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Seismic response of slab-column connections with shear capitals

Wey, Eric Hamilton, M.S.

Rice University, 1991
RICE UNIVERSITY

SEISMIC RESPONSE OF SLAB-COLUMN CONNECTIONS
WITH SHEAR CAPITALS

by

ERIC HAMILTON WEY

A THESIS SUBMITTED
IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE
MASTER OF SCIENCE

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April, 1991
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WITH SHEAR CAPITALS

BY

ERIC HAMILTON WEY

ABSTRACT

The punching shear strength of the slab usually governs the design of slab-column connections in regions of high seismicity. One method of increasing the shear capacity of the connection region is to increase the thickness of the slab using shear capitals. A shear capital is a thickened portion of the slab in the vicinity of the column used solely for improving the shear strength of the connection region.

Three interior and three edge slab-column connections with different size shear capitals were tested under simulated earthquake type loading and compared to connections without shear capitals. Based upon the test results, the presence of a shear capital improves the strength and stiffness of a slab-column connection. A shear capital should be reinforced and have a sufficient length with respect to the depth of the slab when load reversals are expected.
ACKNOWLEDGMENT

This thesis was submitted by Eric Hamilton Wey in partial fulfillment of the requirements for the degree of Master of Science (Civil Engineering) in the George R. Brown School of Engineering at Rice University. The investigation reported herein was sponsored by the National Science Foundation under grant No. CES-8811933, which is gratefully acknowledged. Any opinions, findings and conclusions expressed in this report are those of the author and do not necessarily reflect the views of the sponsor. The research was directed by professor A.J. Durrani, whose guidance and encouragement is greatly appreciated.

The author would like to acknowledge professors J.E. Merwin and P.C. Dakoulas for reviewing the report and offering helpful suggestions. Thanks are also due to Mr. Hugh Hales for the technical support, and to all the undergraduate students who helped in the testing program at various stages of the project. A special thanks is due to family and friends of the author for their encouragement and support throughout the project.
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NOTATION

$A_c$ = area of slab critical section

$b_c$ = perimeter of slab critical section

$b_1$ = width of the critical section measured in the directions of the span for which moments are determined

$b_2$ = width of the critical section measured in the direction perpendicular to $b_1$

$c_1$ = column dimension perpendicular to slab edge

$c_2$ = column dimension parallel to slab edge

$d$ = effective depth of slab

$f'_{c}$ = compressive strength of concrete

$f_y$ = yield strength of slab reinforcement

$f_u$ = ultimate strength of slab reinforcement

$h$ = slab thickness

$K_i$ = initial peak to peak stiffness

$K_p$ = peak to peak stiffness at peak load

$L_{1_{sc}}$ = width of shear capital perpendicular to slab edge

$L_{2_{sc}}$ = width of shear capital parallel to slab edge

$(M_n)_c$ = moment capacity of the slab at the column face

$(M_n)_{sc}$ = moment capacity of the slab at shear capital edge
$M_u =$ observed moment at centerline of column at peak lateral load

$(M_u)_{sc} =$ unbalanced moment at centroid of critical section located $d/2$ outside shear capital in an edge connection at peak lateral load

$T_c =$ concrete torsional strength (ACI eqn. 11-22)

$T_n = T_c + T_s =$ nominal torsional capacity of slab edge

$T_s =$ torsional strength of steel (ACI eqn. 11-23)

$v_c =$ nominal shear capacity of slab at column face

$v_{sc} =$ nominal shear capacity of slab at shear capital face

$v_{ab} =$ shear stress in test specimen at column face negative moment peak load

$v_{sd} =$ shear stress in test specimen at column face positive moment peak load

$v_{sf} =$ shear stress in test specimen at shear capital face negative moment peak load

$v_{gh} =$ shear stress in test specimen at shear capital face positive moment peak load

$V_u =$ direct shear force at peak lateral load

$\gamma_f = 1 - \gamma_u =$ portion of transfer moment resisted by flexure

$\gamma_u =$ portion of transfer moment resisted by eccentric slab shear stress

$\rho_{sc} =$ shear capital reinforcement ratio

$\theta_p =$ angle of punching shear failure surface to horizontal
CHAPTER 1

INTRODUCTION

1.1 GENERAL

One of the major disadvantages of reinforced concrete flat-slab buildings is their inability to sustain large lateral loads such as those due to earthquakes. Buildings of this type often lose strength and stiffness very easily when subjected to such loads. Moment and shear created by lateral loads must be transferred at the slab-column interface. The unbalanced moment is transferred by a combination of flexure, torsion and shear in the slab around the periphery of the column faces. When shear stresses in the region of the slab around the column become too large a punching failure will occur. If a punching failure occurs in slab-column joints which are not properly reinforced it may lead to a progressive collapse of the structure causing catastrophic damage.

In view of the damage potential of flat-slab buildings, proper design and detailing of slab-column connections is essential in regions where seismic loads are expected. Additional lateral load resisting
elements, such as shear walls, are added to buildings in regions of high seismic risk. Slab-column frames alone, however, are considered adequate as the primary lateral load resisting system in regions of low to medium seismicity, such as zones 1 and 2 as defined in the Uniform Building Code (ref. 1).

In buildings with long spans and heavy gravity loads, the region of the slab around the columns is thickened in the form of drop panels to decrease deflections and increase the flexural and shear strength. When the proper size of a drop panel is used the ACI Building Code (ref. 2) requirements allow the designer to reduce the amount of negative moment reinforcement at the column. Also, the depth of the slab may be decreased by as much as ten percent of the depth required to control deflections in an equivalent flat-plate structure without drop panels. The ACI Building Code provides specific recommendations on the sizing of drop panels which are effective in reducing the deflections when subjected to gravity loads. However, the relationship between the size of the drop panel and the improvement of the moment capacity and shear strength has not been clearly established for connections subjected to seismic loads.
A shear capital can be regarded as a drop panel with a small plan area and a more shallow depth than a column capital which is used solely for improving the shear strength of the connection region. The configuration of a shear capital compared to that of a drop panel and column capital is shown in Fig. 1.01. The use of shear capitals in the design of flat-slab structures for improving the shear strength of the connection region has been increasing. There is, however, little experimental data available which will describe the behavior of shear capitals and their improvement of the strength, stiffness and energy dissipation of the connection when subjected to seismic loads. Such data would be instrumental in preparing guidelines for their use in structural design.

1.2 OBJECTIVE

The main purpose of this investigation was to determine the effectiveness of shear capitals on improving the performance of the connection regions of flat-slab structures when subjected to seismic loads. Some of the specific objectives were (1) to understand how the size of the shear capital affected the failure mechanism of a slab column connection, (2) to determine how the presence of a shear capital changed the strength,
stiffness and energy dissipation of connections, (3) to estimate the optimum shear capital plan dimensions which would be effective in improving the behavior of slab-column connections, and (4) to evaluate the adequacy of the current recommendations for the design of slab-column connections with shear capitals.

In order to accomplish these objectives single connection interior and edge, flat slab subassemblies with shear capitals were tested under a simulated earthquake type loading. The dimensions and reinforcement of the slabs and columns and the thickness of the shear capitals remained unchanged in all specimens. Only the plan area of the shear capital was changed. All specimens were subjected to the same reversed cyclic displacements of predetermined amplitudes.

1.3 LITERATURE REVIEW

Early research into the problem of moment transfer in the connection regions of a flat plate structure was conducted by Di Stasio and Van Buren (ref. 2). They proposed a method of calculating the shear stresses around the column periphery due to shear and unbalanced moment due to gravity loads in which the stresses are
distributed linearly about a critical section. In this model equilibrium of moments about the center of the connection region is used to determine the amount of torsion transferred through the faces of the column perpendicular to the axis of bending. This torsion is the portion of the unbalanced moment which causes shear stresses on the column periphery. In Moe’s model (ref. 3) a fraction, \( K \), which was suggested to be between \( 1/5 \) and \( 1/3 \), of the unbalanced moment is to be transferred to the joint by shear stresses. Moe also suggested that the ultimate shear strength of the slab is proportional to \( \sqrt{f_c'} \) and the \( r/d \) ratio, where \( r \) is length of a side of a square column or \( 1/4 \) of a perimeter of a round column and \( d \) is the effective depth of the slab. The 1963 ACI Building Code limited the shear stress on the critical section, located \( d/2 \) from the column, to \( 4\sqrt{f_c'} \) and used Moe’s eccentric shear stress model with \( K = 1/5 \).

Hanson and Hanson (ref. 5) reported 17 tests which include rectangular and square interior columns, square interior columns with adjacent slab openings and square edge columns. Three interior column specimens were subjected to load reversals. From the results of these tests they suggested that \( 2/5 \) of the unbalanced moment should be transferred to a square column by shear
stresses and 3/5 by flexure. These results were implemented in the 1971 code revision.

Many researcher (refs. 6, 7 and 8) have attempted to develop a more rational approach for analyzing the problem of moment and shear transfer in slab column connections. Although some of these methods may be more rational, they are too complicated for practical use. Alexander and Simmons (ref. 6) presented a model which explained that the load transfers occurred by means of a combination of flexure and compression struts fanning out from the column. The method uses a truss analogy and is the best known model for describing the connection behavior.

The historical development of the ACI 318-83 code provisions for transfer of shear and unbalanced moment at slab-column connections is well documented by Grossman (ref. 10). The current code method is simple but underestimates the forces transferred on the side faces of the column (refs. 11 and 12) and neglects the effect of reinforcement and its contribution in the post-cracking stage.

The application of the eccentric shear stress model to exterior and corner slab-column connections has been criticized in recent years (refs. 9, 13 and 14). The
ACI-ASCE committee 352 suggested an alternative method of calculating the shear stress for exterior connections in the revised recommendations for the design of slab column connections (ref. 14). Moehle (ref. 13) suggested that there is no definite interaction between shear and moment at exterior connections.

Several researchers (refs. 4, 15 and 16) have shown through test results that the r/d ratio affected the shear strength of a slab-column connection. As the size of the column became larger with respect to the thickness of the slab the shear capacity of the critical section reduced. Equation (11-37) of the ACI 318-89 Building Code (ref. 2) implies that the allowable shear stress on the critical section decreases as the $b_o/d$ ratio increases.
CHAPTER 2

EXPERIMENTAL PROGRAM

2.1 TESTING PROGRAM AND ASSUMPTIONS

Three interior and three edge slab-column connection subassemblies with shear capitals were tested during this investigation. The results from these tests were compared to two control specimens, one interior connection and one edge connection, without shear capitals tested in a previous study (ref. 17). The test specimens were approximately one-half scale models of slab-column connections of the prototype flat-plate building.

In addition to gravity loads, specimens were subjected to statically applied cyclic lateral loads. The static application of the load allowed detailed observation and recording of the response of each subassembly during the test. Such a quasi-static type of loading has been extensively used in a large number of past similar studies (refs. 5, 17,18 and 19). With a similar loading scheme, the results of this investigation can be compared to test data from previous investigations.

The specimen configuration is based on the
assumption that, under lateral loading, the points of contraflexure in a multistory frame are located at the mid-height of the columns and the mid-span of the slab. The boundaries of the test specimens are located at these inflexion points which are assumed to remain stationary during the loading routine. With this testing arrangement the analysis of the test specimen is considerably simplified since the subassembly is statically determinate and the measured response can be easily verified with theoretical calculations.

2.2 PROTOTYPE DESIGN

The control specimens were based on a prototype building designed in a previous study of slab-column connections (ref. 17). The prototype building consisted of a two bay, five story slab-column frame shown in Fig. 2.01. This frame had 20 in. square columns and 20 ft. spans between column centerlines. The design loading was typical of an office or apartment building located in a moderate earthquake zone. Gravity loads on the prototype building consisted of the slab self weight plus 20 psf superimposed dead load and 50 psf superimposed live load. The NEHRP design recommendations (ref. 20) for a category 2 or moderate earthquake were used to obtain the design lateral earthquake loading. The moments and shears
obtained from lateral load analysis of the frame were added to those obtained from gravity load analysis to get the design slab moments and shears. The ACI 318-83 Building Code (ref. 22) was used to design the slab. Shear capacity of the slab on the critical section controlled the final slab thickness of 9 inches.

