

# Test of a Coupled Wall with High-Performance Fiber-Reinforced Concrete Coupling Beams

by R.D. Lequesne, G.J. Parra-Montesinos, and J.K. Wight

**Synopsis:** Results from the test of a large-scale coupled-wall specimen consisting of two T-shaped reinforced concrete structural walls joined at four levels by precast coupling beams are presented. Each coupling beam had a span length-depth ratio ( $\ell_n/h$ ) of 1.7, and was designed to carry a shear stress of  $7\sqrt{f'_c}$  [psi], ( $0.59\sqrt{f'_c}$  [MPa]). One reinforced concrete coupling beam was included along with three strain-hardening, high-performance fiber-reinforced concrete (HPFRC) coupling beams to allow a comparison of their behavior. When subjected to reversing lateral displacements, the system behaved in a highly ductile manner characterized by excellent strength retention to drifts of 3% without appreciable pinching of the lateral load versus drift hysteresis loops. The reinforced concrete structural walls showed an excellent damage tolerance in response to peak average base shear stresses of  $4.4\sqrt{f'_c}$  [psi], ( $0.34\sqrt{f'_c}$  [MPa]). This paper presents the observed damage patterns in the coupling beams and the structural walls. The restraining effect provided by the structural walls to damage-induced lengthening of the coupling beams is discussed and compared with that observed in component tests. Finally, the end rotations measured in the coupling beams relative to the drift of the coupled-wall system are also presented.

**Keywords:** coupling beam; coupled wall; high-performance fiber-reinforced concrete (HPFRC); precast; seismic; shear.

## 2 Lequesne et al.

**Rémy D. Lequesne** is a PhD Student in civil engineering at the University of Michigan, Ann Arbor. He was the 2007-2008 ACI Pankow Foundation Fellowship recipient, and is a member of ACI Committee 374, Performance-Based Seismic Design of Concrete Buildings. His research interests include the behavior and design of reinforced concrete members, the mechanical and structural behavior of high-performance fiber-reinforced concrete, and the earthquake-resistant design of reinforced concrete structures.

**Gustavo J. Parra-Montesinos** is an Associate Professor of civil engineering at the University of Michigan, Ann Arbor. He is Secretary of ACI Committee 335, Composite and Hybrid Structures; and a member of ACI Committee 318, Structural Building Code, and Joint ACI-ASCE Committee 352, Joints and Connections in Monolithic Concrete Structures. His research interests include the behavior and design of reinforced concrete, hybrid steel-concrete, and fiber-reinforced concrete structures.

**James K. Wight**, FACI, is a Professor of civil engineering at the University of Michigan, Ann Arbor. He is Past Chair of ACI Committee 318, Structural Concrete Building Code, and 318-E, Shear and Torsion, and is a member of Joint ACI-ASCE Committees 352, Joints and Connections in Monolithic Concrete Structures, and 445, Shear and Torsion. His research interests include the earthquake-resistant design of reinforced concrete structures and the use of high-performance fiber-reinforced concrete in critical regions of such structures.

### INTRODUCTION

Concrete structural walls are commonly used as the primary lateral force-resisting system for both medium-and high-rise concrete and steel frame structures. Due to their stiffness and strength, structural walls attract a considerable amount of lateral force when subjected to earthquake-induced displacement reversals. The efficiency of a structural wall system can be improved by proper coupling of two or more consecutive walls through the use of short coupling beams. This coupling action reduces the demand for flexural stiffness and strength from the individual walls by taking advantage of their axial stiffness, strength, and the distance between the centroidal axes of adjacent walls to provide additional resistance to overturning moment.

For satisfactory performance of a coupled-wall system during a seismic event, the short coupling beams must retain a significant, and predictable, strength and stiffness through large displacement reversals. To ensure adequate coupling beam ductility is achieved, ACI 318-08 (ACI Committee 318 2008) requires that diagonal reinforcement be provided to resist all of the shear demand in short and highly stressed coupling beams. This reinforcement detail has been shown by many researchers to provide a stable behavior under earthquake-type displacement reversals, but can be difficult and time consuming to construct. Recent coupling beam component tests (Canbolat et al. 2005; Lequesne et al. 2009) have demonstrated that precasting coupling beams with strain-hardening, high-performance fiber-reinforced concrete (HPFRC) can simplify the construction process without sacrificing performance. The HPFRC coupling beams have exhibited a highly ductile behavior when subjected to large displacement reversals, despite requiring significantly simpler reinforcement detailing than comparable reinforced concrete beams.

The impact that the ductility exhibited by HPFRC coupling beams at the component level has on a coupled-wall system, however, has not been studied experimentally. The test described herein seeks to study the interaction between precast HPFRC coupling beams and structural walls, with special attention being paid to the ductility and strength retention of the system and the deformation demands that each system component is subjected to.

In addition to the performance of the system, the construction process required for this specimen is considered to be of great importance. The most significant limitation of the current design approach is the cumbersome reinforcement that is time consuming and difficult to assemble on-site. For the specimen described herein, the precast coupling beams were inserted between the wall's longitudinal reinforcing bars, and supported by wall formwork. The remaining wall reinforcement was subsequently placed without significant interference from beam reinforcement, which protruded horizontally from the ends of the precast member and through the wall boundary element, as shown in Fig. 1. The construction of this specimen demonstrated that HPFRC coupling beams can be precast and easily embedded into cast-in-place structural walls with minimal reinforcement interference.



Fig. 1—Precast coupling beam embedment detail.

## RESEARCH SIGNIFICANCE

An experimental investigation of the implications of using precast HPFRC coupling beams on the design, constructability, and performance of coupled walls is presented. More specifically, this project seeks to: 1) demonstrate the ease with which precast coupling beams can be embedded in cast-in-place structural wall systems, 2) provide a comparison between various coupling beam details when subjected to similar deformation demands, and 3) study the interaction between HPFRC coupling beams, slabs, and structural walls.

## GENERAL DESIGN AND TEST SETUP

A diagram of the coupled-wall specimen is shown in Fig. 2. Each of the coupling beams had a slightly different reinforcement layout, shown in Fig. 3, which allowed for a comparison of various reinforcement layouts. Slabs were built at the second and

## 4 Lequesne et al.

fourth levels to facilitate lateral load application. Slab reinforcement was perpendicular to the loading direction was continuous through the structural walls, but not the precast coupling beams. The slabs provided an opportunity to observe the interaction between the precast coupling beams and the adjacent slab, and to evaluate the need for design modifications to minimize damage at this connection.

For design of the specimen, the base of each wall was assumed to be fixed, which was achieved experimentally through the use of deep reinforced concrete foundation elements bolted directly to the laboratory strong floor. A vertical force, equivalent to an axial stress of 7% of the design  $f'_c$ , based on the gross area of the walls, was applied at the second story through external prestressing tendons anchored at the bottom of the foundation elements. Steel tube sections cast through each wall above the second story slab transferred the force from the external tendons into the walls. Hydraulic jacks were used to apply this vertical force before any lateral displacement was

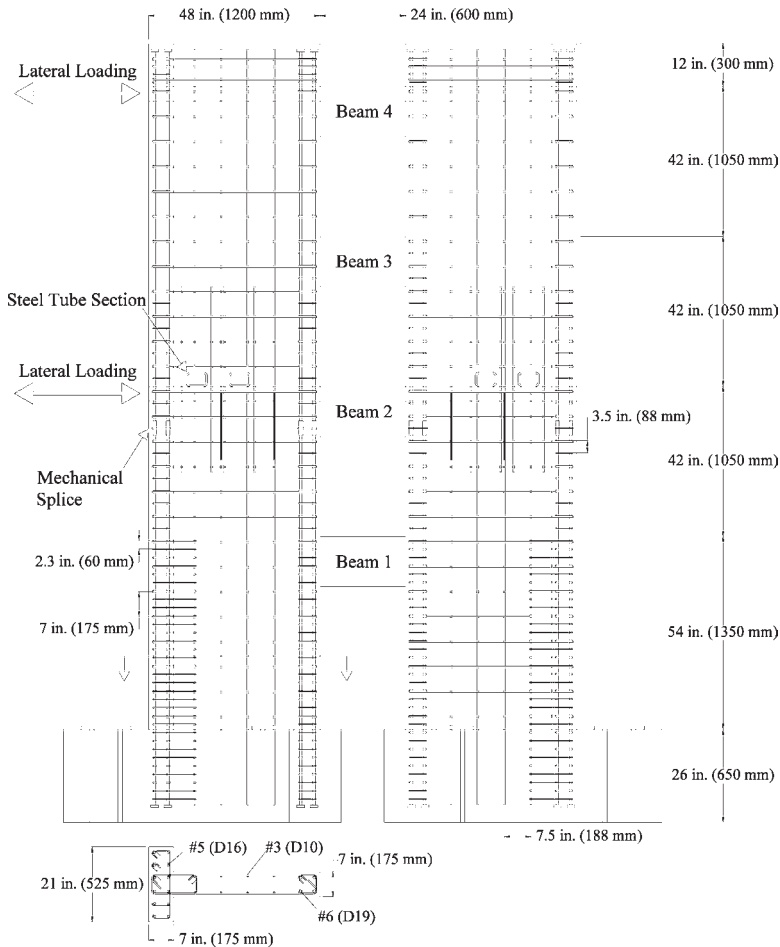


Fig. 2—Coupled-wall specimen reinforcement.

applied, and held it constant throughout the duration of the test. This level of gravity load is consistent with current design practice for structural walls, and was sufficient to offset a majority of the uplift force resulting from the coupling of the walls.

The test setup, shown in Fig. 4, was used to pseudo-statically apply lateral displacement (and load) through the slabs cast at the second and fourth levels. The actuator mounted on the fourth level applied a predetermined sequence of reversing lateral displacements, while the actuator at the second level applied a force equivalent to 60% of the force applied by the top actuator. These lateral forces were transferred to the coupled walls through a yolk and four channel sections that were attached to the top and bottom of the outer edges of the slabs. This was intended to allow for a distribution of lateral force to each of the structural walls that is similar to the load transfer mechanism that develops in a normal building system.

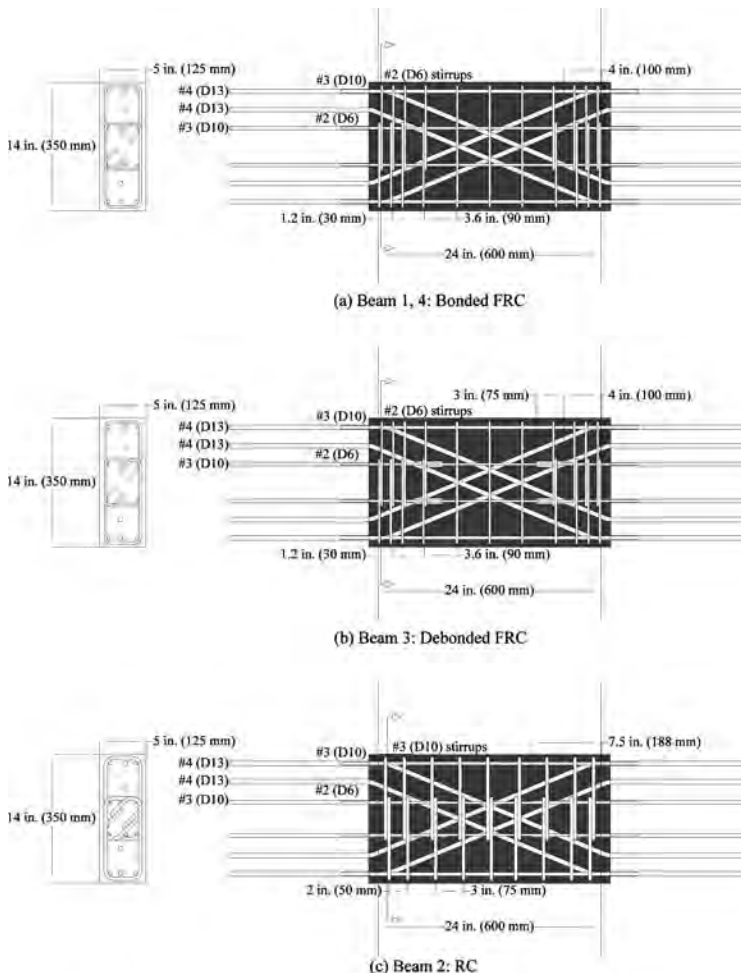


Fig. 3—Coupling beam reinforcement.



Fig. 4—Photo of test setup and specimen.

Efforts were made throughout the construction of the specimen to be as realistic as possible in terms of both construction methods and sequencing. It was felt that this approach was critical for gauging the possible construction scheduling advantages gained by incorporating precast coupling beams. The construction process consisted of first precasting the coupling beams and storing them, ready for placement. The construction of each wall story began with tying the wall reinforcement into position and then placing enough of the wall formwork to support the precast beam. The beam was then slid into position with an overhead crane and placed on the formwork that supported it fully until the wall concrete was placed. Overlapping U-shaped stirrups were used to provide confinement to the wall boundary element in the region where the coupling beam reinforcement intersected the longitudinal wall reinforcement, as shown in Fig. 1. Ensuring adequate anchorage for the special transverse reinforcement is critical, yet the preferred detail is dependent on the layout of the wall boundary element. The detail selected for this specimen consisted of overlapping U-shaped stirrups anchored by 135-degree bends around the longitudinal reinforcement. Finally, the wall formwork assembly was completed, and the concrete was placed. The formwork was then removed, moved up the wall, and the previously described sequence was repeated. The process proved to be efficient.

At the second and fourth levels, where a slab was also present, the top of the precast coupling beam was placed to be flush with the top of the slab. Although the slab concrete was cast against the precast beam, no reinforcement crossed this cold joint. It has been demonstrated that the influence of a slab on the cyclic response of coupling beams is limited to minor stiffening in early drift cycles (Paulay and Taylor 1981; Gong and Shahrooz 2001). This is due to the much lower stiffness observed in slabs relative to coupling beams when subjected to reversed cyclic loads. After the influence of the slab degrades, the slab-beam is expected to behave very similarly to a beam component

without a slab. Therefore, no attempt was made to encourage interaction between the slab and precast coupling beam, which simplified casting and placement of the precast coupling beam without sacrificing performance.

## INSTRUMENTATION

To record the performance of the coupled-wall specimen, strain gauges, linear potentiometers, inclinometers, load cells, and optical position sensors were used. Strain gauges were placed on coupling beam reinforcement, wall longitudinal reinforcement at every level, and transverse reinforcement within the first story of the wall. Linear potentiometers measured deformations in the second, third and fourth coupling beams, monitored the lateral displacement at the second and fourth levels, and measured vertical deformations along the edges of both structural walls. Inclinometers were used to monitor wall rotations at the second, third, and fourth levels. Load cells measured the forces applied laterally to the specimen, and the vertical force applied at the second level was monitored by hydraulic pressure gauges. Finally, an optical system was used to track the location of 144 independent markers placed in a grid covering the first story of both walls and the first coupling beam. This optical system provided reliable data that were used to calculate deformations, rotations, and displacements.

## MATERIAL PROPERTIES

Recent work at the University of Michigan (Liao et al. 2006) has led to the development of a flowable HPFRC with a 1.5% volume fraction of high-strength hooked steel fibers. The mixture also includes coarse aggregate with a maximum nominal size of 0.5 in. (13 mm). The properties of the fibers, as specified by the manufacturer, are summarized in [Table 1](#). This mixture was selected for the precast HPFRC coupling beams used in this study.

Results from compressive tests of 4 by 8 in. (100 by 200 mm) cylinders performed at 28 days and near the test dates are shown in [Table 2](#). The test day values of  $f'_c$  are used throughout this paper. A representative compressive constitutive response, based on cylinder tests, is shown in [Fig. 5](#), and was used for design purposes. A parabola was assumed to represent the ascending branch, followed by a shallow linear descending branch that accounted for the confinement provided by the distributed fiber reinforcement. A maximum useable concrete compressive strain of 0.8% was assumed.

