top Story Drift TEST-1

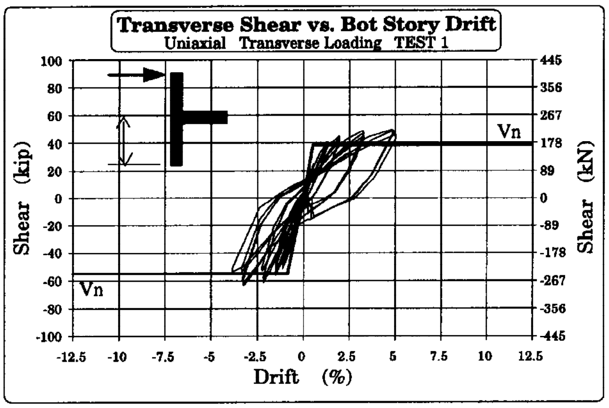

Fig. 12 Transverse Shear versus Bottom Story Drift TEST-1

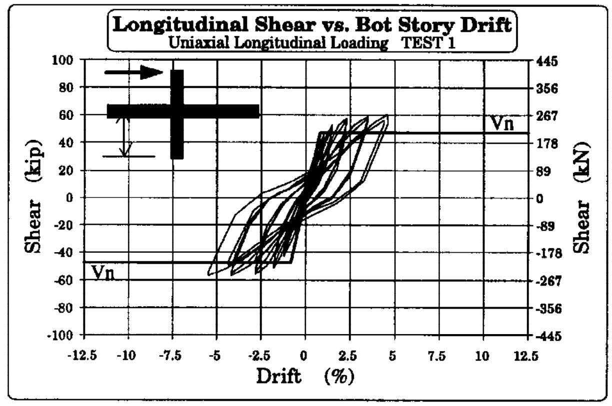

Fig. 13 Longitudinal Shear versus Bottom Story Drift TEST-1

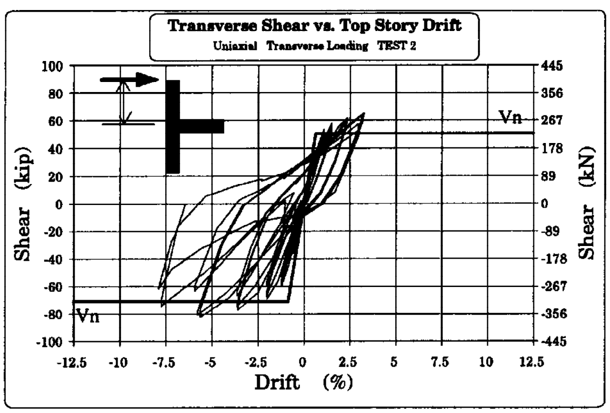

Fig. 17 Transverse Shear versus Top Story Drift TEST-2

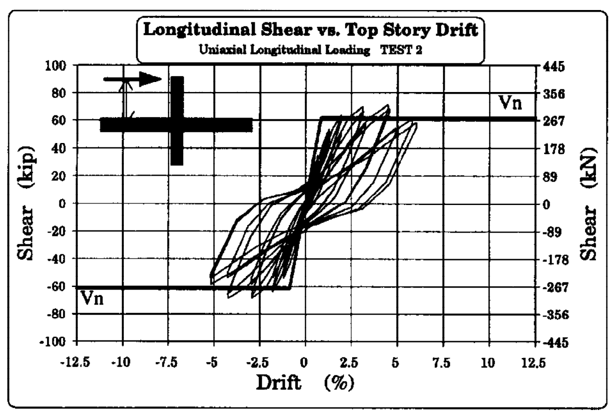

Fig. 14 Longitudinal Shear versus Top Story Drift TEST-2

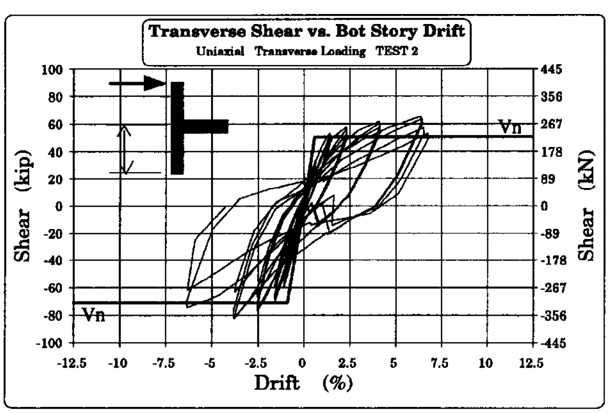

Fig. 15 Transverse Shear versus Bottom Story Drift TEST-2

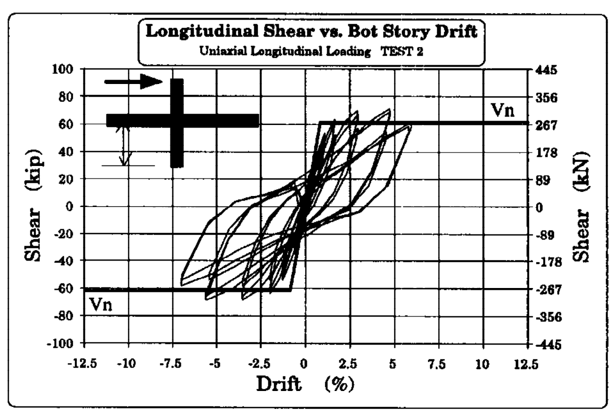

Fig. 16 Longitudinal Shear versus Bottom Story Drift TEST-2