2.3 SUBASSEMBLY DESIGN

The dimensions of the test specimens were governed by the constraints of the testing frame. The width of each test specimen transverse to the direction of the lateral load was 6.5 feet. Previous research studies have observed that discontinuity in the lateral direction may not have a significant effect on the behavior of slab-column connections (ref. 21). The total length of the slab for the interior connection specimens in the direction of loading was 9.5 feet. The length of the slab from the discontinuous edge to centerline of the column for the edge connections was 4.75 feet. For a true half-scale modeling of the chosen prototype, these dimensions would have been 10 ft., 10 ft. and 5 ft., respectively. A typical interior and edge connection subassembly are shown in Figs. 2.02 and 2.03, respectively.

The reinforcement in the slab and column of all
specimens was identical. The column reinforcement consisted of eight No. 7 bars with No. 3 bar ties spaced at 3 inches. The slab reinforcement consisted of No. 3 bars spaced at 3 in. in the effective width for moment transfer, according to the ACI code, of \( c_2 + 3h \) centered on the column and 9 in. elsewhere. The main slab reinforcement layout for an interior connection subassembly is shown in Fig. 2.04. Photo 2.01 shows the reinforcement placement in an interior column connection subassembly.

As required by the ACI code, the reinforcement passing through the column in a direction parallel to the edge was enclosed by No. 2 closed loop stirrups spaced at 2.5 in. on center. Figure 2.05 shows the reinforcement layout for the main slab of a typical edge column connection. The placement of reinforcement in an edge column connection is shown in Photo 2.02.

The depth of the slab was 4.5 in. while the thickness of the shear capital beyond the slab was always 3.5 inches. At the column face the total thickness of the slab was 8 inches. Each of these dimensions remained constant in all test specimens.

### 2.4 Design of Shear Capitals

Three different plan dimensions of shear capitals
were studied during this investigation. An interior and edge column connection subassembly were cast simultaneously for each shear capital size. The largest shear capital, with a 40 in. x 40 in. plan area, can be considered a drop panel. Its proportions met all of the ACI requirements for decreasing the amount of negative moment reinforcing steel over the column and reducing the slab thickness to control deflections. The smallest shear capital, with a 24 in. x 24 in. plan area, had dimensions which were half-scale of a practical size (4 ft. x 4 ft.) convenient for construction. The 32 in. x 32 in. shear capital size was chosen to be exactly in between the two previously mentioned sizes. Table 2.01 contains the size of shear capital for each test specimen and the code name which it will be referred to throughout this report.

Shear capitals were reinforced with Grade 60 No. 3 bars. Two bars passed through the column in each direction between the longitudinal reinforcement of the column. Rebar outside the column was spaced evenly throughout the shear capital. The reinforcement ratio of the shear capital, shown as $\rho_{sc}$ in Table 2.01, was approximately the same in each specimen while using the same size rebar. In the 40 in. x 40 in. shear capital eight bars were placed in each direction, the 32 in. x 32
in. shear capital had six bars in each direction and the 24 in. x 24 in. shear capital had four bars in each direction. The layout of the reinforcement for the different sizes of shear capitals is shown in Fig. 2.06. Photos 2.03 and 2.04 show the placement of reinforcement within the shear capitals of edge and interior connection specimens, respectively.

The shear capital reinforcement was designed with several key considerations in mind. The presence of a shear capital creates a sudden change of slab thickness at a certain distance from the column. Tensile stress concentrations can develop where there is a sudden change of stiffness which may lead to localized failure in the slab. Since specimens were being subjected to reversed loading, the addition of some reinforcement at the bottom face of the shear capital was deemed appropriate to counter tensile stresses caused by positive bending. The typical shape of the rebar used in a shear capital is shown in Fig. 2.07. It should be noted that the shear capital reinforcement which passed through the column can be effective in preventing a progressive collapse in the case of a punching shear failure (ref. 15). A standard hook is used to anchor the vertical portion of the reinforcement into the main slab.
2.5 FABRICATION OF SPECIMENS

The specimens were cast in reusable wooden formwork which was modified after each casting operation to accommodate different shear capital sizes. The formwork was constructed with 3/4 in. plywood stiffened with 2 in. x 4 in. studs. Care was taken to sand and coat the formwork with polyurethane in between each casting operation. Shortly before each casting operation the formwork was coated with a form releasing agent for ease of stripping. The formwork was set up in such a way that the bottom column, the slab and the top column could be cast at one time.

Before the slab reinforcement was assembled and placed inside the formwork, strain gages were installed at appropriate locations for strain measurement. The reinforcement was placed in three layers. The first layer was the shear capital reinforcement, next was the bottom main slab reinforcement and finally the top main slab reinforcement. Typical locations of strain gages in each specimen are given in Figs. 2.04 - 2.06.

For attachment with the frame and for load application to the column, one inch thick steel plates were attached to the top and bottom of the column. Each plate was anchored into the column by four No. 7 reinforcement bars, one foot in length which were
threaded and screwed into the plate and then welded to the plate. Four more holes were tapped into the plate in order to attach a clevis which would connect the specimen to the test frame.

After the casting process was finished plastic sheets were placed over the slab surface to prevent excessive loss of water during the initial setting period. The plastic sheeting was removed after 24 hours in order to place wet burlap over the slab. The wet curing process was continued for one week.

2.6 MATERIAL PROPERTIES

All of the reinforcement was Grade 60 steel except for the stirrups in the edge beams within the slab thickness of the edge connection specimens. These stirrups were made from No. 2 undeformed bars that were A36 steel. Tensile strength tests were conducted on several reinforcing bar the results of which are summarized in Table 2.02. The yield strength for the Grade 60 steel was 82 ksi and 42 ksi for the A36 steel. For convenience and to facilitate fabrication, the stirrups had overlapping legs which were welded together.

The concrete used for the test specimens was obtained from a ready-mix company. The mix had a water-cement ratio of 0.57. It contained limestone aggregate of 1 in.
maximum size, 16.5 oz. of plasticiser per cubic yard and 5.5 sacks of Type I Portland Cement per cubic yard. The target slump was 3 inches.

Concrete was cast three different times, one time for each size of shear capital. The measured slumps ranged from 2.5 in. to 4.5 in. Compression tests were performed on standard 6 in. x 12 in. cylinders made from each casting of specimens. In addition, standard 6 in. x 6 in. x 24 in. beams were also tested for each batch of concrete. The measured properties of concrete on the day of testing of specimens is given in Table 2.03.

2.7 TEST SET UP AND INSTRUMENTATION

The testing frame used in the testing of specimens is shown in Photo 2.05. Test specimens were maneuvered into the testing frame with the use of a 20 ton overhead crane. The testing frame was set up so that a stiff I-beam attached to the top of the column could be loaded and displaced in a horizontal direction by means of a servo-controlled actuator.

The test setup was designed such that all of the column reactions, both shear and axial, could be monitored throughout the test. Figure 2.08 shows a schematic diagram of the test setup. Reversed cyclic displacements were applied by the actuator to the I-beam
which was attached to column of the specimen by a hinged connection. The lower portion of the column was connected to the bottom of the testing frame and restrained from lateral movement but allowed to rotate in the direction of loading. After applying the gravity load the free ends of the slab were attached to the reaction links which simulate roller supports.

The applied load at the top of the column, the reactions at the free ends of the slab, and the bottom of the column were measured independently by load cells. Displacement transducers (LVDTs) were used to measure the displacement at the top of the column, rotation of the slab with respect to the column and the rotation of the edge beam. Numerous strain gages were attached to the reinforcement around the connection region to monitor the extent of yielding in the slab reinforcement.

2.8 DISPLACEMENT HISTORY

During each test, predetermined lateral displacements were applied to the top of the column while the resulting loads, rotations and strains were measured. Each imposed displacement was a certain percentage of the total column height referred to as drift. A drift cycle is the displacement of the column from its zero position to a certain drift in one direction, back to the same
magnitude of drift in the opposite direction and to zero position again. A typical lateral displacement history used during this investigation is shown in Fig. 2.09

The lateral displacement history used in each test consisted of a number of drift cycles of gradually increasing amplitude. It also included drift cycles of small amplitude to study the initial elastic response of the specimens. Cycle at certain drift were repeated to study the strength degradation of the test specimen. Small amplitude cycles were introduced in between drift cycles of large amplitudes to study the loss of stiffness of the specimen throughout the test. Not all specimens were subjected to the full displacement history. Once the connection of the specimen failed the test was terminated.

2.9 TESTING PROCEDURE

After the specimen was secured in the testing frame and the column was fixed in its vertical position the gravity loads were applied. The gravity load, which remained the same in all specimens, simulated the gravity load used in the design of the prototype structure. This load was applied to the slab by hanging 450 pound concrete weights from cables anchored on the top surface of the slab. The slab was allowed to deflect before
attaching the vertical reaction areas. The instruments were all checked and the testing commenced. A computer program was developed to control the actuator displacement and to record measurements at specified times during the loading routine.

At certain points during the test the applied displacement was held constant so that the test specimen could be observed and the crack pattern could be recorded. A typical displacement cycle consisted of three ramps. In the first ramp the actuator moved from its zero displacement position to the peak positive displacement for that cycle. This position was held while the cracking pattern was marked and recorded. In the second ramp the position of the actuator began at the positive peak displacement and ended at the peak negative displacement. The position of the actuator was again held while the cracking pattern was marked and recorded. The actuator then moved from the peak negative displacement to the zero displacement position in the final ramp. This procedure was repeated for each drift cycle. The cracking patterns were observed in the test specimens for all drift cycles until 4.0 percent drift was reached.
CHAPTER 3

TEST RESULTS AND OBSERVATIONS

3.1 INTRODUCTION

The data collected from the test specimens consisted of visual observations and those obtained from instrumentation. Visually observed results included crack patterns, geometry of the punched failure surfaces, location of flexural yield lines in the slab flexural hinging region, and the torsional cracks at the slab discontinuous edge. Results from instrumentation included hysteretic load-deformation plots, rotation measurements of the slab hinging region, and strain histories at selected locations of reinforcing bars. From the measured response, the strength, stiffness and energy dissipation characteristics of each specimen can be deduced.

In the following sections each of the quantities mentioned above are defined and described as separate topics. The general characteristics and common trends that pertain to all of the test specimens are discussed fully. Next the effect of the presence of a shear capital on various response parameters is discussed.
3.2 CRACKING PATTERN

Cracks were marked and recorded at peaks of every drift cycle of less than 4 percent drift. The cracking patterns for all of the test specimens, including the control specimens, are shown diagrammatically in Figs. 3.01 to 3.08. For each specimen with a shear capital, the cracking patterns are shown at 1.5 percent drift and 3.0 percent drift. The cracking pattern for the control specimens are shown at 1.5 percent drift and 3.5 percent drift.

The test specimens generally remained uncracked after the application of the gravity load and before the application of the lateral load. During the initial small cycles the bottom surface of the slab remained uncracked. Cracks on the bottom surface of the slab began to appear as soon as the lateral load became large enough to overcome the negative moment at the face of the column or shear capital caused by the gravity loads. This occurred at about 0.75 percent drift. As the lateral load became larger, new cracks formed and the existing cracks further developed. The crack patterns were generally fully developed by the 3.0 percent drift level. At higher drift levels, few new cracks formed and the existing cracks widened.

The type of cracks most commonly observed on the test
specimens were flexural cracks. These were generally the first to appear on the test specimens and occurred in the slab along lines perpendicular to the direction of loading and most commonly appeared at the edges of the shear capital or the faces of the column. Some of these cracks occurred at locations of transverse steel in the slab.

Torsional cracks in edge connections began on the tension side of the slab at the column faces and propagated along a diagonal at about 45 degrees from the column face as a series of parallel diagonals which continued around the slab edge. If the cracks began on the top of the slab the series of diagonal cracks formed in a counterclockwise direction away from the column. If they began on the bottom the diagonal cracks formed in a clockwise direction away from the column.

Punching shear failure under cyclic lateral loading was observed to occur in stages. When shear capacity is reached on one side of the column, cracks forming on the tension side of the slab propagate diagonally towards the column to the compression side of the slab. Similar cracks form on the opposite side of the slab when the load is reversed. After these cracks have developed, unbalanced moment cannot be transferred from the slab to the column directly by flexure. At this stage, the unbalanced moment is transferred to the slab through faces of the column
parallel to the direction of loading by torsion. Torsion in this region of the slab generates shear stresses which cause the shear cracks to spread in a brittle manner forming a failure surface of a truncated pyramid centered about the column and result in a sudden loss of strength of the connection.