Average tensile properties previously determined for this HPFRC mixture (Liao et al. 2006) were used for design of the test specimens. The representative tensile stress-strain response, shown in [Fig. 6](#), has a peak tensile stress of 500 psi (3.4 MPa) at 0.5% strain, 25% higher than the first cracking stress. This peak is followed by a gradual decrease in tensile stress capacity. On average, tensile specimens still carried 50% of their peak tensile stress at 1.4% strain.

**Table 1—Hooked steel fiber properties**

| Length (in./mm) |    | Diameter (in./mm) |      | $L/d$ | Minimum tensile strength (ksi/MPa) |      |
|-----------------|----|-------------------|------|-------|------------------------------------|------|
| 1.2             | 30 | 0.015             | 0.38 | 80    | 330                                | 2300 |

Table 2—Concrete properties

| Portion of specimen       | Fibers? | 28-day tests                        |                              |     |                                      |      |                                                 |     |     | Test day $f'_c$<br>(ksi/MPa) |    |
|---------------------------|---------|-------------------------------------|------------------------------|-----|--------------------------------------|------|-------------------------------------------------|-----|-----|------------------------------|----|
|                           |         | ASTM C1609/C1609M-05 flexural tests |                              |     |                                      |      |                                                 |     |     |                              |    |
|                           |         | $f'_c$<br>(ksi/MPa)                 | $\sigma_{fc}^*$<br>(psi/MPa) |     | $\sigma_{peak}^\dagger$<br>(psi/MPa) |      | $\sigma_{(\sigma=L/150)}^\ddagger$<br>(psi/MPa) |     |     |                              |    |
| CB-1                      | Y       | 5.5                                 | 38                           | 765 | 5.3                                  | 1090 | 7.5                                             | 540 | 3.7 | 10.4                         | 72 |
| CB-2                      | N       | 5.3                                 | 37                           | —   | —                                    | —    | —                                               | —   | —   | 9.8                          | 68 |
| CB-3                      | Y       | 5.5                                 | 38                           | 765 | 5.3                                  | 1090 | 7.5                                             | 540 | 3.7 | 10.4                         | 72 |
| CB-4                      | Y       | 6.0                                 | 41                           | 850 | 5.9                                  | 1120 | 7.7                                             | 610 | 4.2 | 10.8                         | 74 |
| Foundation                | N       | 5.0                                 | 34                           | —   | —                                    | —    | —                                               | —   | —   | 7.8                          | 54 |
| Wall 1 <sup>st</sup> lift | N       | 5.3                                 | 37                           | —   | —                                    | —    | —                                               | —   | —   | 7.0                          | 48 |
| Wall 2 <sup>nd</sup> lift | N       | 4.1                                 | 28                           | —   | —                                    | —    | —                                               | —   | —   | 6.7                          | 46 |
| Wall 3 <sup>rd</sup> lift | N       | 5.5                                 | 38                           | —   | —                                    | —    | —                                               | —   | —   | 6.6                          | 45 |
| Wall 4 <sup>th</sup> lift | N       | 6.9                                 | 48                           | —   | —                                    | —    | —                                               | —   | —   | 9.5                          | 65 |

\*bending stress at first crack; †bending stress at peak stress; ‡bending stress at a deflection of  $L/150$ .

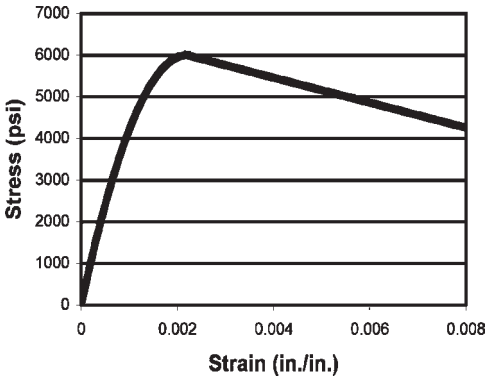


Fig. 5—Compression constitutive model. (Note: 1 in. = 24.5 mm; 1 psi = 0.0069 MPa.)

Bending tests were also performed on short beams with dimensions of 6 x 6 x 20 in. (150 x 150 x 500 mm), in accordance with ASTM C1609/C1609M-05. These tests were performed 28 days after casting each HPFRC coupling beam to characterize the bending properties of the HPFRC used in this study. The results of these tests are summarized in Table 2 by the following three values: the equivalent bending stress at first crack ( $\sigma_{fc}$ ), peak stress ( $\sigma_{peak}$ ), and at a deflection of  $L/150$ , where  $L$  is the beam span length of 18 in. (450 mm). All tests showed pronounced deflection hardening behavior, with peak bending stresses occurring near deflections of  $L/800$ , which exceeded the first cracking stress by more than 30%.

The tensile stress-strain properties of the reinforcing steel used for the construction

of this specimen were evaluated by direct tensile tests on representative coupons. The measured yield and ultimate stresses from these tests are summarized in [Table 3](#).

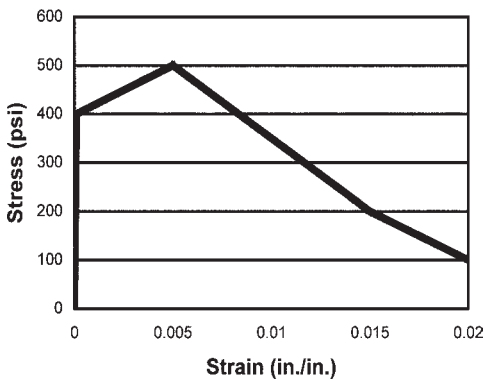


Fig. 6—Tension constitutive model. (Note: 1 in. = 24.5 mm; 1 psi = 0.0069 MPa.)

Table 3—Steel reinforcement properties

| Portion of specimen | Bar size    | Yield stress, ksi (MPa) | Ultimate stress, ksi (MPa) |
|---------------------|-------------|-------------------------|----------------------------|
| Coupling beams      | No. 3 (D10) | 79.4 (550)              | 121 (830)                  |
|                     | No. 4 (D13) | 76.9 (530)              | 115 (790)                  |
| Structural wall     | No. 3 (D10) | 74.2 (510)              | 112 (770)                  |
|                     | No. 5 (D16) | 67.2 (460)              | 109 (750)                  |
|                     | No. 6 (D19) | 68.0 (470)              | 109 (750)                  |

SPECIMEN DESIGN

Coupling beam design

Three different reinforcement details, shown in [Fig. 3](#), were selected for the coupling beams in this system. Care was taken to ensure that all of the designs exhibited similar initial stiffnesses and calculated ultimate flexural capacities, which resulted in shear stresses near  $7\sqrt{f'_c}$  [psi] ( $0.55\sqrt{f'_c}$  [MPa]). This was done to prevent any particular beam from attracting more shear than the others. One unavoidable design issue is the contribution of the fiber reinforcement to the moment capacity of fiber reinforced coupling beams, compared with conventional reinforced concrete beams. Although this contribution will increase the beam capacity near and at the first yield point, the impact will diminish at larger deformations. Significant yielding was observed in the diagonal reinforcement of all four coupling beams by the time the system drift, defined as the lateral displacement of the top slab divided by the wall height off of the foundation, reached 1%. Thus, it is reasonable to assume that shears were relatively evenly distributed between the beams at these larger drifts.

The first coupling beam design, which was used as Beams 1 and 4 in the coupled wall, is labeled “Bonded FRC,” and is shown in [Fig. 3\(a\)](#). This design is comparable to

## 10 Lequesne et al.

the component tests reported by Lequesne et al. (2009). The following notes should be made on this design:

- Diagonal bars were provided to carry approximately half of the expected ultimate shear capacity and to improve the rotational ductility at the beam ends, where plastic hinges were expected to develop. The diagonal bar contribution to shear was originally targeted to be close to 40% of ultimate, but due to scaling issues, a slightly larger diagonal bar contribution resulted. No special transverse reinforcement, except for the beam ends, was provided to prevent buckling of the diagonal bars because strain-hardening HPFRC composites have been shown to confine diagonal reinforcement and arrest any tendency to buckle (Canbolat et al. 2005).
- Longitudinal reinforcement was provided, but only embedded 3 in. (75 mm) into the walls. This is commonly done to limit the contribution of the longitudinal reinforcement to the flexural capacity of the coupling beam.
- To strengthen the interface between the precast fiber-reinforced beam and the structural wall, and to encourage plastic hinging to develop inside the fiber reinforced section, dowel bars were provided across the beam-wall interface and terminated 4 in. (100 mm) into the beam. The high bond stress developed between fiber-reinforced concrete and reinforcing bars, addressed by Chao (2005), made this very short development length sufficient to yield the dowel bars near the interface.
- Transverse reinforcement was provided in the beam for the first  $h/2$  away from the wall face to confine the beam plastic hinge regions. Stirrups were provided throughout the remaining span to carry approximately one-half of the expected shear.

The second coupling beam design, which was used as Beam 3 in the coupled wall, is labeled “Debonded FRC,” and is shown in [Fig. 3\(b\)](#). This design was identical to the previous design, with one detailing change. Within the beam, the dowel bars were extended 3 in. (75 mm) beyond the 4 in. (100 mm) development length and debonded over that added length. The term “debonding” is used herein to describe the use of mechanical means to prevent the fiber reinforced concrete from bonding with the reinforcing bar. This was accomplished by wrapping the bar with a few layers of plastic sheeting, and sealing it with tape. The intent was to delay the development of a single failure plane by eliminating the disturbance resulting from the physical discontinuity of the terminated bar. The motivation for this detail came from the observation that the dowel bars in previous component tests were successful in moving the ultimate failure plane away from the interface to the plane where the dowel bars were terminated. If possible, it would be advantageous to spread that flexural yielding out through a larger portion of the coupling beam, thus further delaying the localization of rotations.

The third coupling beam design, which was used as Beam 2 in the coupled wall, is labeled “RC,” and is shown in [Fig. 3\(c\)](#). This reinforced concrete beam design was unique because it investigated the potential for precasting non-fiber-reinforced concrete coupling beams, which could offer construction time-savings if proven to be successful. To account for the precasting and embedment of this coupling beam, ACI 318-08 requirements were modified, and a detail more similar to the “Bonded FRC” design discussed previously was selected. The following modifications to the “Bonded FRC” design were made to account for the lack of fiber reinforcement.

- The dowel bars were extended 7.5 in. (188 mm) into the span of the beam. This longer development was required to compensate for the lower bond stress capacity developed between conventional concrete and reinforcing steel, compared with fiber reinforced concrete.
- The transverse reinforcement provided in the plastic hinge region was approximately doubled (larger diameter) to compensate for the loss of confinement from the fiber reinforcement.
- The transverse reinforcement in the remaining span was approximately doubled (larger diameter and reduced spacing) when compared with the fiber reinforced beams. This provided confinement to the diagonal bars, preventing buckling, and also compensated for the loss of the contribution of the fiber reinforcement to the shear capacity of the section.

### Structural wall design

The coupling beam dimensions and detailing were of special interest in this project; thus, they were the initial focus of the coupled wall design. The structural walls were subsequently designed following ACI 318-08 to provide the required overturning moment capacity and ductility for the coupled-wall system to behave realistically.

The wall shear design was based on the expected ultimate capacity of the system, assuming that a mechanism controlled by flexural hinging in the base of both walls and in each of the beams would develop. To resist the expected shear demand, the wall concrete was assumed to carry a shear stress equivalent to  $2\sqrt{f'_c}$  [psi] ( $0.17\sqrt{f'_c}$  [MPa]) and wall transverse reinforcement, anchored by alternating 90- and 135-degree bends and representing a transverse reinforcement ratio of 0.45%, was provided to resist the remaining shear.

The coupling ratio, which expresses the overturning moment resistance from the wall axial forces generated by the “coupling action” as a fraction of the total overturning moment resistance of the coupled-wall system, was selected to be between 0.35 and 0.40. While some researchers have used alternative definitions for the coupling ratio, it is defined herein based on the calculated ultimate flexural capacities of the system components. This is a more reasonable definition for this system, because the ductile response expected from fiber-reinforced coupling beams increases the likelihood that a plastic mechanism will develop in which all beams and walls are concurrently near their respective ultimate capacities. Testing of the system demonstrated that a coupling ratio of approximately 0.4 was attained at a system drift near 0.75%, and sustained out to drifts past 2.5%. Given this result, defining the coupling ratio using ultimate capacities appears to be reasonable for systems similar to the one tested and coupled with HPFRC coupling beams. The axial effects in each structural wall due to the combined shearing of the coupling beams and the externally imposed gravity load are critical for the flexural capacity predictions at the base of each wall, and were thus accounted for in the design of the system.

T-shaped wall sections were used to simulate the fact that most coupled walls in a core-wall structural system have a flange at the end of the wall away from the coupling beam. Thus, such walls tend to have different flexural capacities when subjected to “positive” or “negative” bending about their principal axis, as was the case for the

## 12 Lequesne et al.

walls used in this test structure. These flanged sections had the added benefit of providing improved lateral stability during testing.

ACI 318-08 compliant mechanical couplers were used at mid-height of the walls to splice longitudinal reinforcement in the boundary elements. Mechanical anchorages were used to develop that same reinforcement in the foundation block and in the short extension at the top of the walls. Conventional lap splices were used for the distributed wall web reinforcement.

## RESULTS

### Overall behavior and damage

A plot of the overturning moment versus drift response of the coupled-wall specimen is shown in Fig. 7, along with the predicted capacity with and without the effect of the coupling beams. Assuming the plastic mechanism described above developed, and accounting for the measured concrete and reinforcing steel properties, the system overturning moment capacity was under-predicted by only approximately 5%. Ninety percent of the system's ultimate overturning moment capacity was maintained up to drifts of 3%, which is a significant level of deformation for coupled reinforced concrete shear walls. When the system drift exceeded 2.5%, the coupling beams were all subjected to drift demands exceeding 4.5%. Coupling beam drift is used herein to describe the chord rotation referenced in ASCE/SEI 41/06 (2007). The observed coupling beam drift demands exceeding 4.5% emphasize the need for highly ductile coupling beams, and the stable system response is evidence that the coupling beams used in this system were capable of withstanding these drift demands. Furthermore, the full hysteresis loops show no appreciable pinching, which indicates that the response of the system was governed by flexural hinging in the bases of the walls and at the ends of the coupling beams, as predicted.

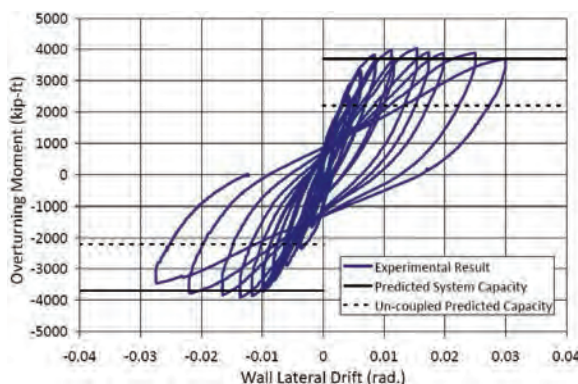


Fig. 7—Overturning moment versus wall lateral drift response. (Note: 1 kip-ft = 1.356 kN-m.)

Throughout the testing process, a coordinated team of students carefully monitored development of cracks over the full height of the specimen. At a system drift of 0.25%, diagonal-shear cracks were observed in the first story of the “compression” wall.

Throughout this paper, “compression” wall is used to refer to the wall that, due to the coupling action of the beams, was subjected to increased compression within a particular loading half-cycle. As previously shown (Aktan and Bertero 1984; Teshigawara et al. 1998b), this increased compression caused the compression wall to attract a higher shear stress than the opposing “tension” wall, resulting in more pronounced shear cracking on the compression side of the system. Similar diagonal-shear cracking developed in the first story of the opposing wall after reversal of the loading direction.

At a system drift of 0.5%, diagonal-shear cracking was observed in all four coupling beams. Also at this drift level, the following pattern of damage was first observed in the lower levels of the coupled wall. Diagonal cracks formed in the second story of the tension wall, progressed through the coupling beam between the first and second stories, and extended as diagonal cracks through the first story of the compression wall to the foundation. This crack pattern indicated that at low drifts, the coupling of the system was sufficiently high to cause the walls to act as one unit with a prominent diagonal compression strut extending through the first two levels of the wall. Although diagonal-shear cracking was prominent in both walls during early drift cycles, the transverse reinforcement ( $\rho_t = 0.45\%$ ) successfully arrested the opening of these diagonal cracks. This allowed a ductile flexural mechanism to develop and accommodate the rotation demands placed on the walls.