3.2.1 INTERIOR CONNECTIONS

Flexural cracks in interior connections first appeared on the top of the slab during the initial small drift cycles. These cracks spread throughout the entire width of the slab by about 1 percent drift. They primarily occurred at the face of the column or at the edges of the shear capital. Similar cracks began to form on the bottom of the slab at about 0.75 percent drift.

Cracks in the slab of specimen PI8 with no shear capital, shown in Fig 3.01, were concentrated near the column. Cracks in specimens with shear capitals, shown in Figs. 3.02 to 3.04 were concentrated around the edges of the shear capital as well as the column but were spread throughout the slab more than in specimen PI8. This shows that the presence of a shear capital made a larger portion of the slab work to resist loads caused by lateral drift. In other words, the shear capital made the slab more efficient in resisting these loads.
Although flexural cracks were predominant in specimen PI8 during low drift cycles, a punching shear failure occurred during higher drift cycles. At about 3.5 percent drift a shear crack occurred near the column on the side in which the top of the slab was in tension. This crack propagated diagonally downward towards the column. When the load was reversed a similar crack occurred in the slab on the opposite side of the column. As the drift increased to about 5 percent drift a complete punching failure occurred in the slab on all faces of the column. The failure surface, which is typical for a punching shear failure in a slab-column connection subjected to cyclic lateral loading (refs. 6 and 17), had a shape resembling a truncated pyramid with its apex below the slab as shown in Fig. 3.33.

Specimen SC6, with a 24 in. shear capital, also experienced a punching shear failure but the failure surface was inverted compared to that of specimen PI8, with no shear capital. At about 4.0 percent drift the positive moment caused by lateral drift on one side of the column became larger than the negative moment caused by gravity loads resulting in an inverted punching shear failure in the slab. The first punching shear crack developed on the bottom of the slab at the edge of the shear capital. Damage to this region of the slab is shown in photo 3.02.
This crack propagated upward towards the column to the top of the main slab. The damage that occurred on the top of the slab at the column due to these shear cracks is shown in photo 3.01. A similar crack developed in the slab on the opposite side of the column when the load was reversed. At about 6 percent drift a punching failure occurred in the slab which had a shape of a truncated pyramid with its apex above the slab as shown in Fig 3.34.

In all interior connections with shear capitals, cracks occurred at the interface of the shear capital and slab at about 1.5 percent drift, as shown in Figs. 3.02 to 3.04. Due to proper detailing of shear capital reinforcement, the shear capitals continued to contribute to the strength and stiffness of the connections after these cracks occurred. The hooks provided in the shear capital reinforcement anchored the shear capital to the slab. Punching shear failure was observed at the interface of the slab and shear capital in the specimen with a 24 in. shear capital at about 4 percent drift. The shear capital, however, never became completely separated from the connection. The slab-shear capital interface was the location of flexural hinging in specimens with 32 in. and 40 in. shear capitals. No punching shear failure was observed in these two specimens which is attributed to sufficient length of the shear capital with respect to the
depth of the slab. Flexural cracks were predominant throughout the test in these two specimens. At about 6 percent drift a flexural hinging mechanism, as shown in Fig. 3.32, developed in the slab. At drifts of this magnitude changes in the deflected shape of the slab could be visually observed, as shown in photo 3.03.

3.2.2 EDGE CONNECTIONS

In the edge connection specimen PE9, with no shear capital, the cracks developed predominantly near the slab-column interface. The first cracks to appear were torsional cracks at the slab discontinuous edge near the column. Flexural cracks occurred at the face of the column and at locations of slab transverse rebar at about 0.75 percent drift. These cracks spread across the entire width of the slab at about 1.5 percent drift. The most significant flexural cracks occurred at the face of the column.

Torsional cracks at the slab discontinuous edge became fully developed by about 1.5 percent drift as shown in Fig. 3.05 (a). More torsional cracks formed near the column and existing cracks opened wider at about 3.0 percent drift as shown in Fig. 3.05 (b). At about 4.0 percent drift concrete cover began to spall off at the discontinuous edge of the slab near the column. No sudden failure was
observed in this specimen. A combined torsional-flexural mechanism occurred at the slab-column interface.

In specimen SC5, with a 24 in. shear capital, flexural cracks at the edge of the shear capital formed across the entire width of the slab at about 0.5 percent drift as shown in Fig. 3.06. Unlike specimen PE9, flexural cracks at the face of the column were not predominant. Flexural cracks at the face of the shear capital were the most significant and contributed most to the failure mechanism of the specimen.

Torsional cracks in specimen SC5 developed inside the shear capital at the slab discontinuous edge during drift cycles less than 1.0 percent, as shown in Fig. 3.06 (a), but remained small and insignificant. At about 2.0 percent drift torsional cracks became fully developed in the slab discontinuous edge outside the shear capital. Concrete cover began to spall off in this area at about 5 percent drift. Torsional damage to the slab discontinuous edge at the end of the test is shown in photo 3.06. As in specimen PE9, there was no sudden failure in this specimen. The interface between the slab and the shear capital suffered severe damage during the test, whereas, the interface between the column and shear capital did not.

No significant torsional cracks occurred at the slab discontinuous edge in specimen SC3, with a 32 in. shear
capital. Some torsional cracks occurred within the shear capital at drift less than 1.5 percent, as shown in Fig. 3.07 (a), but they remained small and insignificant. After being subjected to drifts of up to 7 percent little significant torsional damage occurred at the slab discontinuous edge as shown in photo 3.05. Flexural cracks which occurred at the edge of the shear capital were the most significant. They stretched across the entire slab width by about 0.75 percent drift. As the drift increased these cracks became wider. The failure mechanism was flexural hinging at the edge of the shear capital.

Torsional cracks within the shear capital were the first cracks to appear in specimen SC1, with a 40 in. shear capital. There were flexural cracks at the face of the column as well as fully developed torsional cracks by about 1.5 percent drift, as shown in Fig. 3.08 (a). Significant flexural cracks spread across the entire slab width at the edge of the shear capital. At about 3.0 percent drift the torsional cracks within the shear capital and the flexural cracks at the edge of the shear capital opened wider. At larger drifts concrete between torsional cracks in this region began to spall off as shown in photo 3.04. Like other edge connections, there was no sudden failure during the test of this specimen. Two different failure mechanisms were observed in this specimen. One was a
combined torsional-flexural failure at the column-shear capital interface and the other was flexural hinging at the edge of the shear capital.

3.3 LOAD-DRIFT RESPONSE

The drift referred to in this section is the horizontal displacement of the top of the column relative to the base of the column divided by the column height. In a general sense, it represents the interstory drift. The load refers to the resistance of the specimen to a given amount of lateral displacement.

The load-drift plots for all of the test specimens are shown in Figs. 3.09 to 3.16. The response of all test specimens was essentially elastic until about 0.75 percent drift. This can be seen from the roughly linear load-drift relationships. As more cracks appeared and became wider, the curves became flatter around zero displacement due to pinching effect. The load-drift response to become more rounded near locations of peak load due to the Bauchinger effect in the slab rebar.

It is observed from these plots that there is no distinct cracking point or yielding point as might be expected in tests of a beam-column connection. When a slab begins to crack it does so gradually as the cracks extend across its width. It may take several cycles before a
crack develops across the entire slab width. Yielding of rebar initiated in slab bars closest to the column and progressed across the slab as the drift level increased.

3.3.1 INTERIOR CONNECTIONS

The load-drift response for specimen PI8, with no shear capital, is shown in Fig. 3.09. The distance between the loading and unloading curves of each drift cycle became wider and the peak load increased at a smaller rate with respect to an increase in drift after 1 percent drift. This occurred at the same time the slab steel yielded as indicated from strain gages. The peak load occurred at 3.5 percent drift and punching failure initiated during the following drift cycle. At about 4.5 percent drift the connection experienced a sudden loss of strength of the connection due to punching of the slab.

The load-drift response of specimen SC6, shown in Fig. 3.10, was significantly different than that of specimen PI8. For drifts less than 4 percent, specimen SC6, with a 24 in. shear capital was 30 percent stronger than specimen PI8. At about 4.0 percent drift all of the slab steel yielded, as indicated from the strain gages, and punching shear failure was initiated. A decrease in load at peak drift during larger drift cycles shows the effect of punching in the slab.
The load-drift response of specimen SC4, shown in Fig. 3.11, was very similar to that of specimen SC6 until about 4.0 percent drift. Unlike specimen SC6, the peak loads of drift cycles greater than 4.0 percent drift did not decrease as drift increased. Approximately the same peak load as at 4.0 percent drift was maintained until the end of the test. The load-drift response for specimen SC2 with a 40 in. shear capital, shown in Fig. 3.12, was very similar to that of specimen SC4 except the peak loads for drift cycles greater than 4.0 percent increased with increasing drift.

3.3.2 EDGE CONNECTIONS

The load-drift response for specimen PE9, with no shear capital, is shown in Fig. 3.13. After the initial cracking in the slab at about 0.5 percent drift, the peak load increased at a slower rate with increasing drift. At drift cycles greater than about 3 percent the peak cycle loads decreased with an increase in drift and the unloading portion of the curve reached zero load at about half the cycle drift. This specimen was about half as strong but more ductile than interior connection specimen PI8 which had no shear capital.

The load-drift response for specimen SC5, with a 24 in. shear capital, is shown in Fig. 3.14. After the
initial cracking at about 0.75 percent drift the peak cycle load increased until about 3.0 percent drift. Similar behavior was observed in the load-drift response of specimen SC3, with a 32 in. shear capital, and specimen SC1, with a 40 in. shear capital, shown in Figs. 3.14 and 3.15, respectively. At drift levels greater than 4.0 percent, pinching was observed in the load-drift response of specimens SC1 and SC5 in the region of zero drift. This was attributed to the opening and closing of torsional cracks at the slab discontinuous edge. In load-drift response of specimen SC3, however, this effect was not observed. Instead, the Bauschinger effect was observed in the unloading part of the load-drift curves. This shows that the response of specimen SC3 was governed by the behavior of the slab rebar.

3.4 STRENGTH

The strength envelopes of interior and edge connection specimens are shown in Figs. 3.17 and 3.18, respectively. The envelope is obtained by connecting the peak load points of displacement cycles in both positive and negative directions. While comparing the strength of connections with and without shear capitals it is noted that the presence of shear capitals resulted in a significant increase in the strength of the slab-column connection.
3.4.1 INTERIOR CONNECTIONS

A difference in the size of shear capital had little effect on the strength of interior connections during drift cycles less than 3 percent. Interior connections with shear capitals were about 50 percent stronger than specimen PI8, an interior connection without a shear capital, at about 0.75 percent drift and became increasingly stronger than specimen PI8 as the drift increased. At 3 percent drift interior connections with shear capitals were 80 percent stronger than specimen PI8.

By about 3.5 percent drift, specimen PI8 reached its largest strength of 8.8 kips and began to experience punching shear failure. At 5 percent drift it was able resist only about 60 percent of its largest strength. The highest strength of specimen SC6, with a 24 in. shear capital, was about 90 percent larger than that of specimen PI8. This specimen reached its largest strength and began punching shear failure at about 4 percent drift. The highest strength of specimen SC4, with a 32 in. shear capital, was about 100 percent larger than that of specimen PI8 and occurred at about 6 percent drift and was maintained at 7 percent drift. The highest strength of Specimen SC2, with a 40 in. shear capital, was about 130 percent larger than that of specimen PI8 and also occurred at 6 and 7 percent drift.
3.4.2 EDGE CONNECTIONS

The peak strength in negative bending of edge specimen PE9, with no shear capital, was over 50 percent higher than the peak strength in positive bending. This is attributed to two reasons. First, more load was applied to the specimen in negative bending due to gravity loads. Second, more top steel in the slab provided higher strength in negative bending.

The presence of a shear capital increased the strength of edge slab-column connections as shown in Fig. 3.18. Edge connections with shear capitals which suffered large torsional cracks in the slab discontinuous edge, namely specimens SC1 and SC5, were much weaker in positive bending than in negative bending. If the stirrups at the edge of the slab had been continued into the shear capital the strength of specimens SC1 and SC5 in positive bending would be similar to the strength in negative bending. The strength in positive bending of specimen SC3, which suffered damage due predominantly to flexural cracks, was similar in magnitude to its strength in negative bending. The shear capital in this specimen remained undamaged throughout the loading routine.

3.5 STIFFNESS

Several definitions are possible to estimate the
stiffness of slab-column connection subassemblies on the basis of observed load-deformation response. Some of these include loading, unloading, reloading and zero-displacement stiffness. Only the peak-to-peak stiffness will be used in this discussion. The summation of the two peak loads of a displacement cycle divided by the summation of the displacements at which they occur is defined as the peak to peak stiffness for that drift cycle. In other words, it is the slope of the line joining the peak load points in a given loading cycle. The definition of this stiffness in relation to a load-deformation cycle is shown in Fig. 3.19.

The peak-to-peak stiffness of interior and edge connections are plotted against drift in Figs. 3.20 and 3.21, respectively. The test results show that about an 80 percent increase in slab depth near the column produced a significant increase in the initial peak to peak stiffness to both interior and edge column connections. It can be seen from the figures that an increase in the length of the shear capital did not significantly increase the initial stiffness. This was to be expected since the depth of the shear capital was the same in all tests. An increase in depth caused an increase in the moment of inertia of the slab which increased the stiffness of the connection.

The degradation of stiffness with drift for interior and edge column connections are shown in Figs. 3.22 and
3.23, respectively. For easy comparison, the stiffness is normalized with respect to initial stiffness. It is observed from these plots that edge connections lose their stiffness at lower drift levels than interior connections. At 1 percent drift all interior connections maintained at least 80 percent of their stiffness; whereas, none of the edge connections maintained more than 80 percent of their initial stiffness. At 2 percent drift none of the edge connections maintained 60 percent of their initial stiffness whereas all of the interior connections maintained at least 60 percent of their initial stiffness.

3.5.1 INTERIOR CONNECTIONS

The initial stiffness of interior connection specimens with shear capitals was about 50 percent larger than that of specimen PI8, which had no shear capital. From comparing Fig. 3.20 and 3.22 it can be seen that shear capitals were more effective in increasing the stiffness of the connection than reducing the amount of stiffness degradation. The rate of stiffness degradation was approximately the same for all specimens until about 1.5 percent drift as shown in Fig. 3.22. At about 2 percent drift the effect of the size of shear capital on the stiffness degradation became evident. At drifts ranging
from 2 until 6 percent a 30 percent increase in size of shear capital reduced the stiffness degradation by 15 percent.

3.5.2 EDGE CONNECTIONS

The initial stiffness of edge connection specimens with shear capitals were about 30 percent larger than that of specimen PE9, with no shear capital, as shown in Fig. 3.21. The rate of stiffness degradation was very high at drift less than 1 percent due to torsional cracking of the slab discontinuous edge. Specimen SC3, with a 32 in. shear capital, had a smaller initial rate of stiffness degradation since it did not experience significant torsional cracks. From observing Fig. 3.23 it is noted that the presence of a shear capital reduces the amount of stiffness degradation at all drift levels. At about 3 percent drift an increase in peak-to-peak stiffness with an increase in shear capital size can be seen clearly in Fig. 3.21. The presence of a shear capital reduced the amount of stiffness degradation at all drift levels. An increase in size of shear capital did not significantly decrease the rate of stiffness degradation with increased drift.

3.6 MOMENT-ROTATION RESPONSE

As opposed to the overall response of the specimen the moment-rotation plots show the interaction between the slab
and the column. Moment vs. rotation plots for each test specimen can be found in Figs. 3.24 to 3.32. The moment is calculated at the center of the connection region and the rotation represents the total rotation of the slab with respect to the column over an 8 in. distance from the column face. The rotation was measured by LVDTs placed on both the top and bottom of the slab attached to the column.

The region of the slab at the junction with the column is the most likely place for failure in a flat slab structure because it is the most highly stressed region. A shear capital provides additional stiffness to this region of the slab for the purpose of preventing failure. The moment rotation plots are used to examine the integrity of the slab-column junction compared to the integrity of the overall specimen given in the load-drift plots. This information helps judge the effectiveness of the shear capital to prevent failure damage at the slab-column junction.

A large rotation infers that the region where the slab meets the column has lost its stiffness. If one specimen has a higher rotation than another at the same drift then it has more damage to the region of the slab near the column. If the drift is very high and the
rotation in this region is still quite low then an area of the slab further away from the column than where the rotation has been measured has lost its stiffness.

3.6.1 INTERIOR CONNECTIONS

A punching shear failure in the slab is associated with a large rotation of column with respect to slab. Specimen PI8, with no shear capital, and specimen SC6, with a 24 in. shear capital, experienced punching shear failure. As shown in Figs. 3.24 and 3.25, both of these specimens experienced excessive rotations at drifts which correspond to their punching failure. Specimen SC4, with a 32 in. shear capital, and SC2, with a 40 in. shear capital, did not experience punching shear failure. In these two specimens, the rotation the slab with respect to the column was small compared to that of specimens PI8 and SC6.

The moment-rotation plot for specimen PI8, shown in Fig. 3.24, indicates a sudden increase in rotation during the 5 percent drift cycle. As a result of punching failure in this specimen the slab dropped downward with respect to its original position. The moment-rotation curve for specimen SC6, given in Fig. 3.25, shows that the center of rotation for the last few drift cycles shifted towards negative rotation. This indicates that shear cracks which formed when the bottom of the slab was in tension opened very wide and caused permanent rotation of the slab with
respect to the column. The moment-rotation curve for specimen SC4, given in Fig. 3.26, indicates very little rotation of slab within a distance of 8 inches from the column face. This suggests that most of the rotation in the slab occurred outside the shear capital in this specimen. Therefore, the shear capital provided sufficient strength to the joint between the column and slab to prevent damage such as a punching shear failure from occurring. The rotation within the 40 in. shear capital in specimen SC2 was also very small, as shown in Fig. 3.27. This suggests a flexural hinging mechanism outside the shear capital region.

3.6.2 Edge Connections

A large rotation of the slab with respect to the column in edge connections is associated with torsional damage to the slab discontinuous edge. As shown in Table 3.02, specimen SC3, with a 32 in. shear capital, experienced a flexural failure mode and had a peak rotation not greater than 0.02 radians. Other edge connections experienced torsional damage and had peak rotations much greater than 0.02 radians.

The moment-rotation response of specimen PE9 shows that large rotation of the slab with respect to the column began at small drift. Comparing the moment-rotation plot
for specimen SC5, given in Fig. 3.29, to that of specimen PE9, shown in Fig. 3.28, it is noted that the presence of a shear capital allowed less rotation between the slab and the column and gave the connection greater strength. At drift cycles of 4 percent or larger the rotation near the column became large and more rotation occurred when the slab was in negative bending. The rotation of the slab relative to the column in specimen SC3 remained small throughout the loading routine, as shown in Fig. 3.29. This indicates that rotation did not predominantly occur in a region of the slab adjacent to the column. The rotation of the slab relative to the column for specimen SC1 with a 40 in. shear capital, shown in Fig. 3.31, was much larger than that for specimen SC3 for drift greater than about 3 percent. At 7 percent drift the maximum rotation was twice as large in negative bending than in positive bending.

3.7 ENERGY DISSIPATION

The ability to dissipate energy without significant loss of strength or stiffness is considered an important parameter in evaluating the response of slab-column connection subassemblies. The area enclosed by the load-deformation loop during a given displacement cycle represents the energy dissipated in the subassembly during that cycle. A comparison of the energy dissipated at
different drift cycles for interior and edge column connections is shown in Figs. 3.35 and 3.36, respectively.

It can be seen that the presence of a shear capital significantly improved the energy dissipation of both interior and edge column connections at large drifts. At drifts smaller than 3 percent, the increase of energy dissipation due to the presence of a shear capital is negligible. At drift levels of about 4 percent or greater the energy dissipation in specimens with shear capitals is almost twice that of specimens without shear capitals.

3.7.1 INTERIOR CONNECTIONS

The energy dissipation in interior connections with shear capitals was about the same at drifts up to 4 percent. The variation in the amount of energy dissipation was about 10 percent until 6 percent drift. Specimen SC2, with a 40 in. shear capital and specimen SC3, with a 32 in. shear capital dissipated approximately the same amount of energy at 7 percent drift whereas specimen SC6, with a 24 in. shear capital, dissipated about 20 percent less. This was caused by the reduction in strength of the connection due to punching shear failure.

Specimen PI8, with no shear capital, dissipated approximately the same amount of energy as the interior connections with shear capitals at drifts up to 3 percent. At 4 percent drift specimen PI8 was able to dissipate only
half the amount of energy dissipated by specimens with shear capitals. At 5 percent drift it was able to dissipate only a third of the energy dissipated by specimens with shear capitals. The large difference in the amount of energy dissipated was due to a punching failure.

3.7.2 EDGE CONNECTIONS

Edge Connections with shear capitals dissipated approximately the same amount of energy at all drifts. Specimen PE9, with no shear capital, dissipated about the same amount of energy as edge connections with shear capitals at drifts up to 3 percent. It was able to dissipate 70 percent of the energy dissipated by connections with shear capitals at 4 percent drift and 50 percent at 7 percent drift. Although specimen PE9 was not able to dissipate as much energy as specimens with shear capitals, it did not experience punching shear failure. The additional strength to an edge connection provided by a shear capital allows it to dissipate more energy.

3.8 STRAIN GAGE RESULTS

Strain gages provide qualitative information about the state of stress at discrete locations in rebar. In order to determine the necessity of reinforcing shear capitals, strain gages were installed on rebar within the shear capital. Plots of strain vs. drift at different locations
in rebar placed in shear capitals of interior and edge connections are shown in Fig. 3.37 to 3.48. It is to be noted that, according to the information given in the rebar tension tests, the slab rebar yielded at approximately 2800 micro-strain.

As observed from the strain vs. drift plots, yielding occurred in the vertical legs as well as the horizontal legs of the stain gage reinforcement. This shows that shear capitals should be properly anchored into the slab by vertical reinforcement at locations where the slab changes thickness. It is concluded that it is necessary to reinforce shear capitals subjected to load reversals, such as those encountered in the tests, are expected.

3.8.1 INTERIOR CONNECTIONS

The severe damage to the region of the slab around the 24 in. shear capital in specimen SC6 at large drift levels is evident from the strain history shown in Fig. 3.37. The vertical legs of the shear capital reinforcement experienced yielding at about 2 percent drift. At larger drifts, as the slab separated from the shear capital, the vertical legs of the shear capital reinforcement were no longer effective. The horizontal legs of the shear capital reinforcement also yielded at about 2 percent drift. The strain history in Fig. 3.38 is shown only for drifts less
than 2 percent due to off scale readings at higher drifts. This shows that there were large flexural stresses at the face of the column before punching failure occurred around the shear capital.

The vertical legs of the shear capital reinforcement for specimens with 32 in. and 40 in. shear capitals were effective and experienced strain below yielding throughout their loading routine as shown in Figs. 3.39 and 3.41, respectively. The horizontal legs of the these two specimens experienced strain exceeding or near yielding as shown in Figs. 3.40 and 3.42. It is to be noted that the flexural hinging failure occurred outside the shear capital in both of these specimens.

3.8.2 EDGE CONNECTIONS

The vertical leg of the 24 in. edge shear capital reinforcement experienced large strain levels which increased until 6 percent drift as shown in Fig. 3.43. At this point the strain reduced significantly due to cracks at the face of the shear capital which caused slippage. The vertical legs of the 32 in. and 40 in. shear capitals, however, experienced strain below yielding and showed no evidence of slippage as shown in Figs. 3.45 and 3.47, respectively.

In all edge connections with shear capitals, yielding was reached in the horizontal legs of the shear capital
reinforcement. The strain in the horizontal legs of the 24 in. shear capital reinforcement significantly decreased at about 3 percent drift due to a redistribution of stress caused by cracking as shown in Fig. 3.44. At about 6 percent drift the strain in the horizontal legs of the 32 in. shear capital reinforcement decreased significantly due to flexural yielding at the shear capital face as shown in Fig. 3.46. The strain in the horizontal legs of the 40 in. shear capital reinforcement increased throughout the loading routine as shown in Fig. 3.48.
CHAPTER 4

SHEAR STRENGTH OF INTERIOR CONNECTIONS WITH SHEAR CAPITALS

4.1 GENERAL REMARKS

In an interior slab column connection the net shear force at the center of the connection is primarily caused by gravity loads rather than lateral loads such as wind or earthquakes. The net shear force at the center of the connection caused by lateral loads is small since components from the slab at opposite faces of the column counteract each other. The unbalanced moment at the connection is primarily caused by lateral loads. If the lateral loads produce moments at the joint which are less than or equal to the moments caused by gravity loads and if gravity loads produce equal moments at opposite column faces, such as the case shown in Fig. 4.1 (a), then the unbalanced moment is negligible and does not effect the shear design of the connection. When lateral loads produce moments at the connection which are greater than those produced by the gravity loads, such as the case shown in Fig. 4.1 (b), then the unbalanced moment is significant and should be considered in the shear design of the connection.