ACI 318-08 requirements for structural wall design provided adequate shear resistance and confinement of longitudinal reinforcement to allow for a stable flexural mechanism to develop in the base of both walls. Once the walls and coupling beams reached their respective flexural capacities, which occurred near a system drift of 0.75%, very little additional diagonal cracking was observed in the structural walls. Further increases in lateral drift caused a wider opening of flexural cracks in the walls, and progressively more severe damage to occur in the coupling beams. The test was terminated once the integrity of the structure had been compromised by the fracturing of diagonal reinforcement in the coupling beams and flexural reinforcement in the walls.

Due to the termination of coupling beam flexural reinforcement only 3 in. (75 mm) into the wall, the moment capacity near the beam-wall interface was lower than within the beam span. The bars were terminated to be consistent with current design practice in the United States, but due to the modified flexural and shear design of these coupling beams, the flexural reinforcement represented a higher reinforcement ratio than would commonly be used in practice. The resulting plane of weakness where the coupling beam moment demands are greatest led to a localization of rotations at the precast beam/wall interface. The result was crushing and spalling of the wall concrete near the interface at system drifts exceeding 2.5%, which preceded the development of a more desirable damage pattern within the precast beam. Any meaningful comparison of the respective coupling beam reinforcement layouts was thus impossible. This undesirable pattern of damage at the interface between the precast section and the wall was not observed in component tests (Lequesne et al. 2009), where the flexural reinforcement was fully developed into the wall. Despite this undesirable localization of damage, the coupling ratio and ductility of the overall system were not compromised. In a second coupled-wall specimen, currently being tested, this reinforcement is fully developed into the wall, which will likely force damage to localize within the span of the coupling

## 14 Lequesne et al.

beam, as observed in component tests. This will allow for a more direct comparison of the beam designs that was not possible in this test.

The first yielding in the system was observed in the diagonal reinforcement of all four coupling beams, and was followed by yielding along the tension face of the compression wall. The T-shaped wall section provided a wider compression zone in the compression wall, resulting in a smaller depth to the neutral axis and thus, larger tensile steel strains and rotations despite the significantly higher axial loads acting on the compression side of the system. This trend continued throughout the test, with larger rotations consistently being observed in the base of the wall on the compression side of the system relative to the tension side.

The interaction between the slab and the precast coupling beam was also carefully observed throughout the test. Because no reinforcement was provided across the interface between the precast section and the adjacent slab, the potential for relative vertical displacements requiring repair was a concern. Minor cracking along the slab-beam interface was first observed at a system drift of 1.5%. Measurable relative vertical displacements of approximately 0.125 in. (3 mm) were observed between the top of the precast section and the top of the slab at system drifts exceeding 2.5%, indicating a de-coupling of their respective responses.

### Elongation of coupling beams

When reinforced concrete members are subjected to cyclic displacements large enough to cause significant cracking and yielding of the reinforcement, it is widely acknowledged that the cracks will not close completely upon reversal of the loading direction. Thus, reinforced concrete members have a tendency to expand longitudinally when subjected to earthquake-type cyclic displacements. In most design cases, the resulting axial strain is either too small or insufficiently restrained to cause significant axial forces to develop. The large drift demands placed on short coupling beams, however, result in a strong tendency to expand longitudinally, and the adjacent structural walls and surrounding slab should provide non-negligible resistance to this expansion, as identified by Teshigawara et al. (1998a).

In component tests, few researchers have addressed longitudinal expansion of coupling beams and the possible axial forces that may develop as a result. Most experimental work has allowed for unlimited axial movement, which has been reported to be as high as 4.0% (Kwan and Zhao 2002; Zhao and Kwan 2003). In recent tests of coupling beam components with aspect ratios ( $\ell_n/h$ ) of 1.75 (Lequesne et al. 2009), longitudinal expansion was partially restrained, which led to maximum average axial strains between 0.5 and 1.0%. The result was the development of axial forces within the coupling beams as high as 55% of the applied transverse shear force at coupling beam drifts over 4%.

The coupled-wall test described herein provided an opportunity to observe the magnitude of longitudinal beam strains permitted by a realistic coupled-wall system. Fig. 8 shows the measured average axial strains of the coupling beams at all four levels plotted against the drift imposed at the top of the system. The negative axial strain (compression) observed in the coupling beam at the first level is evidence of a significant axial force acting on this member throughout the test. This axial force resulted in a shifting of base shear towards the compression side of the system consistent with previously

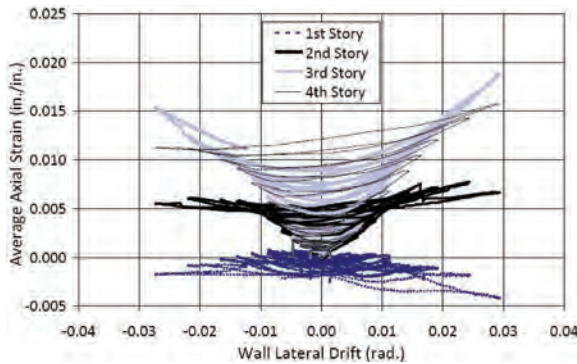


Fig. 8—Coupling beam longitudinal strains. (Note: 1 in. = 24.5 mm.)

tested coupled wall systems (Aktan and Bertero 1984; Teshigawara et al. 1998b).

This may support the observation, based on wall crack patterns, that a diagonal compression strut developed through the first two stories of the system. While the development of this strut may partially have been an artifact of the loading mechanism, the large axial force developed in the first coupling beam warrants further investigation given the strong influence it may have on the response of coupling beams.

Positive (tension) axial strains were observed in the other three beams, with a general trend towards larger axial strains further from the foundation. The coupling beam at the second level did expand longitudinally, with measured maximum axial strains on the order of 0.75%. If correlated with the component tests cited previously, axial strains of this magnitude indicate significant restraint from the adjacent walls, and thus, the development of a significant axial compression force that will affect the capacity and ductility of the coupling beam. The coupling beam expansions at the third and fourth levels were somewhat larger than at the lower levels; however, the beam at the third level exhibited the largest axial strains. This result can be explained by the presence of the slab around the beam at the fourth level, which is evidence that a slab adjacent to a coupling beam may appreciably affect the axial restraint of the beam. This restraint appears to be large enough to develop axial forces in the coupling beam, regardless of the diminished restraint provided by the walls at a point further from the foundation. The extent to which these observations apply to taller and monolithically cast coupled-wall systems will need to be investigated further.

## Beam/wall rotations

More often than not, the design of coupled-wall structural systems is motivated by a need to control the lateral drift of a structure. The design level drifts assumed for coupling beams within the coupled system are then estimated relative to the expected system drift as a function of the distance between the centroidal axes of the structural walls. This relationship is represented by:

$$\theta_b = \frac{\theta_w \cdot \ell}{\ell_b + h_b} \quad (1)$$

where  $\theta_b$  = coupling beam rotation,  $\theta_w$  = wall rotation,  $\ell$  = distance between the centroidal axes of the walls,  $\ell_b$  = clear span of coupling beam, and  $h_b$  = height of coupling beam.

The  $h_b$  term in the denominator provides an estimate of the length over which beam deformations extend into the wall, shown in Fig. 9 (Wight and MacGregor 2009) as  $h_b/2$  into each wall. This relationship has been suggested for cast-in-place systems; however, the simplified construction method proposed as part of this project, which takes advantage of precast coupling beams shallowly embedded into the cast-in-place walls, may behave differently. Thus, the proposed method may require a different equivalent beam length to approximate expected beam rotations.

For the coupled wall tested, Eq. (1) would predict beam rotations 2.1 times greater than the wall rotations. In this case,  $\ell$  is assumed to be the distance between the centroidal axes of the uncracked walls. Although the relationship between the beam

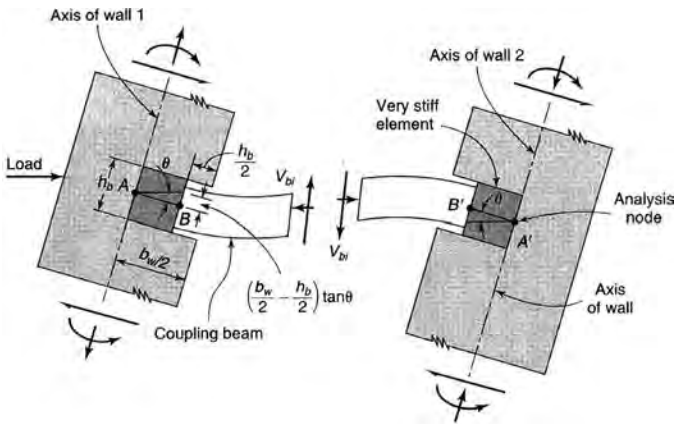


Fig. 9—Coupling beam rotations (Wight and MacGregor 2009).

and wall rotations varied throughout the test, results indicate that  $\theta_b = (1.5 - 2.0) \theta_w$ . It thus appears that the model used for cast-in-place systems may be slightly conservative, but reasonable for use in coupled wall systems with precast HPFRC coupling beams. Additional experimental data, however, are needed to fully validate this observation.

## SUMMARY AND CONCLUSIONS

A large-scale four-story coupled-wall specimen was constructed and tested to investigate the impact that precast HPFRC coupling beams have on the design, construction, and behavior of coupled-wall systems. Although at the time of this paper submission a second complementary coupled-wall specimen is in the process of being tested, the following preliminary conclusions can be drawn from the test described herein.

- Placing the precast coupling beams proved to be simple, and is believed to be a viable alternative method for assembling a coupled-wall system. The proposed development detail allows for the structural wall reinforcement to continue, uninterrupted, through the connection with the coupling beam.

- For precast coupling beams, terminating even moderate amounts of longitudinal reinforcement within the wall near the interface is not recommended. This is believed to have caused a concentration of damage along the precast/cast-in-place interface at larger coupling beam drifts that was not observed in component tests with fully developed longitudinal reinforcement designed as described in Lequesne et al. (2009). The overall system behavior, however, was not significantly impacted by this localization of damage at the coupling beam/wall interface.
- The observed coupling beam rotations, relative to the structural wall drift, were slightly less than those predicted by existing design methods. The proposed precast connection scheme does not seem to significantly impact the relevance of the predictive models traditionally used for cast-in-place systems.
- It is recommended that for systems coupled by ductile HPFRC coupling beams, the expected ultimate capacities for the individual beams and walls should be used to define the design coupling ratio for the system. It was shown that the design coupling ratio, defined in this manner, is achieved at relatively moderate drift levels and largely maintained until failure of the system.
- Measurements taken throughout the test indicate that the tendency for coupling beams to elongate when subjected to reversing displacements is restrained by the adjacent structural walls and floor slabs. When correlated to component tests of coupling beams, the axial elongations permitted by the walls (shown in Fig. 8) imply the development of significant axial forces that will impact both the strength and ductility of the beams.
- The cast-in-place slab adjacent to the precast beam did not develop appreciable differential displacements relative to the beam until system drifts exceeded 1.5%. Even at these large drifts, the interface between the slab and beam was largely undamaged.
- Although not a focus of the current study, ACI 318-08 requirements for structural wall design provided adequate shear resistance and confinement of longitudinal reinforcement to allow for a stable flexural mechanism to develop in the base of both walls.

## ACKNOWLEDGMENTS

This research project is funded by the National Science Foundation under Grant No. CMS 0530383. It is a part of the NEES research program. Special thanks go to Bekaert Corp. and Erico Corp. for donations of materials used in construction of the specimen described herein. The ideas and conclusions expressed are those of the writers, and do not necessarily represent the views of the sponsors.

## REFERENCES

- ACI Committee 318, 2008, "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary," American Concrete Institute, Farmington Hills, MI, 465 pp.
- Aktan, A. E., and Bertero, V. V., 1984, "Seismic Response of R/C Frame-Wall Structures," *Journal of Structural Engineering*, ASCE, V. 110, No. 8, pp. 1803-1821.
- ASCE/SEI 41/06, 2007, "Seismic Rehabilitation of Existing Buildings," American Society of Civil Engineers, Reston, VA.

ASTM C1609/C1609M-05, 2005, "Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam With Third-Point Loading)," ASTM, West Conshohocken, PA.

Canbolat, B. A.; Parra-Montesinos, G. J.; and Wight, J. K., 2005, "Experimental Study on Seismic Behavior of High-Performance Fiber-Reinforced Cement Composite Coupling Beams," *ACI Structural Journal*, V. 102, No. 1, Jan.-Feb., pp. 159-166.

Chao, S. H., 2005, "Bond Characterization of Reinforcing Bars and Prestressing Strands in High Performance Fiber Reinforced Cementitious Composites Under Monotonic and Cyclic Loading," PhD dissertation, University of Michigan, Ann Arbor, MI, 475 pp.

Gong, B., and Shahrooz, B. M., 2001, "Concrete-Steel Composite Coupling Beams II: Subassembly Testing and Design Verification," *Journal of Structural Engineering*, V. 127, No. 6, pp. 632-638.

Kwan, A. K. H., and Zhao, Z. Z., 2002, "Cyclic Behaviour of Deep Reinforced Concrete Coupling Beams," *Structures and Buildings*, V. 152, No. 3, pp. 283-293.

Lequesne, R.; Setkit, M.; Parra-Montesinos, G. J.; and Wight, J. K., 2009, "Seismic Detailing and Behavior of Coupling Beams with High-Performance Fiber Reinforced Concrete," *Antoine E. Naaman Symposium—Four Decades of Progress in Prestressed Concrete, Fiber Reinforced Concrete, and Thin Laminate Composites*, ACI Special Publication. (in press)

Liao, W. C.; Chao, S. H.; Park, S. Y.; and Naaman, A. E., 2006, "Self-Consolidating High Performance Fiber Reinforced Concrete (SCHPFRC)—Preliminary Investigation," *Technical Report* No. UMCEE 06-02, University of Michigan, Ann Arbor, MI, 68 pp.

Paulay, T., and Taylor, R. G., 1981, "Slab Coupling of Earthquake-Resisting Shearwalls," *ACI Structural Journal*, V. 78, No. 2, pp. 130-140.

Teshigawara, M.; Kato, M.; Sugaya, K.; and Matsushima, Y., 1998a, "Energy Absorption Mechanism and the Fluctuation of Shear Force in the Coupled Shear Walls," *Structural Engineers World Congress 1998*, Paper No. T-186-5, Elsevier Science Ltd.

Teshigawara, M.; Sugaya, K.; Kato, M.; and Matsushima, Y., 1998b, "Seismic Test on 12-Story Coupled Shear Wall with Flange Walls," *Structural Engineers World Congress 1998*, Paper No. T-186-4, Elsevier Science Ltd.

Wight, J. K.; and MacGregor, J. G., 2009, *Reinforced Concrete: Mechanics and Design*, fifth edition, Pearson Education, Inc., Upper Saddle River, NJ, 943 pp. (reprinted by permission)

Zhao, Z. Z.; and Kwan, A. K. H., 2003, "Nonlinear Behavior of Deep Reinforced Concrete Coupling Beams," *Structural Engineering and Mechanics*, V. 15, No. 2, pp. 181-198.

# Shear Strength Prediction of Eccentric Beam-Column Joints

by E. Lim, S.-J. Hwang, and H.-J. Lee

**Synopsis:** Several studies showed that the eccentricity between beam and column connections has a detrimental effect on the joint shear strength. With regard to this issue, ACI 318-08 restricts the average shear stress on a horizontal plane within the joint, which equals to the effective joint width times column depth. The formula of effective joint width given in ACI 318-08 may be too conservative for eccentric beam-column joints. This paper suggested a more rational formula of effective joint width associated with the softened strut-and-tie (SST) model for eccentric beam-column joints. Using the proposed effective joint width, the shear strength predictions of SST model agreed well with the results of 18 eccentric joint specimens failed in shear. Furthermore, together with the proposed effective joint width, several available definitions for effective joint width are also used as comparisons to estimate joint shear strength of collected database for eccentric and concentric joints using ACI 318-08 code design equation. The proposed effective joint width was successfully verified with available database of beam-column joints with or without eccentricity in literature.