The main purpose of using a shear capital is to
increase the shear strength of slab-column connections under gravity and lateral loads. If failure occurs because of insufficient flexural capacity of the slab in a region outside of the shear cap; thus, away from the column, the connection is no longer the weakest region and the shear cap has served its purpose. The added shear capacity provided by a shear cap may make the portion of the slab near the column strong enough to cause a shear failure to occur outside the shear cap region. The presence of a shear cap may also change the mode of failure of the connection from shear to torsion or flexure.

The flexural capacity of the slab can also be increased by the presence of a shear cap. It is evident that the increase in slab thickness provides added moment capacity of the slab at the face of the column. The stiffness provided by the shear cap in the transverse direction to the lateral load may also increase the participation of the slab in resisting the bending moment thus resulting in a more even distribution across the slab.

4.2 PRESENT CODE PROVISIONS

Prior to the ACI 318-89 Building Code the critical section for punching shear failure of a slab-column connection was assumed to be located a distance $d/2$ from the column face only. If a flat plate structure were to
fail because of gravity load only then this would be the region of the slab most likely to experience a punching shear failure. This critical section along with the equations used in calculating $A_c$, the area of the critical section, and $J_c$, a property related to the polar moment of inertia of the critical section, are shown in Fig. 4.02.

For slabs with a uniform thickness, the present code considers it sufficient to check its shear capacity only at this critical section. In slabs with changes in thickness such as a shear capital, however, section 11.12.1.2 of the code suggests that shear should be checked at other possible critical sections which include $d/2$ from any change in thickness of the slab as shown in Fig. 4.03. These are the critical section of the slab which are most likely to fail due only to gravity loads which do not produce significant unbalanced moment at the connection. When the connection is subjected to this type of loading there is tension in the top of the slab at all faces of the column.

The eccentric shear stress model for calculating shear stresses at critical sections used by the ACI code is based on a model developed by Di Stasio and Van Buren (ref. 3). In this model the shear stress resulting from moment transfer by eccentricity of shear is assumed to vary linearly about the centroid of the critical section. Only
a fraction of the unbalanced moment at the centroid of the
critical section, \( \gamma_v M_u \), is assumed to be transferred by
eccentricity of shear. This fraction was determined
experimentally by Hanson and Hanson (ref. 5) to be 0.4 for
square interior columns.

The maximum shear stress on the critical section of a
slab-column connection subjected to a direct shear force,
\( V_u \), and an unbalanced moment, \( M_u \), is the larger of the
following:

\[
v_u = \frac{V_u}{A_c} + \frac{\gamma_v M_u c_{AB}}{J_c}
\]

or

\[
v_u = \frac{V_u}{A_c} - \frac{\gamma_v M_u c_{CD}}{J_c}
\]

Where,

\( A_c \) = area of critical section

\( J_c \) = polar moment of inertia of critical section

\[
\gamma_v = 1 - \frac{1}{\left( 1 + \frac{2}{3} \sqrt{b_1/b_2} \right)}
\]

\( b_1 \) = width of the critical section measured

in the direction of the span for which

moments are determined

\( b_2 \) = width of the critical section measured

in the direction perpendicular to \( b_1 \)
The ACI Code limits the shear stress on the critical section to \(4 \sqrt{f_c'}\) for square columns. When rectangular columns are used with the ratio of the long and short sides larger than 2.0 tests (ref. 23) indicate that this is not a conservative value for the shear stress. In such cases the ACI code limits the shear stress to \((2 + 4/\beta_c) \sqrt{f_c'}\) where \(\beta_c\) is the ratio of long to short side of the column, concentrated load or reaction area.

Several researchers (refs. 4, 15, 16) have indicated that the size of the column with respect to the slab depth, or the \(r/d\) ratio, has an effect on the overall shear strength of the connection. As \(r\) gets larger with respect to the effective slab depth the amount of shear stress that the surrounding slab is able to resist decreases. In other words, as the critical section becomes larger the shear stress capacity of the section decreases. The current code suggests that the shear stress on the critical section, \(\nu_c\), decreases as the ratio of the critical perimeter to the effective depth of the slab, \(b_o/d\), increases. This is implied in equation (11-37) of the ACI 318-89 Building Code:

\[
\nu_c = \left( \frac{\alpha_d}{b_o} + 2 \right) \sqrt{f_c'} b_o d
\]
Where \( \alpha \), is 40 for interior columns, 30 for edge columns and 20 for corner columns. The tests from which this equation was developed, however, varied only the size of the column with respect to the slab depth and did not use drop panels or shear capitals.

4.3 COMPARISON WITH ACI APPROACH

The measured response of each interior connection specimen, identified by its code number and shear capital size, is presented in Table 4.01. In column (3) the concrete compressive strength of each specimen at the time of the test is presented. Columns (4) and (5) contain the peak loads measured at the top of the column of the specimens and the drift at which they occur. It is noted that there was a significant difference in strength of the specimen without a shear capital and those which had shear capitals. An increase in shear capital size had a less dramatic effect on the strength of the connection.

The unbalanced moment at the center of the connection region, \( M_u \), is presented in column (6). Moments calculated at the faces of the shear capitals, presented in columns (7) and (8), are calculated using the reactions at the free end of the slab, the applied gravity loads, and the unbalanced moment. At the edge of the shear capital,
\((M_u)_{sf}\) is the negative moment in the slab and, \((M_u)_{gh}\), is the positive moment in the slab. Column (9) shows the net shear force at the center of the column, \(V_w\), which was calculated from the measured reactions of the specimen. A negative net shear force corresponds to uplifting of the column.

Table 4.02 presents the results of an analysis of the interior connection specimens using the eccentric shear stress model presented in the ACI 318-89 Building Code. Columns (11) and (12) show the flexural capacity of slab strips \(c2 + 3h\) wide, as suggested by the code, centered on planes located outside the column and shear capital, respectively.

The results for the analysis using the eccentric shear stress model with the critical section located \(d/2\) from the column face are shown in columns (13) through (15). Columns (13) and (14) give ratios of applied shear stress to shear stress capacity on planes \(a-b\) and \(c-d\), as shown in Fig. 4.03, respectively. The shear capacity of the critical section was taken as the lesser of Equations (11-36), (11-37) and (11-38) given in the ACI 318-89 Building Code. Since all columns were square, Equation (11-36) never governed. At critical sections near the column Equation (11-38) governed the shear capacity. It should be noted that specimen PI8 failed due to punching
shear failure as predicted by the ACI code.

The ratio of the portion of the unbalanced moment, \( \gamma_f M_u = (1 - \gamma_v) M_u \) or 0.6\( M_u \) for square columns, to the flexural capacity of the slab near the column given in column (11) is shown in column (15). If this ratio is greater than 1.0 then flexural capacity of the slab to transfer unbalanced moment to the column, according to the ACI code, has been exceeded. None of the ratios in column (15) are greater than 1.0. This is in agreement with the test results since there was no evidence of flexural failure in this region of the slab in any of the test specimens.

Columns (16) through (18) present the result of the analysis using the eccentric shear stress model with the critical section located d/2 from the shear capital face. The ratios of applied shear stress to the shear stress capacity at sections e-f and g-h, as shown in Fig. 4.03, are given in columns (16) and (17), respectively. It should be noted that the ACI model fails to predict the punching shear failure of specimen SC6.

The ratio of the portion of the unbalanced moment transferred by flexure to the flexural capacity of the slab near the face of the shear capital, given in column (12), is presented in column (18). The data presented in column (18) indicates that the ACI model considerably
underestimates the capacity of the slab outside of the shear capital, to transfer unbalanced moment. It is suggested that the slab width effective in transferring unbalanced moment to the column should be increased from $c_2 + 2h$ to $l_{sc} + 3h$ where $l_{sc}$ is the length of the shear capital transverse to the direction of lateral loading. The strain gage distribution on longitudinal steel gives evidence of a larger effective width of the slab.

4.4 DISCUSSION OF ACI APPROACH

A shear failure will initially occur in a slab on a plane which is the first to be subjected to stress beyond its capacity. This plane does not necessarily have to be the weakest plane because the stress on that plane may not be beyond its capacity. A plane which is subjected to the highest stress may not fail first since it may have a high capacity which may not be reached before a weaker plane reaches its capacity. This becomes important for a slab with a shear capital because sections of the slab at various distances from the column can have different shear capacities as well as different imposed shear stresses.

The critical sections suggested by the ACI Code imply that failure will occur on a plane centered a distance $d/2$ away from either the face of the column or the face of the shear capital. In Fig. 4.04 these planes are shown as
sections a' - a and b' - b, respectively. When gravity load moments are dominant, as shown in Fig. 4.01 (a), one of these two planes is the most likely to fail first. In other words, the critical sections suggested by the ACI code are sufficient. When moments caused by lateral loads are dominant, however, it is possible that a plane through section c' - c, as shown in Fig. 4.04, could be the first to reach its shear capacity. In this case positive moment at the column face causes cracks on the bottom of the slab at the intersection of the shear capital and the slab. This shear crack will propagate diagonally upward towards the column. The resulting punching shear failure surface is inverted compared to that of punching shear failures which originating at sections a' - a and b' - b. If this happens the stresses which cause shear failure in the slab would be completely different from those predicted by the ACI Code.

An examination of the possible locations of a punching shear failure of specimen SC6, which had a 24 in. shear capital, is presented in Fig. 4.05. At each location the eccentric shear stress model of the ACI 318-89 Building Code is used to analyze the shear stress applied to the section. The critical sections which the code suggests should be checked are examined in Figs 4.05 (a) and 4.05 (b) and are the same as those which represent planes a' - a and b' - b in Fig. 4.04, respectively. The maximum shear
stress on these two planes are about half of the slab shear stress capacity as shown in Table 4.02. In other words, according to the ACI 318-89 Building Code this specimen should not experience punching shear failure when subjected to the loads applied during its test. Fig. 4.05 (c) presents the results of eccentric shear stress model used on a critical section analogous to plane $c' - c$ in Fig. 4.04. The critical section was taken $d/2$ from the column face using the effective depth of the slab without the shear capital. On this plane the applied shear stress exceeds the shear stress capacity of the slab causing a failure which was observed during the test. Since the ACI Code fails to predict shear failure on this plane under these load conditions it is not conservative.

When a flat-plate or flat-slab structure is subjected exclusively to gravity loads which do not cause significant unbalanced moment at the connection region a classic punching failure can occur in which the column and attached portion of the slab push through the slab all at once. In this case the shear stress is equal at any point which is the same distance from the column faces and the failure surface takes the shape of a truncated pyramid. The ACI code defines a critical section which is equidistant from all sides of and is centered on the column. If the same structure is subjected to high lateral loads as well as
gravity loads the concept of a critical perimeter is not relevant. The shear capacity of a connection is dependent upon the shear strength region of the slab which is the first to be overstressed not the shear strength of an area of concrete around the whole perimeter of the column. The concept of a critical perimeter can be particularly misleading in the case of a flat-slab structure with shear capitals where the thickness, hence, the shear capacity, of the slab changes.

4.5 OPTIMUM SIZE OF SHEAR CAPITAL

Tests (ref. 6 and 17) of slab-column connections have shown that the average angle of a punching shear failure surface relative to horizontal is usually between 25 and 30 degrees. Since the shear capacity of the slab along this surface is critical, any increase in the thickness of the slab in this region will definitely increase the shear capacity of the connection. The increase in slab thickness will only be effective if it is throughout the region of the slab which includes the failure surface. Therefore, the ratio of the thickness of the main slab to the length of the shear capital from the face of the column, h/l, must be greater than or equal to the tangent of the angle of the punching shear surface to the horizontal, θ. If \( \tan \theta = 0.5 \), then \( l > 2h \), as shown in Fig. 4.06. For an interior column
connection the total length of the shear capital should be at least \( L = 4h + c \), where \( c \) is the column width.

The punching shear failure surface observed in specimen SC6, with a 24 in. shear capital, formed at an angle approximately 30 degrees relative to horizontal. For this specimen the added thickness of the slab provided by the shear capital was not effective in providing shear strength in the plane at which it failed. The presence of the shear capital did not move the shear failure away from the column.

For the test specimens in this investigation the thickness of the main slab was 4.5 in. and the width of the column was 10 inches. In order to prevent reverse punching failure due to excessive earthquake type loads, the length of the shear capital, according to the \( L = 4h + c \) criteria, should be 28 inches. The test data supports this criteria. The interior connection with a 24 in. shear capital experienced a punching shear failure whereas the interior connections with 32 in. and 40 in. shear capitals did not.
CHAPTER 5

SHEAR STRENGTH OF EDGE CONNECTIONS
WITH SHEAR CAPITALS

5.1 GENERAL REMARKS

The behavior of edge slab-column connections with
shear capitals subjected to earthquake type loading is
presented in this chapter. The discussion is limited to
bending about an axis parallel to the slab discontinuous
edge. The test results are reviewed with reference to the
present ACI Code (ref. 2) provisions for transfer of
unbalanced moment and direct shear at the slab-column
connection. Based on the observed behavior of connections,
a design procedure that more realistically reflects the
observed response is suggested.

The behavior of an edge slab-column connection is
inherently different from that of an interior slab-column
connection. Due to the lack of continuity at the edge
column, the transfer of moment and shear is only possible
through three faces of the column. The transfer of moment
in the slab through the column faces normal to the slab
discontinuous edge is dependent upon the torsional strength
of the slab edge. Due to the lack of confinement at the edge column, the slab is susceptible to significant damage from shear and torsion.

5.2 OBSERVED RESPONSE

Table 5.01 summarizes the test results at peak load for all edge connection specimens. Each test specimen is identified by its code name and the size of its shear capital. The peak loads obtained throughout the entire displacement routine of each specimen are presented in column (4). Negative and positive loads correspond to loads causing tension and compression on the top of the slab, respectively. The percentages of lateral drift at which these loads occur are presented in column (5). It is observed that the edge connections were stronger in negative bending than in positive bending. This is attributed to the fact that there was more rebar on the top of the slab. Edge connections with shear capitals which experienced torsional damage, namely specimens SC1 and SC5, were much weaker in positive bending than in negative bending. This is attributed to the type of stirrups used within the shear capital at the slab discontinuous edge. Stirrups only enclosed the slab rebar and did not extend into the shear capital. As a result, once the concrete cracked on the bottom of the shear capital at the slab
discontinuous edge no further torsional strength was available.

The unbalanced moment was measured at the center of the slab-column joint. The unbalanced moment at other locations were calculated from the reaction at the free end of the slab, the applied gravity loads, and the unbalanced moment at the center of the slab-column joint. Column (6) gives the unbalanced moment, \((M_u)_{c'}\), at the centroid of the critical section when located a distance \(d/2\) from the column faces. The unbalanced moment, \((M_u)_{sc'}\), at the centroid of the critical section located a distance \(d/2\) from the faces of the shear capital, is presented in column (7). The total direct shear force at peak loads are presented in column (8). A negative shear force corresponds to the uplifting of the column.

5.3 PRESENT CODE PROVISIONS

The ACI 318 Building Code has traditionally used the same eccentric shear stress model developed for interior connections subjected to combined shear and unbalanced moment to predict the shear stresses around edge connections. The eccentric shear stress model applied to an edge connection subjected to bending about an axis parallel to the exterior face is shown in Fig. 5.01. This model assumes a three-sided critical perimeter, \(b_0\), which
is located a distance $d/2$ from all faces of the column. The surface area on which the direct shear, $V_u$, is acting is equal to the critical perimeter times the effective depth of the slab. The shear stress produced by a portion of the unbalanced moment is assumed to vary linearly within the critical section in the direction of lateral load. A property of the critical section related to the polar moment of inertia, $J_c$, is taken about the centroid of the critical section which is different than the centroid of the column in an edge connection. The ACI 318-89 Code states that the critical section can occur at the face of the shear capital as well as the column.

Table 5.02 presents the results of the analysis of edge column connection specimens using the ACI Code. When two data appear in one block the top number corresponds to the top of the slab in compression. The number on the bottom corresponds to the top of the slab in tension. The nominal flexural capacities of the slab at the column face and the face of the shear capital are shown in columns (11) and (12), respectively.

5.4 DISCUSSION OF ACI APPROACH

The ACI Code uses an empirical model to describe the transfer of unbalanced moment and shear which was developed from test results (ref. 5) of interior slab-column
connections. The same proportions of unbalanced moment transferred by eccentric shear and flexure are not necessarily valid for edge slab-column connections. The ACI Code overestimates the ability of the connection to transfer a portion of unbalanced moment by flexure through a region of the slab adjacent to the column, as shown in column (15) in Table 5.02. For instance, specimen PE9, with no shear capital, suffered large flexural and torsional damage in the region of the slab adjacent to the column. The ACI Code analysis indicates that only 80 percent of its moment transfer capacity was reached at peak load. Specimen SC1, with a 40 in. shear capital, also experienced large torsional and flexural damage adjacent to the column. The ACI Code analysis indicates that only 70 percent of its moment transfer capacity was reached at peak load. The ACI Code also considerably underestimates the amount of moment which can be transferred by a portion of the slab outside the shear capital. For instance, Specimen SC3, with a 32 in. shear capital, was able to transfer 75 percent more moment at peak load than predicted by the ACI analysis.

An edge slab-column connection has only one face of the column through which a portion of the unbalanced moment can be transferred directly by flexure. The remaining portion of the unbalanced moment must be transferred
indirectly through the other two faces by torsion. The linear variation of eccentric shear along the sides of the critical section parallel to the direction of unbalanced moment given in the eccentric shear stress model of the ACI code is the result of torsion transmitted to the column by the edge beams. Torsion on the faces of the column parallel to the direction of unbalanced moment is an indirect mechanism by which unbalanced moment is transferred in a slab-column connection.

The ACI model fails to take into account the torsional capacity of the portion of the slab which intersects the sides of the column parallel to the direction of unbalanced moment. Many tests of edge slab-column connections subjected to loads which cause moments about an axis parallel to the free edge (ref. 9) have shown that the torsional capacity of the edge region can govern the design. Yet, the ACI Code requires that only flexural capacity and shear capacity be sufficient for the transfer of shear and unbalanced moment.

The ACI-ASCE Committee 352 recently made recommendations (ref. 14) for the design of slab-column connections. These recommendations suggested an alternate procedure for the design of edge slab-column connections. A typical torsional yield line in an edge slab-column connection, shown in Fig. 5.02, have a projection of
approximately 45 degrees with respect to the column faces perpendicular to the axis of unbalanced moment. The width of the slab effective in transferring unbalanced moment by flexure in edge slab-column connections was recommended to be changed from \( c_2 + 3h \) to \( c_2 + 2c_1 \), where \( c_2 \) is the dimension of the column face parallel to the axis of unbalanced moment, \( c_1 \) is the dimension of the column perpendicular to the axis of unbalanced moment and \( h \) is the thickness of the slab.

5.5 ALTERNATIVE APPROACH

In order to model the behavior of an edge slab-column connection the torsional capacity of the edge beam in the slab should be taken into account. The size of the edge beam and the amount of torsional reinforcement must be taken into account when the amount of unbalanced moment transferred to the column by torsion is determined. The peak load results for edge connections are compared to an analysis of the test specimens using the eccentric shear stress model considering the torsional capacity of the slab edge in Table 5.03.

The nominal torsional moment strength given by Eqn. (11-21) of the ACI Code, \( T_n = T_c + T_s \), was used to determine the torsional capacity of the edge of the slab of each test specimens. The torsional capacity of the slab edge inside
the shear capital on one side of the column, \((T_n)_{c'}\) is shown in column (20). Column (24) shows the torsional capacity of the slab edge outside the shear capital. The torsional capacity was found to be almost twice as large inside the shear capital than outside the shear capital.

The capacity of the slab to transfer unbalanced moment to the column by flexure, \(M_{f}\), is determined by calculating the flexural capacity of slab within an effective width \(c_2 + 2c_1\) centered on the column, where \(c_2\) is the width of the column parallel to the direction of load and \(c_1\) is the width of the column perpendicular to the direction of load. This flexural capacity, \((M_f)_{c'}\), was calculated for each edge connection and shown in column (21). If a shear capital is used the capacity of the slab to transfer unbalanced moment to the column may be limited by the flexural capacity outside the shear capital. This capacity, \((M_f)_{sc}\), is determined by calculating the flexural capacity of slab for an effective width of \(L_2_{sc} + L_1_{sc}\) centered on the column where \(L_2_{sc}\) and \(L_1_{sc}\) are the widths of the shear capital parallel and perpendicular to the slab edge, respectively, and is shown in column (25).

The summation of the torsional capacity of the edge beam on each side of the column and the capacity of the slab to transfer unbalanced moment by flexure is the unbalanced moment capacity of the connection, \(M_n\). In
order to prevent a combined flexural and torsional failure of the connection this capacity must always be greater than the unbalanced moment caused by the design loads.

\[ M_{ub} < M_n = M_f + 2T_n \]

The ratio of the measured unbalanced moment to the unbalanced moment capacity at the face of the column and at the border of the shear capital, \((M_{ub})_c/(M_{ub})_c\) and \((M_{ub})_{sc}/(M_{n})_{sc}\), are given in columns (22) and (26), respectively.

Two of the edge connections, specimen PE9 and SC1, exceeded their ability to transfer unbalanced moment at the face of the column. As shown in column (22) of table 5.03, the ability of specimen PE9, with no shear capital, to transfer unbalanced moment by flexure and torsion was exceeded by 7 percent at peak load under negative bending at the connection. This failure was observed during the test of specimen PE9. Specimen SC1, with a 40 in. Shear capital, exceeded its ability to transfer unbalanced moment inside the shear capital by 4 percent and outside the shear capital by 15 percent at peak load under negative bending at the connection. During the test of specimen SC1 a sudden change of slope of the deflected shape at the edge of the shear capital was observed at high drifts. At peak load torsional cracks at the slab discontinuous edge inside the shear capital opened wide and flexural hinging was
observed at the border of the shear capital.

The shear capacity of an edge slab-column connection was also calculated and compared to the applied shear stress on the connection using the modified eccentric shear stress model considering the torsional capacity of the slab discontinuous edge. It is possible that a punching shear failure may occur in the connection region before its capacity to transfer unbalanced moment by flexure and torsion is reached. Shear forces, which are transferred directly from the slab to the column, and torsional moments, which are transferred to the column through the edge beams, cause stresses that are highest in the region of the slab around the faces of the column or the edges of the shear capital. The ultimate shear stress that the slab can sustain must be less than the sum of the direct shear stress and the shear stress due to torsion transferred through the edge beams. A modified version of the ACI Code's eccentric shear stress model can be used to check the shear stresses in the connection region.

\[ u_u < u_n = \frac{V_u}{A_c} \pm \frac{T_u c}{J_e} \]

Punching shear failure will initiate in the slab on a plane which the shear capacity has been reached. When shear capitals are used the plane with the highest shear stresses will not necessarily reach its shear capacity
first. The weakest plane, such as that of the main slab outside the shear capital edge, may not be the first to reach its shear capacity since it is far enough away from the connection region that the stresses are smaller. Ratios of the applied shear stress on the connection at peak load to the shear stress capacity measured at critical sections d/2 from the column and d/2 from the shear capital, \((v_u)_c/(v_n)_c\) and \((v_u)_sc/(v_n)_sc\), are given in columns (23) and (27) of Table 5.03, respectively. The analysis shows that none of the edge connection specimens reached more than 60 percent of their shear capacity at peak loads. This supports the test data in which there was no indication of punching shear failure in the edge connection specimens.