**Keywords:** eccentric beam-column joint; effective joint width; shear strength; softened strut-and-tie.

**Erwin Lim** is a PhD student in the Department of Civil Engineering at the National Taiwan University, Taipei, Taiwan, where he earned his MS in 2009. He received his BS from Institute of Technology Bandung, Indonesia, in 2005. His research interests include seismic behavior of beam-column joints and shear strength of deep beams.

**Shyh-Jiann Hwang** is Professor of Civil Engineering at the National Taiwan University, Taipei, Taiwan. He received his PhD from the University of California at Berkeley in 1989. He is a member of ACI Committees 318, Structural Concrete Building Code, and 369, Seismic Repair and Rehabilitation. His research interests include seismic behavior of beam-column joints, shear strength of reinforced concrete members, and seismic retrofitting of reinforced concrete structures.

**Hung-Jen Lee** is Associate Professor of Construction Engineering at the National Yunlin University of Science and Technology, Yunlin, Taiwan. He received his PhD from the National Taiwan University of Science and Technology, Taipei, Taiwan, in 2000. He is an associate member of Joint ACI-ASCE Committee 352 (Joints and Connections in Monolithic Concrete Structures). His research interests include earthquake-resistant design, modeling, and testing on reinforced concrete elements and structures.

### INTRODUCTION

When designing buildings with seismic threats, plastic hinges are expected to be at beam ends to ensure good energy dissipation during earthquakes. This causes very large forces to be transferred among elements through the beam-column joints. Although concentric connections are more desirable to guarantee a smooth force transfer, eccentric connections ([Fig. 1](#)) are sometimes unavoidable due to architectural considerations.

During the Tokachioki Earthquake in 1968, Joh et al. (1991) found that several reinforced concrete (RC) frame buildings which had eccentric joints were heavily damaged. Several researchers in Japan also reported similar findings from the Hyogoken-Nanbu earthquake in 1995 (Goto and Joh 2003). ACI 318-08 (ACI Committee 318 2008) addresses the issue of joint eccentricity by strictly limiting the effective joint width to be equal to the beam width if beams are flush with columns. ACI 352R-02 (ACI-ASCE Committee 352 2002) further introduced the eccentricity between the column and beam axes in the design equations. The study concluded that ACI 318-05 (ACI Committee 318 2005) gives a very conservative provision for the effective joint width. Meanwhile, ACI 352R-02 was reported to be less conservative in terms of the effective joint width, and only addresses that issue to a limited extent (LaFave et al. 2005). Based on comparisons with test data, LaFave et al. (2005) suggested that the effective joint width can be taken as the average between the beam and column width for the ACI 318-05 code design equation, yielding reasonable estimations of shear strength for eccentric beam-column joints.

Moreover, the available code approach is realized to be an empirical approach. As future development of building design may move towards the high-rise buildings using high-strength materials, the issue of wide columns and narrow beams in high-rise buildings may become a challenge, and may not be treated well by the current

code provision. Therefore, a more rational approach in determining the joint strength is needed.

Among the available joint models, Hwang and Lee (1999, 2000) developed a softened strut-and-tie (SST) model as an analytical approach for predicting the shear strength of concentric joints. This model satisfies the force equilibrium, Mohr's compatibility, and softened reinforced concrete constitutive laws.

This paper adopts the SST model together with a rational derivation on how to determine the effective joint width to predict the shear strength of eccentric beam-column joints. The accuracy of the proposed model is gauged by the available experimental data for eccentric joints. As many as 18 specimens of eccentric beam column joints with joint reinforcement and shear failure are collected. Furthermore, 22 specimens of concentrically connected interior joints and 23 specimens of concentrically connected exterior joints with joint reinforcement larger than half of that required by ACI 318-08 are also collected. These databases are used to verify the accuracy and applicability of proposed effective joint width with ACI 318-08 code equation. Several available definitions for effective joint width are also studied as comparison to the analytically derived effective joint width.

## RESEARCH SIGNIFICANCE

This paper extends the applicability of SST model by deriving a more rational equation for effective beam-column joint width. For the design purpose, the derived effective joint width is further simplified to be studied with the current ACI 318-08 design equation. Several definitions of effective beam-column joint width are also used as comparisons to predict joint shear strength of collected database for both concentric and eccentric joints. The accuracy and applicability of different definitions of effective joint width are reported.

## SOFTENED STRUT-AND-TIE MODEL

A common strut-and-tie model applied to joint region usually only satisfies the equilibrium condition (Hsu 1996). Hwang and Lee (1999, 2000) further developed this model to satisfy the basic principles of mechanics of materials for predicting the shear strength of concentric joints. This model, the SST model, satisfies the non-uniformly distributed stresses by checking the force equilibrium using strut-and-tie model (Fig. 2(a)). Meanwhile, the two-dimensional strain compatibility conditions is satisfied through Mohr's compatibility (Fig. 2(b)), and the adopted constitutive law for the cracked reinforced concrete is based on the panel test which represents uniform stress distribution as indicated in Fig. 2(c) (Zhang and Hsu 1993). The failure criteria assumed in this model is the concrete compression failure at strut adjacent to the nodal zone. Figure 3 gives a conceptual solution procedure for the general SST approach (Hwang and Lee 1999, 2000). This approach, however, requires a trial-and-error procedure to obtain a convergent solution, and is therefore difficult to use in practice.

In 2002, Hwang and Lee (2002) further simplified the solution procedure and suggested that the joint shear strength,  $V_j$ , can be predicted using the following equation:

$$V_j = K \zeta f'_c b_j a_s \cos \theta \quad (1)$$

where  $K$  is the strut-and-tie index;  $\zeta$  is the softening coefficient;  $f'_c$  is the compressive

## 22 Lim et al.

strength of a standard concrete cylinder;  $b_j$  is the effective joint width;  $a_s$  is the depth of the compression strut; and  $\theta$  is the angle of strut inclination to the horizontal axis. In this paper, the effective joint width is redefined for the eccentric beam-column joint.

Due to the presence of joint reinforcement and intermediate column bars which act as tension ties, more concrete is activated to resist shear strain. A factor representing this beneficial effect in the shear strength is given by the strut-and-tie index,  $K$  (Hwang and Lee 2002):

$$K = K_h + K_v - 1 \quad (2)$$

where  $K_h$  and  $K_v$  are the horizontal and vertical tie indices as shown:

$$K_h = 1 + \frac{(\bar{K}_h - 1)A_{th}f_{yh}}{\bar{F}_h} \leq \bar{K}_h \quad (3a)$$

$$K_v = 1 + \frac{(\bar{K}_v - 1)A_{iv}f_{yv}}{\bar{F}_v} \leq \bar{K}_v \quad (3b)$$

Here,  $A_{th}$  and  $A_{iv}$  are the areas of the horizontal ties and column intermediate bars within the effective joint width (Hwang and Lee 1999, 2000), respectively;  $f_{yh}$  and  $f_{yv}$  represent the yield strength of the horizontal ties and the column intermediate bars, respectively; and  $K_h$  and  $K_v$  are the horizontal and vertical tie index with sufficient horizontal and vertical reinforcement (at which the horizontal and vertical ties are kept in elastic range while the maximum strength is attained) which are estimated to be:

$$\bar{K}_h \approx \frac{1}{1 - 0.2(\gamma_h + \gamma_h^2)} \quad (4a)$$

$$\bar{K}_v \approx \frac{1}{1 - 0.2(\gamma_v + \gamma_v^2)} \quad (4b)$$

where  $\gamma_h$  is the fraction of horizontal shear transferred by horizontal ties in the absence of intermediate vertical column bars and  $\gamma_v$  is the fraction of vertical shear transferred by the intermediate vertical column bars in the absence of the horizontal ties (CEB-FIP 1993), such that:

$$\gamma_h = \frac{2 \tan \theta - 1}{3} \quad \text{for } 0 \leq \gamma_h \leq 1 \quad (5a)$$

$$\gamma_v = \frac{2 \cot \theta - 1}{3} \quad \text{for } 0 \leq \gamma_v \leq 1 \quad (5b)$$

Here,  $\bar{F}_h$  and  $\bar{F}_v$  are the balanced amount of the horizontal tie force and the column intermediate bar force, respectively:

$$\bar{F}_h = \gamma_h \times (\bar{K}_h \zeta f'_c b_j a_s) \times \cos \theta \quad (6a)$$

$$\bar{F}_v = \gamma_v \times (\bar{K}_v \zeta f'_c b_j a_s) \times \sin \theta \quad (6b)$$

Figure 2(b) describes the strain compatibility satisfied by an SST model. The first strain invariant satisfies Mohr's two dimensional compatibility as given in Eq. (7):

$$\varepsilon_v + \varepsilon_h = \varepsilon_d + \varepsilon_r \quad (7)$$

where  $\varepsilon_v$  and  $\varepsilon_h$  are average normal strains in  $v$ - and  $h$ - directions, respectively and  $\varepsilon_d$  and  $\varepsilon_r$  are average normal strains in  $d$ - and  $r$ - directions, respectively.

The constitutive laws satisfied by this model adopts from softened reinforced concrete stress-strain curve suggested by Zhang and Hsu (1993). The ascending branch of the softened reinforced concrete stress-strain curve is:

$$\sigma_d = -\zeta f'_c \left[ 2 \left( \frac{-\varepsilon_d}{\zeta \varepsilon_0} \right) - \left( \frac{-\varepsilon_d}{\zeta \varepsilon_0} \right)^2 \right] \quad \text{for} \quad \frac{-\varepsilon_d}{\zeta \varepsilon_0} \leq 1 \quad (8a)$$

where  $\sigma_d$  is the average normal stress in the  $d$ -direction, negative for compression, the softening coefficient,  $\zeta$ , is defined as:

$$\zeta = \frac{5.8}{\sqrt{f'_c}} \frac{1}{\sqrt{1 + 400\varepsilon_r}} \leq \frac{0.9}{\sqrt{1 + 400\varepsilon_r}} \quad (8b)$$

and  $\varepsilon_0$  is the strain at the peak stress of a standard concrete cylinder as determined in Eq. (9) as proposed by Foster and Gilbert (1996):

$$\varepsilon_0 = 0.002 + 0.001 \left( \frac{f'_c - 20}{80} \right) \quad \text{for} \quad 20 \leq f'_c \leq 100 \text{MPa} \quad (9)$$

Furthermore, the softening coefficient,  $\zeta$ , was simplified by assigning the horizontal ties and vertical column intermediate bar strain to be the yielding strain of steel ( $\varepsilon_h = \varepsilon_r = 0.002$ ) and the average diagonal compressive strain in the  $d$ -direction as  $\varepsilon_d = -0.001$ . Hence, the principal tensile strain,  $\varepsilon_r$ , can be calculated using Eq. (7). By assigning the predetermined principal tensile strain,  $\varepsilon_r = -0.001$ , to Eq. (8b) (Hwang and Lee 2002), we have:

$$\zeta \approx \frac{3.35}{\sqrt{f'_c}} \leq 0.52 \quad (10)$$

The previously derived SST model (Hwang and Lee 1999, 2000) for concentric beam-column joints used the ACI 318-95 definition to determine the effective joint width,  $b_j$ . The equations to determine the effective joint width, however, are realized to be empirical. Moreover, for eccentric beam-column joints, the ACI 318-08 equations were

shown to give results that were too conservative (LaFave et al. 2005). Therefore, a more rational derivation on how to determine the effective joint width is proposed based on the failure criteria of SST model.

### DERIVATION OF EFFECTIVE JOINT WIDTH

For concentric joints with sufficient confinement provided by framing beams, the effective joint width,  $b_j$ , can always be taken as the whole column width. The same condition, however, does not apply when eccentricity exists between the center of the column and the longitudinal axis of the beam. The determination of the effective joint width is believed to be sensitive to the geometric constraints provided by the framing beams outside the joint core.

For a typical interior joint with slab, the lower end of the diagonal compressive strut is only confined by the compression zone of the beam web while the upper end is confined by both the beam and the slab (Fig. 4). Hence, the strut area, which is defined as the product of effective joint width and depth of compression strut, is much smaller at the lower end (without the presence of slab), and therefore governs the strength.

The effective joint width defined at the lower end is determined as follows: by firstly taking the failure criteria of SST model to be the crushing of strut area adjacent to the nodal zone; hence, the effective joint width should be determined at the location of concrete crushing, which is taken to be at the resultant of column compressive stress. The joint width is originally assumed to be as wide as beam width,  $b_b$ , at the beam-column interface. As the compression stress of beam spreads out into the joint zone with a certain slope, the effective joint width also gets wider accordingly and ends at the location of crushing. Because most of the specimens were designed following strong column-weak beam philosophy, the column is still in elastic range when the maximum joint capacity is attained. Hence, the resultant of column compression strength can be taken at one-third of depth of column compression zone ( $c_c/3$ ). Meanwhile, the slope of stress dispersion is taken to be 1:2, following the ACI 318-08 recommendation for calculating the bearing strength. Finally, the effective joint width can be written as:

$$b_j = b_b + \sum \min\left(2 \times \frac{c_c}{3}; x_i\right) \quad (11)$$

where  $b_b$  is width of the beam;  $c_c$  is the depth of column compression zone; and  $x_i$  is the perpendicular distance from the longitudinal axis of the beam to the column side. The illustration on the determination of effective joint width is given in Fig. 5(a).

Meanwhile, as described in Fig. 5(b), the depth of the strut is defined to be perpendicular to the concrete compression strut:

$$a_s = c_c \sin\theta + c_b \cos\theta \quad (12)$$

where  $c_b$  is the depth of the beam compression zone.

### EXPERIMENTAL VERIFICATION

A total of 18 specimens of eccentric beam column joints (Table 1) were collected

from Joh et al. (1991), Raffaele and Wight (1995), Teng and Zhou (2004), Shin and LaFave (2004), Goto and Joh (2004), Kusuhabara et al. (2004), and Lee and Ko (2007). All of the selected specimens are those failing in joint shear and having transverse joint reinforcements. Among the assembled databanks, two of them are joints with slabs, another two are T-shape joints, and the other 14 specimens are cruciform-type joints.

The originally derived SST model (Hwang and Lee 1999, 2000) was a complete solution procedure (termed as SST general approach). This solution procedure, however, was tedious and impractical for design practice. Therefore, a simplified version (termed as simplified SST approach) was introduced to serve as design tool for determining the shear strength of discontinuity regions failing in diagonal compressions (Hwang and Lee 2002). The SST model assumes failure at the compression strut adjacent to the nodal zone; hence, the determination of the strut area is very important and sensitive to the end condition provided by the framing members of beam-column joints, especially the depth of beam and column compression zone.

Therefore, in the shear strength prediction for eccentric beam-column joints using the SST model, three tiers of analysis (referred to as Analysis 1, 2, and 3) in terms of the sophistication of SST solution procedure and the accuracy of determination of the depth of the beam and column compression zone (Analysis 1 being the simplest and Analysis 3 being the most sophisticated) are performed. Analysis 1 and Analysis 2 predict the joint strength by using a simplified SST approach (Hwang and Lee 2002) while Analysis 3 uses a general SST approach (Hwang and Lee 1999, 2000).