It was observed from the test results that improvement of the strength, stiffness and ductility of edge connections by the presence of a shear capital was limited by the torsional capacity of the slab discontinuous edge as well as the flexural capacity of the slab. The overall response of the connection was improved by an increase in shear capital size. It is suggested that the improvement of the response would be more significant if closed loop stirrups at the edge of the slab extended into the shear capital and enclosed the transverse shear capital
reinforcement. A designer should be able to use the torsional capacity provided by stirrups to increase the strength of the connection.
CHAPTER 6

SUMMARY AND CONCLUSIONS

Six slab-column connection subassemblies with shear capitals were tested under simulated earthquake loads. The effects on the response due to an increase in plan dimensions of the shear capital were investigated. Three were interior connections and three were edge connections. One interior connection and one edge connection was made for each different size of shear capital. They were compared to an interior connection and an edge connection, each without a shear capital, from a previous study. All of the specimens were subjected to a similar lateral displacement routine.

A detailed account of the response and mode of failure of each specimen was presented. An evaluation of the behavior of slab-column connections with shear capitals and how the response is affected by a change in size was made. The current ACI procedure for the design of slab-column connections were evaluated based on the observed and measured test results.

Based on the test results reported in this study, the following conclusions were made regarding the behavior of slab-column connections with shear capitals subjected to
earthquake-type loads.

1. The presence of a properly detailed shear capital in a slab-column connection subjected to lateral as well as gravity loads increases the strength, stiffness and energy dissipation of that connection. Improvement to the response of the connection by an increase in size of the shear capital occurs at drifts larger than 2 percent.

2. When lateral loads produce positive moments at the connection which are larger than the negative moments produced by gravity loads the length of the shear capital should not be less than four times the height of the main slab plus largest column dimension.

\[ L > 4h + c \]

Otherwise an inverted punching failure can occur in which the additional depth provided by the shear capital is not effective in increasing the shear capacity.

3. If the critical section is located near the face of the shear capital, the width of the slab effective in transferring a portion of the unbalanced moment to the column by flexure should be increased from \( c + 3h \) to \( L_{sc} + 3h \) where \( L_{sc} \) is the length of the shear capital transverse to the direction of lateral loading.

4. When the critical section of a slab-column connection is located outside the shear capital, unbalanced
moment is transferred from the slab to the shear capital predominantly by flexure. Shear stress is largest when the critical section is located near the column.

5. If lateral loads which produce positive moments at the slab-column connection are expected then flexural reinforcing steel should be provided in the shear capital.

6. In an edge connection with a shear capital, torsional reinforcement should be provided in order to ensure sufficient torsional strength of the connection after cracking. Closed loop stirrups should be provided which enclose the shear capital reinforcement as well as the slab reinforcement.

7. The initial stiffness of a slab-column connections with shear capitals does not significantly increased by an increase in shear capital size. An increase in depth of the slab due to the presence of a shear capital had a more significant effect.

8. Shear capitals should be properly anchored into the slab by vertical reinforcement at locations where the slab changes thickness. This is done in order to insure the integrity of the shear capital and prevent damage due to stress concentrations in this region.
REFERENCES


2. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary - ACI 318R-89," American Concrete Institute, Detroit, 1989.


22. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)", American Concrete Institute, Detroit, 1983.

### TABLE 2.01: Test Specimen Parameters

<table>
<thead>
<tr>
<th>SPEC. No.</th>
<th>CONNECTION TYPE</th>
<th>SHEAR CAPITAL SIZE (in)</th>
<th>$\rho_{sc}$</th>
<th>CASTING No.</th>
<th>REINF. BATCH No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI8</td>
<td>INTERIOR</td>
<td>NONE</td>
<td>N/A</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>SC6</td>
<td>INTERIOR</td>
<td>24</td>
<td>0.0052</td>
<td>3</td>
<td>1</td>
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<td>INTERIOR</td>
<td>32</td>
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<td>2</td>
<td>1</td>
</tr>
<tr>
<td>SC2</td>
<td>INTERIOR</td>
<td>40</td>
<td>0.0063</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>PE9</td>
<td>EDGE</td>
<td>NONE</td>
<td>N/A</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>SC5</td>
<td>EDGE</td>
<td>24</td>
<td>0.0052</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>SC3</td>
<td>EDGE</td>
<td>32</td>
<td>0.0059</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>SC1</td>
<td>EDGE</td>
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<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

$\rho_{sc} = \text{shear capital reinforcement ratio}$
### TABLE 2.02: Reinforcement Properties

<table>
<thead>
<tr>
<th>BATCH</th>
<th>SPECIMEN NUMBERS</th>
<th>BAR SIZE</th>
<th>AREA $A_s$ (in²)</th>
<th>MEAN $f_y$ (ksi)</th>
<th>MEAN $f_u$ (ksi)</th>
<th>ELASTIC MODULUS $E_s$(ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SC1,SC2,SC3,</td>
<td>#3 Type 2</td>
<td>0.1104</td>
<td>82.43</td>
<td>120.5</td>
<td>29 520</td>
</tr>
<tr>
<td>1</td>
<td>SC4,SC5,SC6</td>
<td>#2 smooth</td>
<td>0.0491</td>
<td>46.15</td>
<td>57.0</td>
<td>29 090</td>
</tr>
<tr>
<td>2</td>
<td>PI8,</td>
<td>#3 Type 2</td>
<td>0.1168</td>
<td>76.13</td>
<td>116.9</td>
<td>29 460</td>
</tr>
<tr>
<td>2</td>
<td>PE9</td>
<td>#2 smooth</td>
<td>0.0475</td>
<td>46.67</td>
<td>59.1</td>
<td>29 500</td>
</tr>
</tbody>
</table>

### TABLE 2.03: Concrete Properties

<table>
<thead>
<tr>
<th>CASTING NUMBER</th>
<th>DATE OF CASTING</th>
<th>SLUMP (in.)</th>
<th>$f_{c'}$ (ksi)</th>
<th>$f_r$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>8-25-89</td>
<td>5.0</td>
<td>5.015</td>
<td>0.580</td>
</tr>
<tr>
<td>2nd</td>
<td>10-25-89</td>
<td>2.0</td>
<td>5.336</td>
<td>0.613</td>
</tr>
<tr>
<td>3rd</td>
<td>12-08-89</td>
<td>4.0</td>
<td>5.650</td>
<td>0.613</td>
</tr>
</tbody>
</table>

$f_{c'}$ = Concrete Compressive Strength

$f_r$ = Modulus of Rupture
### TABLE 3.01: Behavior of Interior Connection Specimens

<table>
<thead>
<tr>
<th>SPEC. No.</th>
<th>SHEAR CAPITAL SIZE (in.)</th>
<th>DRIFT ** (%)</th>
<th>PEAK LOAD (kips)</th>
<th>$K_p/K_i$ *** (%)</th>
<th>ROT. ** (rads)</th>
<th>FAILURE MODE</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI8</td>
<td>NONE</td>
<td>3.5</td>
<td>8.85</td>
<td>42</td>
<td>0.035</td>
<td>SHEAR</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-3.5</td>
<td>-8.75</td>
<td>42</td>
<td>0.028</td>
<td></td>
</tr>
<tr>
<td>SC6</td>
<td>24</td>
<td>5.0</td>
<td>18.4</td>
<td>32</td>
<td>0.022</td>
<td>SHEAR</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-4.0</td>
<td>-14.9</td>
<td>41</td>
<td>0.013</td>
<td></td>
</tr>
<tr>
<td>SC4</td>
<td>32</td>
<td>6.0</td>
<td>18.1</td>
<td>30</td>
<td>0.01</td>
<td>FLEXURE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-5.0</td>
<td>-17.3</td>
<td>38</td>
<td>0.01</td>
<td></td>
</tr>
<tr>
<td>SC6</td>
<td>40</td>
<td>6.0</td>
<td>20.8</td>
<td>36</td>
<td>0.014</td>
<td>FLEXURE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-7.0</td>
<td>-20.4</td>
<td>30</td>
<td>0.018</td>
<td></td>
</tr>
</tbody>
</table>

### TABLE 3.02: Behavior of Edge Connection Specimens

<table>
<thead>
<tr>
<th>SPEC. No.</th>
<th>SHEAR CAPITAL SIZE (in.)</th>
<th>DRIFT ** (%)</th>
<th>PEAK LOAD (kips)</th>
<th>$K_p/K_i$ *** (%)</th>
<th>ROT. ** (rads)</th>
<th>FAILURE MODE</th>
</tr>
</thead>
<tbody>
<tr>
<td>PE9</td>
<td>NONE</td>
<td>3.5</td>
<td>3.69</td>
<td>27</td>
<td>0.025</td>
<td>TORSION &amp; FLEXURE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-3.0</td>
<td>-5.60</td>
<td>24</td>
<td>0.025</td>
<td></td>
</tr>
<tr>
<td>SC5</td>
<td>24</td>
<td>6.0</td>
<td>5.82</td>
<td>18</td>
<td>0.012</td>
<td>TORSION &amp; FLEXURE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-5.0</td>
<td>-10.99</td>
<td>22</td>
<td>0.022</td>
<td></td>
</tr>
<tr>
<td>SC3</td>
<td>32</td>
<td>7.0</td>
<td>9.20</td>
<td>18</td>
<td>0.014</td>
<td>FLEXURE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-7.0</td>
<td>-10.50</td>
<td>18</td>
<td>0.020</td>
<td></td>
</tr>
<tr>
<td>SC1</td>
<td>40</td>
<td>6.5</td>
<td>7.54</td>
<td>25</td>
<td>0.020</td>
<td>TORSION &amp; FLEXURE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-5.0</td>
<td>-12.93</td>
<td>35</td>
<td>0.025</td>
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**TABLE 4.01: Measured Response of Interior Connections**

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>SHEAR CAPITAL SIZE (in.)</th>
<th>$f_c'$ (psi)</th>
<th>PEAK LOAD (kips)</th>
<th>PEAK DRIFT (%)</th>
<th>MOMENT $M_u$ (kip-in)</th>
<th>MOMENT $(M_u)_{ch}$ (kip-in)</th>
<th>MOMENT $(M_u)_{ef}$ (kip-in)</th>
<th>SHEAR $V_u$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P18</td>
<td>NONE</td>
<td>5700</td>
<td>8.85</td>
<td>3.5</td>
<td>549</td>
<td>N/A</td>
<td>N/A</td>
<td>14.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-8.75</td>
<td>-3.5</td>
<td>543</td>
<td>N/A</td>
<td>N/A</td>
<td>15.98</td>
</tr>
<tr>
<td>SC6</td>
<td>24</td>
<td>5650</td>
<td>18.4</td>
<td>5.0</td>
<td>1,143</td>
<td>376</td>
<td>531</td>
<td>11.53</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-14.9</td>
<td>4.0</td>
<td>924</td>
<td>312</td>
<td>423</td>
<td>15.98</td>
</tr>
<tr>
<td>SC4</td>
<td>32</td>
<td>5336</td>
<td>18.1</td>
<td>6.0</td>
<td>1,124</td>
<td>319</td>
<td>446</td>
<td>11.83</td>
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<tr>
<td></td>
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<td></td>
<td>-17.3</td>
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<td>1,069</td>
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<td>437</td>
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<td>5015</td>
<td>20.8</td>
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<td>1,289</td>
<td>370</td>
<td>470</td>
<td>12.19</td>
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<td></td>
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<td>-20.4</td>
<td>7.0</td>
<td>1,264</td>
<td>369</td>
<td>458</td>
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</table>

* unbalanced moment at the center of connection
** moment at the face of shear capital
<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>$(M_n)_c$ (kip-in)</th>
<th>$(M_n)_sc$ (kip-in)</th>
<th>$v_{ab}/u_c$</th>
<th>$v_{cd}/u_c$</th>
<th>$\gamma_fM_u/(M_n)_c$</th>
<th>$v_{af}/u_c$</th>
<th>$v_{gh}/u_c$</th>
<th>$\gamma_fM_u/(M_n)_sc$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI8</td>
<td>399</td>
<td>N/A</td>
<td>1.01</td>
<td>0.53</td>
<td>0.83</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>SC6</td>
<td>1027</td>
<td>398</td>
<td>1.02</td>
<td>0.50</td>
<td>0.82</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>SC4</td>
<td>1163</td>
<td>397</td>
<td>0.60</td>
<td>0.44</td>
<td>0.67</td>
<td>0.59</td>
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<tr>
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<td>1163</td>
<td>397</td>
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<td>0.49</td>
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<td>1.39</td>
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<td>SC2</td>
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<td>395</td>
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<td>0.55</td>
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<td>0.18</td>
<td>1.62</td>
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</table>