Meanwhile, in terms of determination of the depth of beam and column compression zone, because all of the collected database for tests of beam-column joints followed the strong column-weak beam design philosophy, most likely the columns are still in the elastic range when the maximum joint capacity is reached. For quick estimation, in determining the column depth of the compression zone,  $c_c$ , Analysis 1, being the simplest and most practical analysis, adopts the empirical formula proposed by Paulay and Priestley (1992):

$$c_c = \left( 0.25 + 0.85 \frac{N}{A_g f'_c} \right) h_c \quad (13)$$

where  $N$  is the column axial load,  $A_g$  is the gross area of the column section, and  $h_c$  is the depth of the column. Furthermore, the beam depth of the compression zone,  $c_b$ , is calculated using Whitney's stress block by simply neglecting the beam compression bars so that:

$$c_b = \frac{A_s f_y}{\beta_1 0.85 f'_c b_b} \quad (14)$$

where  $A_s$  is the area of the tensile beam longitudinal reinforcement bars,  $f_y$  is the yield strength of longitudinal reinforcing bars, and  $\beta_1$  is a factor relating the depth of equivalent rectangular compressive stress block to the neutral axis depth. On the other hand, for Analysis 2 and Analysis 3, the depths of the compression zones of beams and columns are obtained using sectional analysis at the maximum response of joint resistance to get

more accurate values for beam and column depth of compression zones. Due to the bond deterioration, the compression reinforcing bars in the beam may not reach the yielding strength. As suggested by the New Zealand Standards (1995), the maximum attained stress of the compression reinforcing bars should not exceed  $0.7f_y$ . Some joint specimens with eccentricity, however, do not possess adequate joint confinement as required by ACI 318-08 (Table 1). This inadequacy is realized to create more severe bond deterioration compared with that anticipated by New Zealand Standards (NZS 3101:1995); therefore, in this paper, the value of  $0.5f_y$  is used. A more detailed calculation can be found in the Appendix.

Table 2 and Fig. 6 show the strength ratios of the test values to the predicted values using SST analysis. Analysis 1, as the simplest analysis, gives a mean value of 1.60 and a coefficient of variation (COV) of 0.28. Analysis 2 gives 1.34 and 0.22 for the mean value and COV, respectively. Analysis 3, as the most sophisticated analysis, results in 1.29 for the mean value and 0.19 for the COV. It is clearly seen that the accuracy of the analysis tool is getting better as the more refined approach is used. Moreover, it is also observed that the determination of strut area (and correspondingly, the determination of depth of beam and column compression zone) plays a more significant effect compared with the solution procedure itself. For the T-shape joints tested by Lee and Ko (2007), however, the SST predicts greater strengths than what is observed. Although it is an analytical model, several assumptions were still needed. Hence, the applicability of SST analysis in predicting joint shear strength of eccentric beam-column joints should be limited by the experimental parameters, such as: maximum axial load ratio,  $N/A_g f'_c \leq 0.17$ ; eccentricity,  $e/b_c \leq 0.3$ ; and column aspect ratio,  $b_c/h_c \leq 0.17$  (as indicated in Table 1).

## DESIGN IMPLEMENTATION

The ACI 318-08 code provision and ACI 352R-02 design recommendation use the same approach to estimate the shear strength of a joint:

$$V_j = \gamma \sqrt{f'_c} b_j h_c \quad (15)$$

where  $\gamma$  is a constant to account for the confinement effect provided by framing beams. The difference between the two lies with the definition of the effective joint width. LaFave et al. (2005), showed that the ACI 318-05 formulation is too conservative and that the ACI 352R-02 recommendation is also conservative to a lesser extent. Hence, a more rational approach to the derivation of the effective joint width is needed and is described in the following paragraph.

For design purposes, a simpler approach is needed. Equation (11) is further simplified to serve the aforementioned purpose. By taking the axial load ratio  $N/A_g f'_c$  as 0.15 for a common building and substituting this value into Eq. (13), one can obtain:

$$c_c = (0.25 + 0.85 \times 0.15) h_c = 0.377 h_c$$

Further substitution of Eq. (16) into Eq. (11) yields the proposed effective joint width such that:

$$b_j = b_b + \sum \min \left( 2 \times \frac{0.377 h_c}{3}; x_i \right) \quad (17a)$$

$$b_{j, proposed} = b_b + \sum \min \left( \frac{h_c}{4}; x_i \right) \quad (17b)$$

The proposed effective joint width is verified for both eccentric and concentric joints to ensure its applicability. The same database for eccentric joints ([Table 1](#)) is used. Furthermore, a total of 45 specimens of concentrically connected interior and exterior joints failing in shear and having a ratio of provided joint reinforcement to the required joint reinforcement according to ACI 318-08 that is greater than 0.5 are used for verification.

[Table 2](#) and [Fig. 7](#) show the comparison of statistical values of the strength ratio for eccentric joints using code design approach. Comparing the mean values and coefficient of variations, ACI 318-08 gives predictions that are too conservative, while ACI 352R-02 is also conservative to a lesser extent. Meanwhile, the strength estimations using the simple average width,  $(b_b + b_c)/2$  as proposed by LaFave et al. (2005), give the best accuracy and the proposed effective joint width (Eq. (17b)) provides a slightly more conservative prediction compared with the simple average width.

From past experience, a whole column width can be used as effective joint width for concentric joints with typical column dimension. In spite of the simplicity and accuracy of simple average width equation in predicting shear strength of eccentric joints, this equation, which is also adopted in ACI 352R-02 design guideline, may give unnecessary conservatism if applied to concentric joints.

[Tables 3](#) and [4](#), together with [Fig. 8](#) and [9](#), show the experimental verification between the test value and the predicted strength for the concentrically connected interior and exterior joints database, respectively. It is clearly shown that the proposed effective joint width gives a consistent result to that adopted in ACI 318-08 equation for joint shear strength. Although the estimated strengths work well for interior joints, they give unconservative results for exterior joints. With regard to this unconservatism, ACI 318-08 has acknowledged it. It may be interesting to note, however the Architectural Institute of Japan (1999) guideline with regard to this problem. In this guideline, the joint depth used for exterior joints is limited by the horizontally projected length of a 90-degree hook (Otani 1991), rather than the whole column depth as currently adopted in ACI 318-08 (Eq. (15)). If this value (horizontally projected 90-degree hook) is used to replace column depth,  $h_c$ , and the ACI 318-08 effective joint width definition is used, then, based on the available exterior joint database in [Table 4](#), one can obtain the mean value to be 1.19 with COV to be 0.19.

The analysis also shows that ACI 352R-02 method gives a lower estimation about 10% for both interior and exterior joints ([Tables 3](#) and [4](#)) compared with that given by ACI 318-08, because the simple average width,  $(b_b + b_c)/2$ , is used instead of the whole column width. The authors think that this conservatism may not be necessary because the current ACI 318-08 equation for effective joint width has been adopted and worked well since 1989.

It is interesting to note that from the collected databases, most of the specimens have the ratio of column width to beam width ranging from 1.1 to 1.4, except for one

exterior joint specimen (Wo, as shown in Table 4) tested by Lee and Ko (2007), which has a ratio of 2.0. For typical column dimensions, both ACI 318-08 effective joint width and the proposed effective joint width yield the same value, which is the whole column width. For the large ratio of column to beam width (wide column narrow beam joint), however, the proposed effective joint width is smaller than that of ACI 318-08 (Table 4). In light of the available research data, the proposed effective joint width is expected to be able to solve the wide column narrow beam joint problem.

## CONCLUSIONS

A more rational formula associated with SST model for determining effective joint width has been proposed. Several available definitions of effective joint width are studied together with the one proposed to estimate the shear strength of collected database for concentric and eccentric joints.

The strength predictions of an SST model using a proposed effective joint width show good agreement with the experimental values of 18 collected specimens failed in shear under limited test parameters.

The proposed effective joint width is further simplified to fit the design equation adopted in ACI 318-08. Along with several definitions of effective joint width, the proposed effective joint width is verified with a collected database for concentric and eccentric beam-column joints. The analysis shows that the proposed effective joint width works well for both concentric and eccentric joints. The proposed effective joint width can also address the issue of wide column narrow beam joint.

## ACKNOWLEDGMENTS

The authors would like to thank the National Science Council and the National Center for Research on Earthquake Engineering (NCREE) in Taiwan (ROC) for their support.

## NOTATION

|                  |   |                                                                                   |
|------------------|---|-----------------------------------------------------------------------------------|
| $a_s$            | = | depth of compression strut                                                        |
| $A_g$            | = | gross area of column section                                                      |
| $A_s$            | = | area of tensile beam longitudinal reinforcement                                   |
| $A_{th}, A_{tv}$ | = | area of horizontal ties and column intermediate bars, respectively                |
| $b_b$            | = | width of beam                                                                     |
| $b_c$            | = | width of column                                                                   |
| $b_j$            | = | effective joint width                                                             |
| $c_b$            | = | depth of beam compression zone                                                    |
| $c_c$            | = | depth of column compression zone                                                  |
| $C_d$            | = | diagonal compressive strength                                                     |
| $D$              | = | compressive force in diagonal strut                                               |
| $e$              | = | eccentricity between the beam centerline to column centerline                     |
| $E_s$            | = | steel modulus of elasticity                                                       |
| $f'_c$           | = | compressive strength of standard concrete cylinder                                |
| $f_y$            | = | yield strength of beam longitudinal bar                                           |
| $f_{yh}, f_{yv}$ | = | yield strength of the horizontal ties and column intermediate bars, respectively  |
| $F_h, F_v$       | = | horizontal and vertical tie force, respectively                                   |
| $F_{yh}, F_{yv}$ | = | yielding forces of the horizontal ties and column intermediate bars, respectively |

|                                |   |                                                                                                             |
|--------------------------------|---|-------------------------------------------------------------------------------------------------------------|
| $\bar{F}_h, \bar{F}_v$         | = | balanced amount of horizontal tie force and vertical column intermediate bar force, respectively            |
| $h_c$                          | = | depth of column                                                                                             |
| $K$                            | = | strut-and-tie index                                                                                         |
| $K_h, K_v$                     | = | horizontal and vertical tie index, respectively                                                             |
| $\bar{K}_h, \bar{K}_v$         | = | horizontal and vertical tie index with sufficient horizontal or vertical reinforcement, respectively        |
| $\ell_h$                       | = | distance between extreme longitudinal reinforcement in column                                               |
| $\ell_v$                       | = | distance between extreme longitudinal reinforcement in beam                                                 |
| $N$                            | = | column axial load                                                                                           |
| $x$                            | = | perpendicular distance from longitudinal axis of beam to column side                                        |
| $V_j$                          | = | joint shear strength                                                                                        |
| $\beta_1$                      | = | factor relating depth of equivalent rectangular compressive stress block to neutral axis                    |
| $\varepsilon_o$                | = | strain at peak stress of standard concrete cylinder                                                         |
| $\varepsilon_d, \varepsilon_r$ | = | average normal strains in d- and r- directions, respectively                                                |
| $\varepsilon_v, \varepsilon_h$ | = | average normal strains in the v- and h- directions, respectively                                            |
| $\gamma$                       | = | constant to account for confinement effect provided by framing beam                                         |
| $\gamma_h$                     | = | fraction of horizontal shear transferred by horizontal tie with absence of vertical column intermediate bar |
| $\gamma_v$                     | = | fraction of vertical shear transferred by vertical column bar with absence of horizontal ties               |
| $\theta$                       | = | angle of strut inclination to the horizontal axis                                                           |
| $\sigma_d$                     | = | average normal stress in d-direction (negative for compression)                                             |
| $\zeta$                        | = | softening coefficient                                                                                       |

## REFERENCES

- ACI-ASCE Committee 352, 2002, "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-02)," American Concrete Institute, Farmington Hills, MI, 37 pp.
- ACI Committee 318, 2005, "Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05)," American Concrete Institute, Farmington Hills, MI, 430 pp.
- ACI Committee 318, 2008, "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary," American Concrete Institute, Farmington Hills, MI, 465 pp.
- Alameddine, F. F., 1990, "Seismic Design Recommendation for High Strength Concrete Beam-to-Column Connections," PhD thesis, University of Arizona, 257 pp.
- Architectural Institute of Japan, 1999, "Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings based on Inelastic Displacement Concept," Architectural Institute of Japan, Tokyo, 440 pp. (in Japanese)
- Au, F. T. K.; Huang, K.; and Pam, H. J., 2005, "Diagonally-Reinforced Beam-Column Joints Reinforced under Cyclic Loading," *Proceedings of The Institution of Civil Engineers*, Paper 13310, pp. 21-40.
- Birss, G. R., 1978, "The Elastic Behaviour of Earthquake Resistant Reinforced Concrete Interior Beam-Column Joints," *Research Report*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, 105 pp.
- Comité Euro-International du Béton (CEB)-Fédération Internationale de la Précontrainte (FIP), 1993, "Model Code 1990, 1993, (MC90)," Thomas Telford, London, 437 pp.
- Durrani, A. J., and Wight, J. K., 1985, "Behavior of Interior Beam-to-Column Connections under Earthquake-Type Loading," *ACI Structural Journal*, V. 82, No. 3, May-June, pp. 343-349.
- Ehsani, M. R., and Wight, J. K., 1985, "Exterior Reinforced Concrete Beam-to-Column Connections Subjected to Earthquake-Type Loading," *ACI Structural Journal*, V. 82, No. 5, Sept.-Oct., pp. 492-499.

Ehsani, M. R.; Moussa, A. E.; and Vallenilla, C. R., 1987, "Comparison of Inelastic Behavior of Reinforced Ordinary and High Strength Concrete Frames," *ACI Structural Journal*, V. 84, No. 2, Mar.-Apr., pp. 161-169.

Fenwick, R. C., and Irvine, H. M., 1977, "Reinforced Concrete Beam-Column Joints for Seismic Loading," *School of Engineering Report No. 142*, Department of Civil Engineering, University of Auckland, Auckland, New Zealand, 50 pp.

Foster, S. J., and Gilbert, R. I., 1996, "The Design of Nonflexural Members with Normal and High-Strength Concretes," *ACI Structural Journal*, V. 93, No. 1, Jan.-Feb., pp. 3-10.

Fujii, S., and Morita, S., 1991, "Comparison between Interior and Exterior RC Beam-Column Joint Behavior," *Design of Beam-Column Joints for Seismic Resistance*, SP-123, J. O. Jirsa, ed., American Concrete Institute, Farmington Hills, MI, pp. 145-165.

Goto, Y., and Joh, O., 2004, "Shear Resistance of RC Interior Eccentric Beam-Column Joints," *Proceedings of the 13th World Conference on Earthquake Engineering*, Paper No. 649, Vancouver, BC, Canada, 13 pp.

Goto, Y., and Joh, O., 2003, "Experimental Study on Shear Resistance of RC Interior Eccentric Beam-Column Joints," *Proceedings of the Ninth East Asia-Pacific Conference on Structural Engineering and Construction*, Bali, Indonesia, pp. 170-176.

Hsu, T. T. C., 1996, "Toward a Unified Nomenclature for Reinforced Concrete Theory," *Journal of Structural Engineering*, ASCE, V. 122, No. 3, pp. 275-283.

Hwang, S. J., and Lee, H. J., 1999, "Analytical Model for Predicting Shear Strengths of Exterior Reinforced Concrete Beam-Column Joints for Seismic Resistance," *ACI Structural Journal*, V. 96, No. 5, Sept.-Oct., pp. 846-858.

Hwang, S. J., and Lee, H. J., 2000, "Analytical Model for Predicting Shear Strengths of Interior Reinforced Concrete Beam-Column Joints for Seismic Resistance," *ACI Structural Journal*, V. 97, No. 1, Jan.-Feb., pp. 35-44.

Hwang, S. J., and Lee, H. J., 2002, "Strength Prediction for Discontinuity Regions by Softened Strut-and-Tie Model," *Journal of Structural Engineering*, ASCE, V. 128, No. 12, pp. 1519-1526.

Joh, O.; Goto, Y.; and Shibata, T., 1991, "Behavior of Reinforced Beam-Column Joints with Eccentricity," *Design of Beam-Column Joints for Seismic Resistance*, SP-123, J. O. Jirsa, ed., American Concrete Institute, Farmington Hills, MI, pp. 317-357.

Kanada, K., Kondo, G., Fujii, S., and Morita, S. 1984. "Relation between Beam Bar Anchorage and Shear Resistance at Exterior Beam-Column Joints," *Transactions of the Japan Concrete Institute*, V. 6, pp. 433-440.

Kusuhara, T.; Takimoto, H.; and Tanaka, S., 2004, "Mechanical Behavior of Reinforced Concrete Beam-Column Assemblages with Eccentricity," *Proceedings of the 13th World Conference on Earthquake Engineering*, Paper No. 4, Vancouver, British Columbia, Canada, 10 pp.

LaFave, J. M.; Bonacci, J. F.; Burak, B.; and Shin, M., 2005, "Eccentric Beam-Column Connections," *Concrete International*, V. 27, No. 9, Sept., pp. 58-62.

Lee, H. J., and Ko, J. W., 2007, "Eccentric Reinforced Concrete Beam-Column Connections Subjected to Cyclic Loading in Principal Directions," *ACI Structural Journal*, V. 104, No. 4, July-Aug., pp. 459-467.

Leon, R. T., 1990, "Shear Strength and Hysteretic Behavior of Interior Beam-Column Joints," *ACI Structural Journal*, V. 87, No. 1, Jan.-Feb., pp. 3-11.

Meinheit, D. F., and Jirsa, J. O., 1977, "The Shear Strength of Reinforced Concrete Beam-Column Joints," *CESRL Report No. 77-1*, Department of Civil Engineering, University of Texas at Austin, Austin, TX, 271 pp.

New Zealand Standards, 1995, "Concrete Structures Standard, Part 2—Commentary on The Design of Concrete Structures," NZS 3101: Part 2, 264 pp.

Otani, S., 1991, "The Architectural Institute of Japan (AIJ) Proposal of Ultimate Strength Design

Requirements for RC Buildings with Emphasis on Beam-Column Joints,” *Design of Beam-Column Joints for Seismic Resistance*, SP-123, J. O. Jirsa, ed., American Concrete Institute, Farmington Hills, MI, pp. 125-144.

Park, R.; Gaerty, L.; and Stevenson, E. C., 1981, “Tests on an Interior Reinforced Concrete Beam-Column Joint,” *Bulletin of the New Zealand National Society for Earthquake Engineering*, V. 14, No. 2, pp. 81-92.

Park, R., and Milburn, J. R., 1983, “Comparison of Recent New Zealand and United States Seismic Design Provisions for Reinforced Concrete Beam-Column Joints and Tests Results from Four Units Designed According to the New Zealand Code,” *Bulletin of the New Zealand National Society for Earthquake Engineering*, V. 16, No. 1, pp. 3-24.

Paulay, T., and Priestley, M. J. N., 1992, *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley and Sons, 744 pp.

Raffaella, G. S., and Wight, J. K., 1995, “Reinforced Concrete Eccentric Beam-Column Connections Subjected to Earthquake-Type Loading,” *ACI Structural Journal*, V. 92, No. 1, Jan.-Feb., pp. 45-55.

Shin, M., and LaFave, J. M., 2004, “Seismic Performance of Reinforced Concrete Eccentric Beam-Column Connections with Floor Slabs,” *ACI Structural Journal*, V. 101, No. 3, May-June, pp. 403-412.

Teng, S., and Zhou, H., 2003, “Eccentric Reinforced Concrete Beam-Column Joints Subjected to Cyclic Loading,” *ACI Structural Journal*, V. 100, No. 2, Mar.-Apr., pp. 139-148.

Zhang, L. X. B., and Hsu, T. T. C., 1993, “Behavior and Analysis of 100 MPa Concrete Membrane Elements,” *Journal of Structural Engineering*, ASCE, V. 119, No. 12, pp. 3590-3610.

Table 1—Specimen parameters for eccentric beam-column joints

| No. | Specimens                     |                   | $f'_c$ ,<br>MPa<br>(psi) | $\frac{N}{A_g f'_c}$ | Column<br>(b x h),<br>mm (in.) | Beam<br>(b x h),<br>mm (in.) | e,<br>mm<br>(in.) | $\frac{A_{sh\_prov.}}{A_{sh\_req.}}$ |
|-----|-------------------------------|-------------------|--------------------------|----------------------|--------------------------------|------------------------------|-------------------|--------------------------------------|
| 1   | Joh et al. (1991)             | JXo-B5            | 23.0<br>(3336)           | 0.13                 | 300×300<br>(12×12)             | 150×350<br>(6×14)            | 75 (3)            | 0.32                                 |
| 2   | Raffaelle and<br>Wight (1995) | S1                | 28.6<br>(4148)           | 0.02                 | 350×350<br>(14×14)             | 254×381<br>(10×15)           | 50.8<br>(2.0)     | 1.37                                 |
| 3   |                               | S2                | 26.8<br>(3887)           | 0.03                 | 350×350<br>(14×14)             | 178×381<br>(7×15)            | 88.9<br>(3.5)     | 1.47                                 |
| 4   |                               | S3                | 37.7<br>(5468)           | 0.02                 | 350×350<br>(14×14)             | 190.5×381<br>(7.5×15)        | 82.5<br>(3.25)    | 1.04                                 |
| 5   |                               | S4                | 19.3<br>(2799)           | 0.04                 | 350×350<br>(14×14)             | 190.5×600<br>(7.5×22)        | 82.5<br>(3.25)    | 2.03                                 |
| 6   | Teng and Zhou<br>(2003)       | S2                | 34.0<br>(4931)           | 0.11                 | 400×300<br>(16×12)             | 200×400<br>(8×16)            | 50<br>(2)         | 0.91                                 |
| 7   |                               | S3                | 35.0<br>(5076)           | 0.10                 | 400×300<br>(16×12)             | 200×400<br>(8×16)            | 100<br>(4)        | 0.89                                 |
| 8   |                               | S5                | 39.0<br>(5656)           | 0.12                 | 400×200<br>(16×8)              | 200×400<br>(8×16)            | 50<br>(2)         | 0.84                                 |
| 9   |                               | S6                | 38.0<br>(5511)           | 0.12                 | 400×200<br>(16×8)              | 200×400<br>(8×16)            | 100<br>(4)        | 0.86                                 |
| 10  | Shin and LaFave<br>(2004)     | S1*               | 29.9<br>(4337)           | 0.00                 | 457×330<br>(18×13)             | 279×406<br>(11×16)           | 89<br>(3.5)       | 1.03                                 |
| 11  |                               | S2*               | 36.2<br>(5250)           | 0.00                 | 457×330<br>(18×13)             | 279×406<br>(11×16)           | 140<br>(5.5)      | 0.85                                 |
| 12  | Goto and Joh<br>(2004)        | UM-60             | 24.6<br>(3568)           | 0.17                 | 450×300<br>(18×12)             | 200×350<br>(8×14)            | 60<br>(2.4)       | 0.39                                 |
| 13  |                               | UM-125            | 25.2<br>(3655)           | 0.17                 | 450×300<br>(18×12)             | 200×350<br>(8×14)            | 125<br>(5)        | 0.38                                 |
| 14  |                               | UU-125            | 25.4<br>(3684)           | 0.17                 | 450×300<br>(18×12)             | 200×350<br>(8×14)            | 125<br>(5)        | 0.75                                 |
| 15  | Kusuhara et al.<br>(2004)     | JE-55             | 27.0<br>(3916)           | 0.00                 | 320×280<br>(12.6×11)           | 180×300<br>(7×12)            | 55<br>(2.2)       | 0.19                                 |
| 16  |                               | JE-55S            | 27.0<br>(3916)           | 0.00                 | 320×280<br>(12.6×11)           | 180×300<br>(7×12)            | 55<br>(2.2)       | 0.88                                 |
| 17  | Lee and Ko<br>(2007)          | W75 <sup>†</sup>  | 30.4<br>(4409)           | 0.10                 | 600×400<br>(24×16)             | 300×450<br>(12×18)           | 75<br>(3)         | 1.40                                 |
| 18  |                               | W150 <sup>†</sup> | 29.1<br>(4221)           | 0.10                 | 600×400<br>(24×16)             | 300×450<br>(12×18)           | 150<br>(6)        | 1.40                                 |

\*Specimens with slabs.

†Specimens in T-shape form (exterior joint).

Table 2—Statistical value of shear strength ratios between experimental and predicted values

| Specimens |                            |        | SST Analysis              |                               |                               |                               | Code formula                   |                                |                               |                                 |
|-----------|----------------------------|--------|---------------------------|-------------------------------|-------------------------------|-------------------------------|--------------------------------|--------------------------------|-------------------------------|---------------------------------|
| No.       | Author                     | ID     | $V_{test}^*$<br>kN (kips) | $\frac{V_{test}}{V_{SST\_1}}$ | $\frac{V_{test}}{V_{SST\_1}}$ | $\frac{V_{test}}{V_{SST\_3}}$ | $\frac{V_{test}}{V_{ACI-318}}$ | $\frac{V_{test}}{V_{ACI-352}}$ | $\frac{V_{test}}{V_{LaFave}}$ | $\frac{V_{test}}{V_{proposed}}$ |
| 1         | Joh et al. (1991)          | JXo-B5 | 294 (665)                 | 1.42                          | 1.02                          | 0.99                          | 1.36                           | 1.05                           | 0.91                          | 0.91                            |
| 2         |                            | S1     | 605 (136)                 | 1.50                          | 1.35                          | 1.38                          | 1.25                           | 1.04                           | 1.04                          | 0.93                            |
| 3         | Raffaelle and Wight (1995) | S2     | 421 (952)                 | 1.45                          | 1.18                          | 1.17                          | 1.29                           | 0.99                           | 0.86                          | 0.86                            |
| 4         |                            | S3     | 472(1061)                 | 1.24                          | 1.03                          | 1.07                          | 1.13                           | 0.89                           | 0.79                          | 0.77                            |
| 5         |                            | S4     | 412 (93)                  | 1.92                          | 1.78                          | 1.65                          | 1.39                           | 1.08                           | 0.97                          | 0.94                            |
| 6         | Teng and Zhou (2003)       | S2     | 758 (170)                 | 2.01                          | 1.61                          | 1.51                          | 1.45                           | 1.45                           | 1.45                          | 1.33                            |
| 7         |                            | S3     | 743 (167)                 | 2.25                          | 1.79                          | 1.67                          | 2.09                           | 1.71                           | 1.40                          | 1.52                            |
| 8         |                            | S5     | 434 (98)                  | 1.64                          | 1.11                          | 1.04                          | 1.16                           | 1.16                           | 1.16                          | 1.16                            |
| 9         |                            | S6     | 437 (98)                  | 1.94                          | 1.58                          | 1.46                          | 1.77                           | 1.54                           | 1.18                          | 1.42                            |
| 10        | Shin and LaFave (2004)     | S1*    | 645 (145)                 | 1.96                          | 1.35                          | 1.35                          | 1.28                           | 1.09                           | 0.97                          | 0.99                            |
| 11        |                            | S2*    | 649 (146)                 | 2.15                          | 1.42                          | 1.42                          | 1.84                           | 1.44                           | 1.03                          | 1.26                            |
| 12        | Goto and Joh (2004)        | UM-60  | 779 (175)                 | 1.14                          | 1.35                          | 1.33                          | 1.59                           | 1.81                           | 1.61                          | 1.54                            |
| 13        |                            | UM-125 | 666 (149)                 | 1.18                          | 1.36                          | 1.35                          | 2.21                           | 1.80                           | 1.36                          | 1.61                            |
| 14        |                            | UU-125 | 632 (142)                 | 1.12                          | 1.28                          | 1.27                          | 2.09                           | 1.71                           | 1.29                          | 1.52                            |
| 15        | Kusuhara et al. (2004)     | JE-55  | 519 (116)                 | 2.05                          | 1.60                          | 1.47                          | 1.70                           | 1.50                           | 1.43                          | 1.35                            |
| 16        |                            | JE-55S | 516 (116)                 | 1.96                          | 1.54                          | 1.47                          | 1.69                           | 1.50                           | 1.42                          | 1.34                            |

Table 2—Statistical value of shear strength ratios between experimental and predicted values (con't)

| Specimens |                      |        | SST Analysis                  |                               |                               |                                | Code formula                   |                               |                                 |  |
|-----------|----------------------|--------|-------------------------------|-------------------------------|-------------------------------|--------------------------------|--------------------------------|-------------------------------|---------------------------------|--|
| No.       | Author               | ID     | $\frac{V_{test}}{V_{SST\_1}}$ | $\frac{V_{test}}{V_{SST\_1}}$ | $\frac{V_{test}}{V_{SST\_3}}$ | $\frac{V_{test}}{V_{ACI-318}}$ | $\frac{V_{test}}{V_{ACI-352}}$ | $\frac{V_{test}}{V_{LaFave}}$ | $\frac{V_{test}}{V_{proposed}}$ |  |
| 17        | Lee and Ko<br>(2007) | W75**  | 0.85                          | 0.78                          | 0.80                          | 0.79                           | 0.79                           | 0.79                          | 0.75                            |  |
| 18        |                      | W150** | 1.00                          | 0.92                          | 0.94                          | 1.14                           | 0.95                           | 0.76                          | 0.86                            |  |
| Mean      |                      |        | 1.60                          | 1.34                          | 1.29                          | 1.51                           | 1.30                           | 1.13                          | 1.17                            |  |
| COV       |                      |        | 0.28                          | 0.22                          | 0.19                          | 0.26                           | 0.26                           | 0.24                          | 0.25                            |  |

\*Specimens with slabs.  
\*\*Specimens in T-shape form (exterior joint).

**Table 3—Shear strength ratios between experimental and predicted values for concentrically connected interior joints**

| Specimens |                           |       | $\frac{V_{test}}{V_{ACI-318}}$ | $\frac{V_{test}}{V_{ACI-352}}$ | $\frac{V_{test}}{V_{proposed}}$ |
|-----------|---------------------------|-------|--------------------------------|--------------------------------|---------------------------------|
| No.       | Authors                   | ID    |                                |                                |                                 |
| 1         | Meinheit and Jirsa (1977) | SI    | 1.13                           | 1.22                           | 1.13                            |
| 2         |                           | SIII  | 1.26                           | 1.37                           | 1.26                            |
| 3         |                           | SIV   | 1.28                           | 1.36                           | 1.28                            |
| 4         |                           | SVIII | 1.19                           | 1.29                           | 1.19                            |
| 5         |                           | SIX   | 1.52                           | 1.65                           | 1.52                            |
| 6         |                           | SX    | 1.44                           | 1.56                           | 1.44                            |
| 7         |                           | SXII  | 1.74                           | 1.89                           | 1.74                            |
| 8         |                           | SXIII | 1.28                           | 1.39                           | 1.28                            |
| 9         |                           | SXIV  | 1.42                           | 1.50                           | 1.42                            |
| 10        | Fenwick and Irvine (1977) | 1     | 0.98                           | 1.09                           | 0.98                            |
| 11        |                           | 3     | 1.09                           | 1.21                           | 1.09                            |
| 12        | Birss (1978)              | B1    | 0.88                           | 0.99                           | 0.88                            |
| 13        | Park et al. (1981)        | 1     | 1.06                           | 1.21                           | 1.06                            |
| 14        | Park and Milburn (1983)   | 1     | 1.01                           | 1.15                           | 1.01                            |
| 15        | Durrani and Wight (1985)  | X1    | 0.88                           | 0.99                           | 0.88                            |
| 16        |                           | X2    | 0.90                           | 1.01                           | 0.90                            |
| 17        |                           | X3    | 0.69                           | 0.78                           | 0.69                            |
| 18        | Leon (1990)               | BCJ2  | 0.85                           | 0.94                           | 0.85                            |
| 19        |                           | BCJ3  | 0.94                           | 1.04                           | 0.94                            |
| 20        | Fujii and Morita (1991)   | A4    | 1.37                           | 1.59                           | 1.37                            |
| 21        | Au et al. (2005)          | H-0.0 | 1.09                           | 1.19                           | 1.09                            |
| 22        |                           | H-0.3 | 0.98                           | 1.07                           | 0.98                            |
| Mean      |                           |       | 1.14                           | 1.25                           | 1.14                            |
| COV       |                           |       | 0.23                           | 0.21                           | 0.23                            |