TABLE 4.02: Analysis of Interior Connections Using the ACI Code
<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>SHEAR CAPITAL SIZE (IN.)</th>
<th>$f_c'$ (psi)</th>
<th>PEAK LOAD (kips) *</th>
<th>PEAK DRIFT (%) *</th>
<th>MOMENT $(M_{ub})_c$ (kip-in) *</th>
<th>MOMENT $(M_{ub})_{sc}$ (kip-in) *</th>
<th>SHEAR Vu (kips) *</th>
</tr>
</thead>
<tbody>
<tr>
<td>PE9</td>
<td>NONE</td>
<td>5700</td>
<td>3.69</td>
<td>3.5</td>
<td>225</td>
<td>N/A</td>
<td>-0.63 ***</td>
</tr>
<tr>
<td>SC5</td>
<td>24</td>
<td>5650</td>
<td>5.82</td>
<td>6.0</td>
<td>346</td>
<td>332</td>
<td>-2.52</td>
</tr>
<tr>
<td>SC3</td>
<td>32</td>
<td>5336</td>
<td>-10.99</td>
<td>5.0</td>
<td>614</td>
<td>555</td>
<td>16.30</td>
</tr>
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<td>5015</td>
<td>9.20</td>
<td>7.0</td>
<td>540</td>
<td>489</td>
<td>-6.48</td>
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</tbody>
</table>

* upper number corresponds to positive bending
lower number corresponds to negative bending

** drift at peak load

*** corresponds to uplifting of the column
<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>((M_n)_c) (kip-in)</th>
<th>((M_n)_{sc}) (kip-in)</th>
<th>(u_{(ab)}/u_c) (13)</th>
<th>(u_{(ad)}/u_c) (14)</th>
<th>(u_{(ef)}/u_{sc}) (16)</th>
<th>(u_{(gh)}/u_{sc}) (17)</th>
<th>(\gamma_f M_u/(M_n)_{sc}) (18)</th>
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<td>PE9</td>
<td>166</td>
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<td>1.46</td>
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<tr>
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<td>615</td>
<td>229</td>
<td></td>
<td></td>
<td>0.57</td>
<td>0.42</td>
<td>1.28</td>
</tr>
<tr>
<td>SC1</td>
<td>545</td>
<td>167</td>
<td>0.41</td>
<td>0.78</td>
<td>0.49</td>
<td>0.19</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>612</td>
<td>228</td>
<td></td>
<td></td>
<td>0.71</td>
<td>0.40</td>
<td>1.47</td>
</tr>
</tbody>
</table>

* upper number corresponds to positive bending
  lower number corresponds to negative bending

** at negative bending side of slab

*** at positive bending side of slab

TABLE 5.02: Analysis of Edge Connections Using the ACI Code
**Table 5.03: Eccentric Shear Stress Model Using Torsional Capacity of Slab Edge**

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>AT (kip-in) (19)</th>
<th>COLUMN (kip-in) (20)</th>
<th>FACE</th>
<th>AT (kip-in) (24)</th>
<th>SHEAR (kip-in) (25)</th>
<th>CAPITAL (kip-in) (26)</th>
<th>FACE (kip-in) (27)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( T_n ) (_c)</td>
<td>( M_f ) (_c) *</td>
<td>( (M_{ub})_c ) (_c) *</td>
<td>( (v_u)_c ) (_c)</td>
<td>( (T_n)_{sc} ) (_c) *</td>
<td>( (M_f)_{sc} ) (_c) *</td>
<td>( (M_{ub})_{sc} ) (_c) *</td>
</tr>
<tr>
<td>PE9</td>
<td>33</td>
<td>168</td>
<td>0.96</td>
<td>0.38</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>231</td>
<td>0.60</td>
<td>1.07</td>
<td>0.60</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SC5</td>
<td>51</td>
<td>422</td>
<td>0.66</td>
<td>0.22</td>
<td>29</td>
<td>307</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>260</td>
<td>0.36</td>
<td>0.87</td>
<td>0.36</td>
<td>372</td>
<td>1.29</td>
<td>0.33</td>
</tr>
<tr>
<td>SC3</td>
<td>51</td>
<td>543</td>
<td>0.84</td>
<td>0.27</td>
<td>29</td>
<td>311</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>260</td>
<td>0.36</td>
<td>0.82</td>
<td>0.36</td>
<td>375</td>
<td>1.12</td>
<td>0.24</td>
</tr>
<tr>
<td>SC1</td>
<td>47</td>
<td>541</td>
<td>0.70</td>
<td>0.26</td>
<td>28</td>
<td>375</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td>260</td>
<td>0.40</td>
<td>1.04</td>
<td>0.40</td>
<td>440</td>
<td>1.15</td>
<td>0.22</td>
</tr>
</tbody>
</table>

\( T_n \) = torsional capacity of the slab edge  
\( M_f \) = capacity of slab to transfer unbalanced moment by flexure  
\( M_n = 2T_n + M_f \) = unbalanced moment capacity of connection  
\( M_{ub} \) = peak unbalanced moment resisted by the test specimen  
\( v_n \) = shear stress capacity of connection (ksi)  
\( v_u \) = peak shear stress resisted by the test specimen (ksi)  
* upper number corresponds to positive bending  
  lower number corresponds to negative bending
Fig. 1.01: Various Flat-Slab Configurations
SLAB TOP REINFORCEMENT

SLAB BOTTOM REINFORCEMENT

x = strain gage location

Fig. 2.04: Interior Connection Slab Reinforcement Layout
Fig. 2.05: Edge connection slab reinforcement layout.
Fig. 2.06: Shear Capital Reinforcement Layout
Fig. 2.07: Shear Capital Reinforcement Detail
Fig. 2.09: Lateral Displacement History
Fig. 3.01: Crack Pattern - Specimen PI8 [no shear capital]
Fig. 3.02: Crack Pattern - Specimen SC6 [24 in. shear capital]
Fig. 3.03: Crack Pattern - Specimen SC4 [32 in. shear capital]
Fig. 3.04: Crack Pattern - Specimen SC2 [40 in. shear capital]
(a) 1.5 percent drift
(b) 3.5 percent drift

Fig. 3.05: Crack Pattern - Specimen PE9 [no shear capital]
Fig. 3.06: Crack Pattern - Specimen SC5 [24 in. shear capital]
Fig. 3.07: Crack Pattern - Specimen SC3 [32 in. shear capital]
Fig. 3.08: Crack Pattern - Specimen SC1 [40 in. shear capital]
Fig. 3.09: Load vs. Drift - Specimen PI8 [no shear capital]
Fig. 3.10: Load vs. Drift - Specimen SC6 [24 in. shear capital]
Fig. 3.11: Load vs. Drift - Specimen SC4 [32 in. shear capital]
Fig. 3.12: Load vs. Drift - Specimen SC2 [40 in. shear capital]
Fig. 3.13: Load vs. Drift - Specimen PB9 [no shear capital]
Fig. 3.14: Load vs. Drift - Specimen SC5 [24 in. shear capital]
Fig. 3.15: Load vs. Drift - Specimen SC3 [32 in. shear capital]
Fig. 3.16: Load vs. Drift - Specimen SCI [40 in. shear capital]
Fig. 3.18: Strength Envelopes - Edge Connections
peak to peak stiffness = \frac{|P1| + |P2|}{|d1| + |d2|}

P = peak load
d = lateral displacement at peak load

Fig. 3.19: Peak to Peak Stiffness
Fig. 3.20: Peak to Peak Stiffness - Interior Connections
Fig. 3.21: Peak to Peak Stiffness - Edge Connections
FIG. 3.22: Stiffness Degradation - Interior connections
Fig. 3.23: Stiffness Degradation - Edge Connections
Fig. 3.25: Moment vs. Rotation – Specimen SC6 [24 in. shear capital]
Fig. 3.26: Moment vs. Rotation - Specimen SC4 [32 in. shear capital]
Fig. 3.27: Moment vs. Rotation - Specimen SC2 [40 in. shear capital]
Fig. 3.28: Moment vs. Rotation - Specimen PE9 [no shear capital]
Fig. 3.29: Moment vs. Rotation - Specimen SC5 [24 in. shear capital]
Fig. 3.30: Moment vs. Rotation - Specimen SC3 [32 in. shear capital]
Fig. 3.31: Flexural Hinging Mechanism of Interior
Connections with Shear Capitals
Fig. 3.33: Punching Shear Failure Surface

Specimen PI8  [no shear capital]
Fig. 3.34: Punching Shear Failure Surface

Specimen SC6  [24 in. shear capital]
Fig. 3.35: Energy Dissipation – Interior Connections
Fig. 3.36: Energy Dissipation - Edge Connections
FIG. 3.37: Reinforcement Strain vs. Drift - SC6
FIG. 3.38: Reinforcement Strain vs. Drift - SC6
FIG. 3.39: Reinforcement Strain vs. Drift - SC4
FIG. 3.40: Reinforcement Strain vs. Drift - SC4
FIG. 3.41: Reinforcement Strain vs. Drift - SC2
FIG. 3.42: Reinforcement Strain vs. Drift - SC2
EDGE CONNECTION strain gage 37
24 INCH SHEAR CAPITAL

FIG. 3.43: Reinforcement Strain vs. Drift - SC5
FIG. 3.44: Reinforcement Strain vs. Drift - SC5
FIG. 3.45: Reinforcement Strain vs. Drift - SC3
FIG. 3.46: Reinforcement Strain vs. Drift - SC3
FIG. 3.47: Reinforcement Strain vs. Drift - SC1
FIG. 3.48: Reinforcement Strain vs. Drift - SC1
Fig. 4.01: Moment Diagram for Interior Connections Subjected to Combined Gravity and Lateral Loads

(a) dominant gravity load

(b) dominant lateral load
Concrete Area of Critical Section:

\[ A_c = 2(a + b)d \]

Modulus of Critical Section:

\[ \frac{J_c}{c} = \frac{(ad(a + 3b) + a^3)}{3} \]

where \( c = a/2 \)

Assumed Distribution of Shear Stress:

\[ v_{at} = \frac{V}{A_t} + \frac{\gamma_s Mc}{J_t} \]

\[ v_{cd} = \frac{V}{A_c} - \frac{\gamma_s Mc}{J_c} \]

FIG. 4.02: Critical Section at Column for Punching Failure in an Interior Column Connection
Fig 4.03: Possible Critical Sections for an Interior Connection

with Changes in Slab Thickness According to ACI 318-89
Fig. 4.04: Possible Locations for the Initiation of Punching Shear

Failure in an Interior Column Connection with Shear Capital
Fig 4.05: Possible Locations for the Initiation of Punching Shear Failure in Specimen SC6
Punching Shear Failure Occurs When

\[ \theta = 25^\circ \rightarrow 30^\circ \]

\[ \tan \theta = \frac{h}{l_{sc}} \]

Let \( \tan \theta = 0.5 \)

\[ l_{sc} = 2h \]

\[ L_{sc} = 4h + c \]

FIG. 4.06: Optimum Shear Capital Size
(bending perpendicular to edge)

Concrete Area of Critical Section:

\[ A_e = 2(a+b)d \]

Modulus of Critical Section:

\[ \frac{J_c}{c} = \frac{(2ad(a+2b)+a^3(2a+b)/a)/6}{c} \]

where \( c = a^2/(2a+b) \)

Assumed Distribution of Shear Stress:

\[ V_{sb} = \frac{V}{A_e} + \frac{\gamma_u Mc}{J_c} \]

FIG. 5.01: Critical Section at Column for Punching Failure

in an Edge Column Connection
Fig. 5.02: Yeild Lines in Edge Connection
PHOTO 2.01: Interior Connection Slab Reinforcement

PHOTO 2.02: Edge Connection Slab Reinforcement
PHOTO 2.03: Edge Connection Shear Capital Reinforcement
PHOTO 2.04: Interior Connection Shear Capital Reinforcement

PHOTO 2.05: Test Frame with Interior Connection
PHOTO 3.01: Punching Shear Failure - SC6 - Top View

PHOTO 3.02: Punching Shear Failure - SC6 - Bot. View
PHOTO 3.03: Specimen SC2 - 7 percent drift

PHOTO 3.04: Torsional Damage - SC1 - 40 in. Shear Capital
PHOTO 3.05: Torsional Damage - SC3 - 32 in. Shear Capital

PHOTO 3.06: Torsional Damage - SC5 - 24 in. Shear Capital