**Table 4—Shear strength ratios between experimental and predicted values for concentrically connected exterior joints**

| Specimens |                         |      | $\frac{V_{test}}{V_{ACI-318}}$ | $\frac{V_{test}}{V_{ACI-352}}$ | $\frac{V_{test}}{V_{proposed}}$ |
|-----------|-------------------------|------|--------------------------------|--------------------------------|---------------------------------|
| No.       | Authors                 | ID   |                                |                                |                                 |
| 1         | Kanada et al. (1984)    | U41L | 0.73                           | 0.78                           | 0.73                            |
| 2         |                         | U42L | 0.68                           | 0.73                           | 0.68                            |
| 3         | Ehsani and Wight (1985) | 1B   | 1.07                           | 1.14                           | 1.07                            |
| 4         |                         | 3B   | 1.03                           | 1.10                           | 1.03                            |
| 5         |                         | 4B   | 1.06                           | 1.14                           | 1.06                            |
| 6         |                         | 5B   | 1.00                           | 1.06                           | 1.00                            |
| 7         |                         | 6B   | 0.62                           | 0.66                           | 0.62                            |
| 8         | Ehsani et al. (1987)    | 2    | 0.64                           | 0.68                           | 0.64                            |
| 9         |                         | 3    | 0.75                           | 0.80                           | 0.75                            |
| 10        |                         | 4    | 0.69                           | 0.74                           | 0.69                            |
| 11        | Alameddine (1990)       | LL8  | 0.91                           | 0.96                           | 0.91                            |
| 12        |                         | LH8  | 0.88                           | 0.93                           | 0.88                            |
| 13        |                         | HL8  | 1.04                           | 1.10                           | 1.04                            |
| 14        |                         | HH8  | 1.04                           | 1.10                           | 1.04                            |
| 15        |                         | LL11 | 0.70                           | 0.74                           | 0.70                            |
| 16        |                         | LH11 | 0.86                           | 0.90                           | 0.86                            |
| 17        |                         | HL11 | 0.89                           | 0.94                           | 0.89                            |
| 18        |                         | HH11 | 0.94                           | 0.99                           | 0.94                            |
| 19        |                         | LL14 | 0.72                           | 0.76                           | 0.72                            |
| 20        |                         | LH14 | 0.73                           | 0.77                           | 0.73                            |
| 21        |                         | HH14 | 0.85                           | 0.90                           | 0.85                            |
| 22        | Fujii and Morita (1991) | B4   | 1.08                           | 1.25                           | 1.08                            |
| 23        | Lee and Ko (2007)       | Wo   | 0.60                           | 0.80                           | 0.72                            |
| Mean      |                         |      | 0.86                           | 0.93                           | 0.87                            |
| COV       |                         |      | 0.19                           | 0.19                           | 0.18                            |

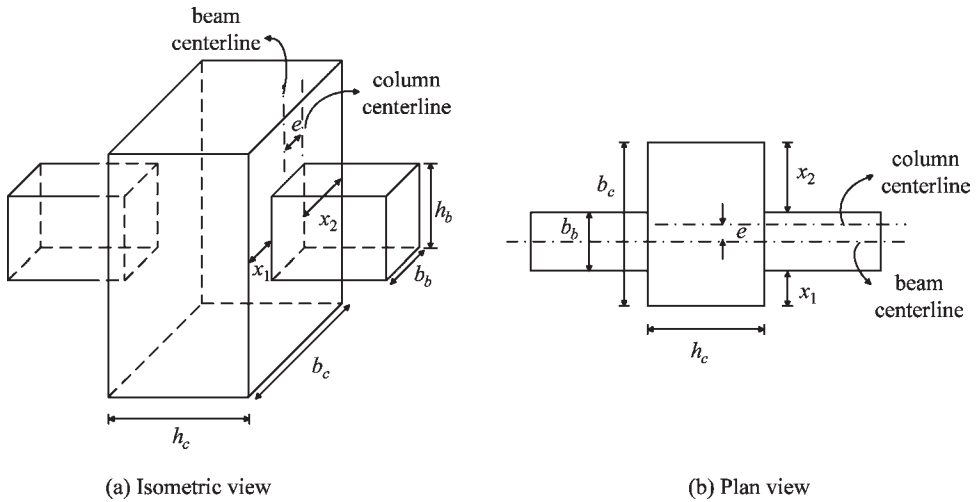
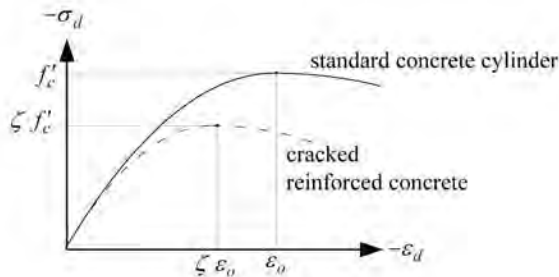
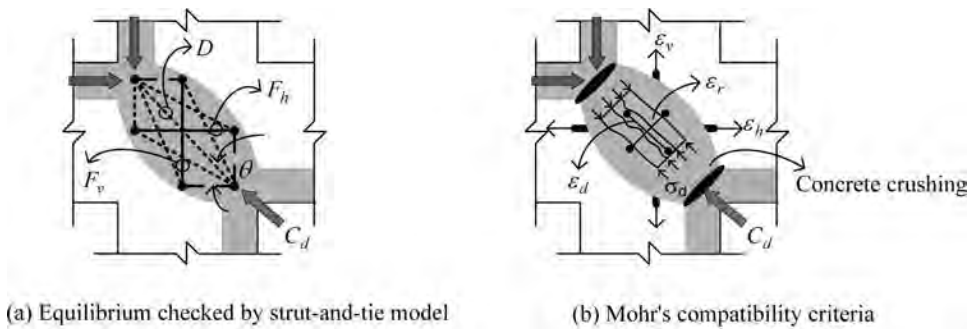


Fig. 1—Typical eccentric reinforced concrete beam-column joint.



(c) Constitutive law for softened reinforced concrete

Fig. 2—Softened strut-and-tie model (Hwang and Lee 1999, 2000).

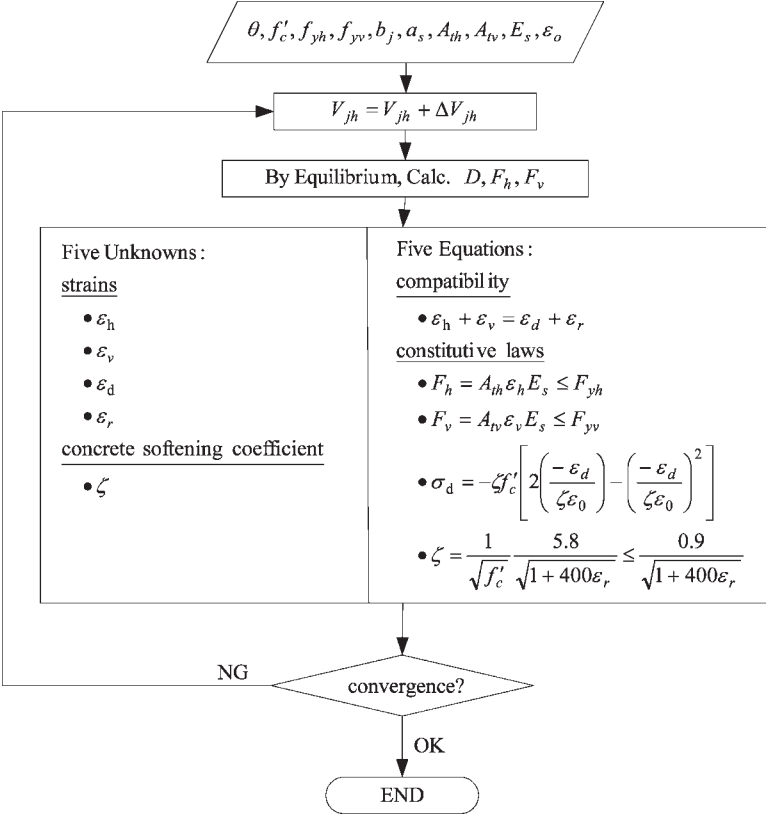


Fig. 3—Solution algorithm for general SST approach (Hwang and Lee 1999, 2000).

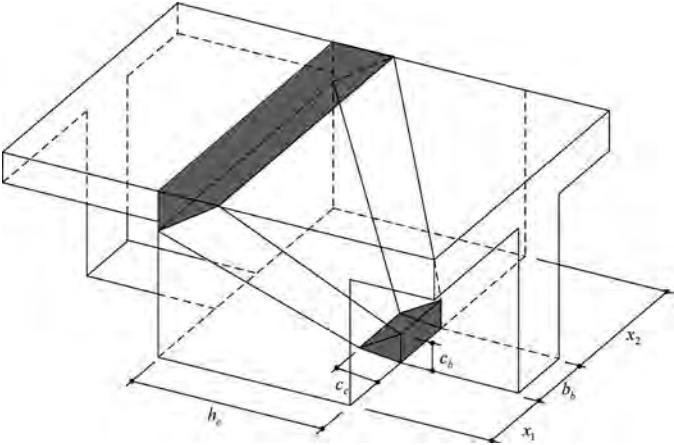


Fig. 4—Schematic view of diagonal compression strut within joint.

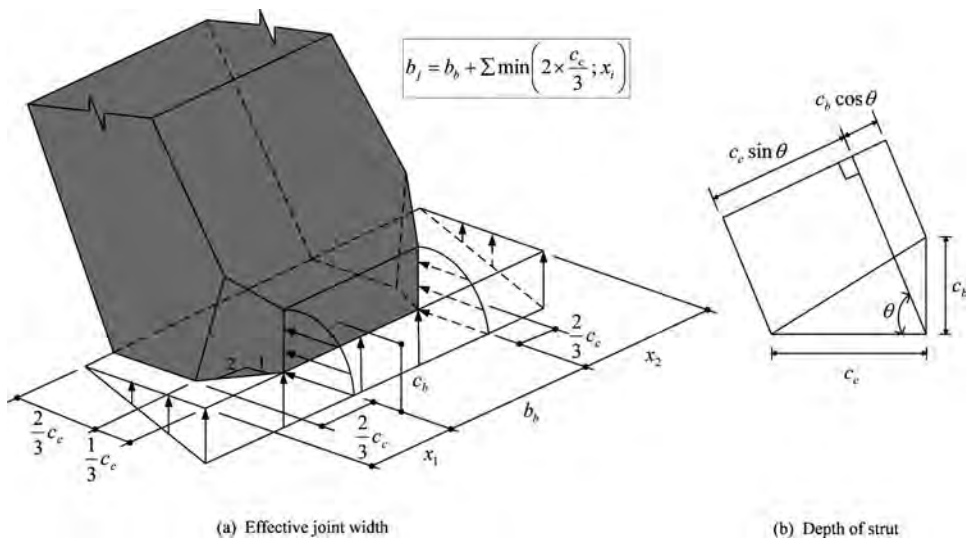


Fig. 5—Illustration on determination of the strut area.

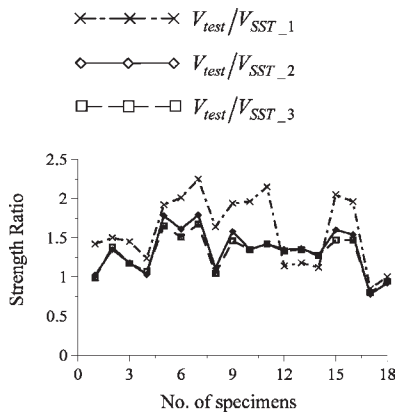


Fig. 6—Strength ratio versus number of specimens for eccentric joints calculated using SST analysis.

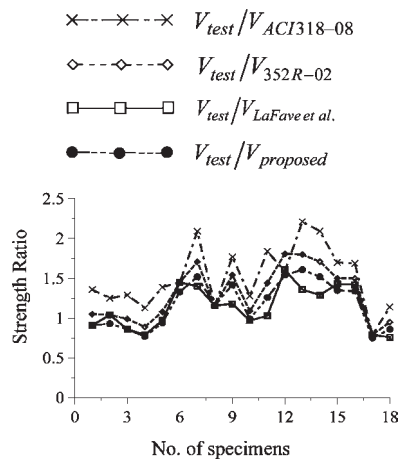


Fig. 7—Strength ratio versus number of specimens for eccentric joints calculated using ACI 318-08 Code equation.

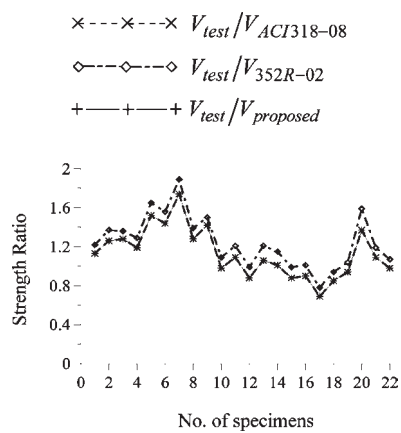


Fig. 8—Strength ratio versus number of specimens for concentrically connected interior joints.

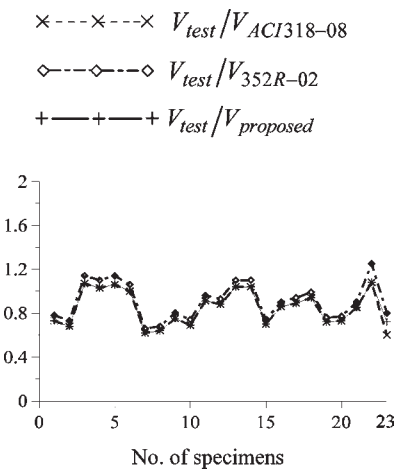


Fig. 9—Strength ratio versus number of specimens for concentrically connected exterior joints.

APPENDIX: EXAMPLE SHOWING ANALYSIS USING SST MODEL

Table A1 shows the calculation example for an interior joint shear strength of Specimen UM-6o tested by Goto and Joh (2004) (Fig. A1). The total length of the column is 1750 mm (68.9 in.). The lengths of the beams on either side of the joint are 1350 mm (53.1 in.).

Table A1—Calculation example using SST approach

|                                               | Analysis 1                                                                                                                                                                                                                                                                     | Analysis 2                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       | Analysis 3 |
|-----------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------|
| Input                                         |                                                                                                                                                                                                                                                                                | $\theta = \tan^{-1} \frac{\ell_v}{\ell_h} = \tan^{-1} \frac{290}{240} = 50.4^\circ$<br><br>$f'_c = 24.6 \text{ MPa}; f_y = 384 \text{ MPa}; f_{ph} = 355 \text{ MPa}$<br>$A_{th} = 3 \times 1 \times 28.3 \text{ mm}^2 = 85 \text{ mm}^2$<br>$A_{wv} = 1 \times 201 \text{ mm}^2 = 201 \text{ mm}^2$<br>$F_{yph} = 85 \times 355 = 30.1 \text{ kN}$<br>$F_w = 201 \times 384 = 77.2 \text{ kN}$                                                                                                                                                                                                                  |            |
| Depth of compression zones of beam and column | $c_c = \left( 0.25 + 0.85 \frac{N}{A_g f'_c} \right) h_c$<br>$= (0.25 + 0.85 \times 0.17) 300$<br>$= 118 \text{ mm}$<br><br>$c_b = \frac{A_s f_y}{0.85 f'_c \beta_1 b_b}$<br>$= \frac{3 \times 387 \times 697}{0.85 \times 24.6 \times 0.85 \times 200}$<br>$= 228 \text{ mm}$ | Joint maximum response : <ul style="list-style-type: none"><li>Maximum story shear force = 160 kN</li><li>Corresponding maximum shear at beam <math>\frac{160 \times 1750/2}{1350 + 300/2} = 93.3 \text{ kN}</math></li><li>The maximum bending moments occurred at joint interface for column and beam respectively are :<br/><math>M_{col} = 160 \times (875 - 475)/1000 = 112 \text{ kN} - \text{m}</math><br/><math>M_{beam} = 93.3 \times 1350 = 126 \text{ kN} - \text{m}</math></li><li>Through sectional analysis,<br/><math>c_c = 139 \text{ mm}</math><br/><math>c_b = 131 \text{ mm}</math></li></ul> |            |
| Depth of strut                                | $a_s = c_c \sin \theta + c_b \cos \theta = 236 \text{ mm}$                                                                                                                                                                                                                     | $a_s = c_c \sin \theta + c_b \cos \theta = 191 \text{ mm}$                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       |            |

Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

Table A1—Calculation example using SST approach (con't)

|                       | Analysis 1                                                                                                                                                                                                          | Analysis 2                                                                                                                                                                                       | Analysis 3                                                                                                                                    |
|-----------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------|
| Effective joint width | $b_j = b_b + \sum \min \left( 2 \times \frac{C_c}{3}; x_i \right)$ $200 + \min \left( 2 \times \frac{118}{3}; 185 \right) + \min \left( 2 \times \frac{118}{3}; 65 \right)$ $= 344 \text{ mm}$                      | $b_j = b_b + \sum \min \left( 2 \times \frac{C_c}{3}; x_i \right)$ $= 200 + \min \left( 2 \times \frac{139}{3}; 185 \right) + \min \left( 2 \times \frac{139}{3}; 65 \right)$ $= 358 \text{ mm}$ |                                                                                                                                               |
| Force distribution    | $\gamma_h = (2 \tan \theta - 1)/3 = 0.47$ $\gamma_v = (2 \cot \theta - 1)/3 = 0.22$                                                                                                                                 |                                                                                                                                                                                                  | Through solution procedure :<br>$V_h = 584.6 \text{ kN}$<br>$F_h = 31.0 \text{ kN}$<br>$F_v = 77.2 \text{ kN}$                                |
| Strain compatibility  | Assume $\epsilon_h = \epsilon_v = 0.002$ and $\epsilon_h = -0.001$<br>$\epsilon_r = \epsilon_v + \epsilon_h - \epsilon_d = 0.002$                                                                                   |                                                                                                                                                                                                  | $\epsilon_h = \frac{F_h}{A_h E_s} = 0.0018$<br>$\epsilon_v = \frac{F_v}{A_v E_s} = 0.0019$<br>$\epsilon_d = -0.0011$<br>$\epsilon_r = 0.0048$ |
| Softening coefficient | $\zeta = \frac{5.8}{\sqrt{f'_c}} \frac{1}{\sqrt{1 + 400 \epsilon_r}} \leq \frac{0.9}{\sqrt{1 + 400 \epsilon_r}}$ $\zeta = \frac{3.35}{\sqrt{f'_c}} = \frac{3.35}{\sqrt{24.6}} = 0.68 > 0.52$<br>take $\zeta = 0.52$ |                                                                                                                                                                                                  | Through solution procedure:<br>$\zeta = 0.68 > 0.53$ ;<br>take $\zeta = 0.53$                                                                 |

Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

Table A1—Calculation example using SST approach (con't)

|                                         | Analysis 1                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       | Analysis 2                                                                                                                                                                                                 | Analysis 3                         |
|-----------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------|
| Strut-and-tie index                     | $\overline{K}_h = 1/(1 - 0.2\gamma_h - 0.2\gamma_h^2) = 1.16$<br>$\overline{K}_v = 1/(1 - 0.2\gamma_v - 0.2\gamma_v^2) = 1.06$<br>$\overline{F}_h = \gamma_h \times (\overline{K}_h \zeta f'_c b_j a_s) \times \cos \theta = 362 \text{ kN}$<br>$\overline{F}_v = \gamma_v \times (\overline{K}_v \zeta f'_c b_j a_s) \times \sin \theta = 184 \text{ kN}$<br>$K_h = 1 + (\overline{K}_h - 1) \frac{F_{yh}}{F_h} = 1.013 \leq \overline{K}_h$<br>$K_v = 1 + (\overline{K}_v - 1) \frac{F_{yv}}{F_v} = 1.023 \leq \overline{K}_v$ | $\overline{K}_h = 1.16$<br>$\overline{K}_v = 1.06$<br>$\overline{F}_h = 376.7 \text{ kN}$<br>$\overline{F}_v = 191.4 \text{ kN}$<br>$K_h = 1.013 \leq \overline{K}_h$<br>$K_v = 1.024 \leq \overline{K}_v$ | —                                  |
| Shear strength                          | $V_{SST-1} = (K_h + K_v - 1) \zeta f'_c b_j a_s \cos \theta$<br>$= 685 \text{ kN}$                                                                                                                                                                                                                                                                                                                                                                                                                                               | $V_{SST-2} = 577 \text{ kN}$                                                                                                                                                                               | $V_{SST-3} = V_h = 585 \text{ kN}$ |
| Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN. |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                  |                                                                                                                                                                                                            |                                    |

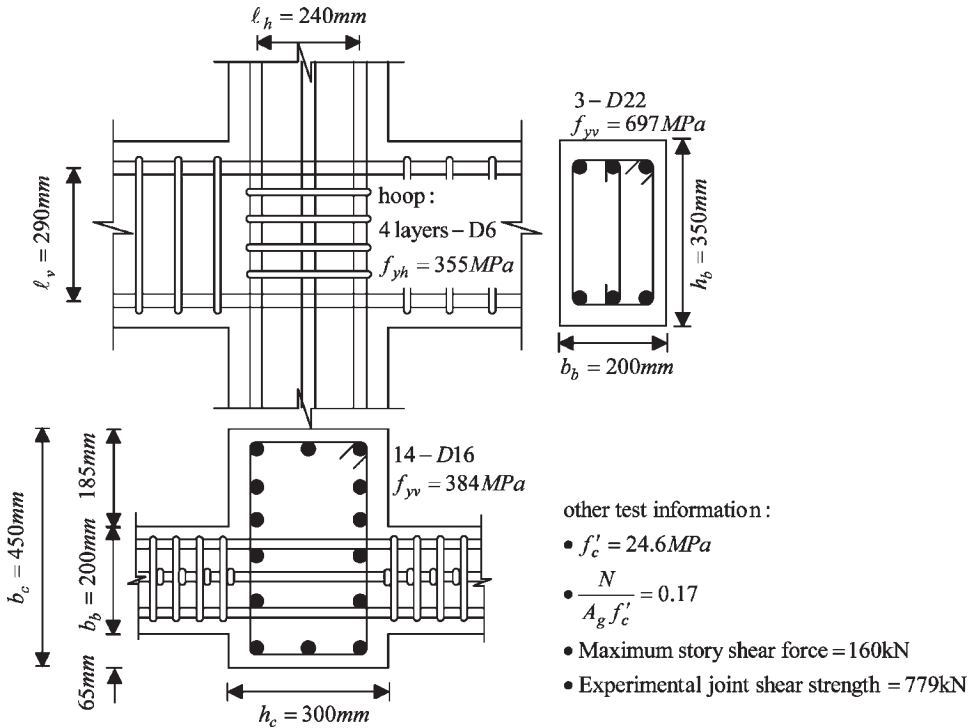


Fig. A1—Detailing of Specimen UM-60 (Goto and Joh 2004). (Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN; 1 MPa = 145 psi.)



# Reversed Cyclic Behavior of Reinforced Concrete Shear Walls with Diagonal Steel Grids

by J.-X. Zhong, Y.-L. Mo, and W.-I. Liao

**Synopsis:** Past reinforced concrete panel tests performed at the University of Houston have shown that reinforced concrete membrane elements under reversed cyclic loading have much greater ductility and energy dissipation when steel bars are provided in the direction of the principal tensile stress. This paper presents the experimental results of two low-rise and two mid-rise shear walls under reversed cyclic loading. The low-rise shear walls have a height-width ratio of 0.5, and the two mid-rise shear walls have a height-width ratio of 1.5. In critical regions, the wall reinforcements were designed in the orientation close to the principal stress direction. Furthermore, nonlinear finite element analyses of the tested walls were performed using the finite element analysis program Simulation of Reinforced Concrete Structures (SRCS), which was recently developed at the University of Houston. SRCS was developed by implementing the cyclic softened membrane model (CSMM) to the finite element framework OpenSees. The comparison showed good correlation between the predicted and experimental results of the four shear walls in terms of initial stiffness, ultimate strength, hysteretic loops, and energy dissipation, and the capability of SRCS to assess the cyclic behavior of shear walls with diagonal steel grids was validated.

**Keywords:** nonlinear finite element analysis; reinforced concrete; shear wall.

**Jian-Xia Zhong** is a Structural Engineer at MMI Engineering Inc., Houston, TX. She received her PhD from the University of Houston in 2005. Her research interests include nonlinear analysis of concrete structures and testing and analysis of structural walls.

**Yi-Lung Mo**, FACI, is a Professor in the Civil and Environmental Engineering Department and the Director of the Thomas T.C. Hsu Structural Research Laboratory at the University of Houston. He is a member of Joint ASCE-ACI Committees 445, Shear and Torsion, and 447, Finite Element Analysis of Reinforced Concrete Structures. He has 27 years of experience in studies of reinforced and prestressed concrete structures. He has written five books and book chapters and has published more than 100 journal papers, 100 conference papers, and 45 technical reports.

**Wen-I. Liao** is an Associate Professor in the Department of Construction Engineering at National Taipei University, Taipei, Taiwan. His research interests include experimental mechanics and smart materials and structures.

### INTRODUCTION

Reinforced concrete (RC) structural walls are commonly used as lateral force-resisting elements of structures under seismic loading. Structural walls can be divided into three groups according to the height-width ratio of the wall. When the height-width ratio of a wall is greater than 2.0, the wall is considered to be a high-rise structural wall dominated by flexure, and can be treated similar to a cantilever beam. When the height-width ratio of a wall is less than 1.0, the wall is considered to be a low-rise structural wall dominated by shear. When the height-width ratio of a wall is between 1.0 and 2.0, it is called a mid-rise structural wall, and is governed by both flexure and shear. Low-rise and mid-rise structural walls, which are mainly governed by shear, are also called shear walls.

Stiffness characteristics and strength of shear walls have been investigated by many researchers in the past (Lefas et al. 1990; Wood 1990; Farrar and Baker 1992, 1993; Cheng et al. 1993; Colotti 1993; Eberhard and Sozen 1993; Mickleborough et al. 1999). A great number of experimental studies have been conducted, and ductility factors, hysteretic characteristics, and energy dissipation capacities of shear walls under seismic loading have been studied (Oesterle et al. 1976, 1978; Fiorato and Corley 1979; Elnashai et al. 1990; Pilakoutas and Elnashai 1995, Mo and Kuo 1998a, b; Mo and Lee 2000; Tasnimi 2000; Lopes 2001; Han et al. 2002; Oh et al. 2002; Massone and Wallace 2004). Test results have shown that low-rise and mid-rise shear walls have less ductility and energy dissipation capacity when compared with flexural elements. Undesired pinching hysteresis has also been observed in the reversed cyclic response of shear walls. It should be pointed out that most of the previously tested shear walls were designed with wall reinforcement in the horizontal and vertical directions. Such walls are noted as “conventional shear walls” in this paper.

Since the 1980s, researchers have proposed to design conventional shear walls with diagonal web reinforcement and have performed reversed cyclic tests on these walls (Paulay et al. 1982; Mansur et al. 1992; Sittipunt and Wood 1995; Salonikios et al. 1999; Sittipunt et al. 2001). Experimental results have shown that diagonal web reinforcement can control the pinching effects in the hysteresis loops of shear

walls and increase their energy dissipation capacities.

Although previous research has shown that the effect of reinforcement orientation affects the cyclic behavior of shear walls, the research is not thorough, and needs further experimental and analytical study. First, for low-rise and mid-rise shear walls with a height-width ratio less than 2.0, the tests were conducted on shear walls with conventional orthogonal reinforcements and additional diagonal web reinforcements. Hence, the effect of web reinforcement orientations has not been investigated alone for low-rise and mid-rise shear walls. Second, most of the tested shear walls were high-rise and mid-rise shear walls. More experimental results on low-rise shear walls are needed. Moreover, very little work was done to study the seismic behavior of shear walls with diagonal steel grids using nonlinear finite element method. The only theoretical analysis in the literature by Sittipunt and Wood (1995) was on conventional shear walls. No comparison was made with the experimental results of shear walls with diagonal steel grids.

Recent tests on RC panels with reinforcement in various orientations were conducted at the University of Houston (Mansour and Hsu 2005a). Test results showed that the effect of the steel grid orientation played an important role on both the reversed cyclic shear stress-shear strain relationships and pinching effect of RC panels. Both the energy dissipation capacity and ductility were much greater when steel bars were placed in the principal tensile stress direction. Although the stiffness and the shear capacity of the panels with different reinforcement orientations were not directly compared, due to the fact that the steel ratios used in the panels were not the same, the experimental and analytical results (Mansour and Hsu 2005a, b) indicated that the stiffness and the shear capacity will be higher when the steel bars are placed in the principal tensile stress direction.

This paper presents the experimental results of two low-rise shear walls and two mid-rise shear walls under reversed cyclic loading. The low-rise shear walls have a height-width ratio of 0.5 and the two mid-rise shear walls have a height-width ratio of 1.5. In critical regions, the wall reinforcements were designed in the orientation close to the direction of average principal tensile stress. The test results of the shear walls are presented, including the cracking patterns, failure modes, and hysteretic characteristics. The effect of steel grid orientations on the reversed cyclic behavior of low-rise and mid-rise shear walls has been experimentally investigated.

On the other hand, shear research at the University of Houston performed over the past two decades resulted in a series of constitutive models, in which the cyclic softened membrane model (CSMM) (Mansour and Hsu 2005a,b) is the most recently developed. The CSMM model is a smeared crack model that treats the cracked RC as a continuous material. As such, the force equilibrium equations and the displacement compatibility equations are based on well-established continuum mechanics principles. The CSMM model is able to accurately predict the cyclic shear behavior of RC membrane elements (Mansour and Hsu 2005a, b). By adopting OpenSees (Fenves 2001) as a framework, CSMM was formulated and implemented into OpenSees and a nonlinear finite element analysis program, Simulation for Reinforced Concrete Structures (SRCS), was developed (Zhong 2005; Mo et al. 2008). SRCS has been applied to predict the cyclic response of conventional shear walls under reversed cyclic loading (Mo et al. 2008), and the

prediction showed excellent agreement with the test outcomes. In this paper, SRCS was used to perform nonlinear dynamic response analyses on the four tested shear walls and the predicted responses were compared with the test data. It was found that the analytical and experimental results are in reasonably good agreement in terms of initial stiffness, ultimate strength, hysteretic loops, and energy dissipation and the capability of SRCS to assess the cyclic behavior of shear walls with diagonal steel grids was validated.

## EXPERIMENTAL PROGRAM

### Test parameters

Four specimens, RLN, RLB, RMN, and RMB, were tested subjected to reversed cyclic horizontal loading. The labels of all specimens start with the letter “R,” signifying that the loading type was reversed cyclic loading. The second letter, “L” or “M,” represents low-rise or mid-rise wall. The third letter, “B” or “N,” means that the walls were designed with boundary columns or no boundary columns. A list of the specimen properties is shown in [Table 1](#), where the test parameters are indicated. All specimens have a thickness of 120 mm (4.7 in.) in the wall panel.

### Steel grid orientations

As per the test results of RC panels performed at the University of Houston (Mansour and Hsu 2005a), energy dissipation capacity and ductility of the RC panels were much greater when steel bars were placed in the principal tensile stress direction. Note that the RC panel represents a single wall component and the principal stress direction in the panel remained the same during the entire test procedure.

The shear wall is a structure composed of many elements. The principal stress directions in the wall may be different at different locations. Moreover, the principal stress direction in the wall will also change from the initial elastic stage to the post yielding stage when concrete cracking occurs and steel starts to yield. Therefore, it is impossible to have the wall reinforcement match the directions of the principal stresses all round the shear wall during the entire test. Instead, the steel grids were located to provide the critical region of the shear wall with orientations close to the average principal stress direction.

For the two low-rise shear walls which are mainly governed by shear, the principal stress direction of the wall is close to 45 degrees. The orientations of the steel bars in the two low-rise shear walls (RLN and RLB) were provided at 45 degrees to the horizontal.

For the two mid-rise shear walls (RMN and RMB) controlled by shear and flexure, the lower third of the walls was considered as the critical region. The upper two-thirds of the walls were less critical and reinforced with steel bars in the conventional horizontal and vertical direction. In the lower third of the wall, half of the wall reinforcement was provided in diagonal directions with 65 degrees to the horizontal to approximate the average principal stress direction, and another half of the wall reinforcement was provided in the conventional horizontal and vertical directions to provide a certain level of continuity of the reinforcement with the upper part.

### Specimens

In the series of specimens with a height-width of 0.5, that is, Specimens RLN and