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Seismic resistance of slab-column connections in non-ductile flat-plate buildings

Du, Yong, Ph.D.
Rice University, 1993
RICE UNIVERSITY

SEISMIC RESISTANCE OF SLAB-COLUMN CONNECTIONS IN NON-DUCTILE FLAT-PLATE BUILDINGS

by

YONG DU

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE DOCTOR OF PHILOSOPHY

APPROVED, THESIS COMMITTEE

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May, 1993
ABSTRACT

SEISMIC RESISTANCE OF SLAB-COLUMN CONNECTIONS
IN NON-DUCTILE FLAT-PLATE BUILDINGS

BY

YONG DU

In order to determine the seismic response of existing reinforced concrete non-ductile flat-plate buildings, four reinforced concrete slab-column connection subassemblies designed for gravity load only were built at half-scale of prototype structure, and tested at Ryon laboratory of the Rice University. Each specimen had two exterior and one interior connections. The variables included the presence of spandrel beam, magnitude of gravity load, and reinforcing arrangement.

The behavior of entire subassemblies and the individual connections under the cyclic loading was studied in terms of failure mode, stiffness, ductility, shear capacity, and moment transfer capacity in the two loading directions. Based on the test results, a procedure to predict the flexural and shear strengths of the interior and exterior connections is recommended.

A hysteretic model for non-ductile slab-column connections is proposed. This model reflects the unsymmetrical moment transfer behavior of non-ductile connections. Furthermore,
the model is incorporated in a non-linear dynamic analysis program and a typical flat-plate building is analyzed to demonstrate the application of the analytical model.
ACKNOWLEDGEMENT

In writing this dissertation, it is pleasure to express my deep appreciation to my advisor, Professor Ahmad Jan Durrani for his guidance and personal encouragements.

Thanks are extended to Professors Y.C. Angel, P.C. Dakoulas for reading the dissertation and serving on the committee.

This study was sponsored by the National Center for Earthquake Engineering Research at SUNY Buffalo under grant NCEER 89-1001D and 90-1001D, which is gratefully acknowledged.

Thanks are also due to all my friends for their support. Finally, I express my deep appreciation to my parents and my wife for their understanding and unceasing support.
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CHAPTER 1

INTRODUCTION

1.1 Introduction

Eastern part of the United States has a large inventory of flat-slab buildings which were constructed forty to fifty years ago and thus have design and reinforcing detail for resisting gravity loads only. Recent concerns over the possibility of a moderate size earthquake in the eastern United States necessitate the evaluation of damage potential of such buildings. Buildings of this type are typically five to fifteen stories high and are used for office and apartment type dwellings. Experience from recent earthquakes (1,2) have demonstrated the vulnerability of such buildings to strong ground motion. The survival of such buildings in the event of an earthquake depends mostly on the performance of slab-column connections which are typically the most vulnerable link in the resistance of flat-plate buildings to lateral loading. Because of the inadequate reinforcing detail the unbalanced moment transfer capacity and shear resistance of such connections under reversed cyclic loading is relatively limited. Furthermore, the progressive collapse precipitated by punching failure of slabs is of serious concern and was the cause of extensive life and property damage during the Mexico City earthquake. The modern seismic codes require continuity of slab bottom reinforcement through the column to prevent such collapse. The old buildings lack such detail
and thus require a realistic evaluation of the possibility of such a failure.

This investigation was carried out to assess the inherent resistance of such buildings to earthquake loading, and to develop analytical tools for estimating their lateral drift response during strong ground motion. Knowing the capacity and response mechanisms, it should be possible to develop effective retrofitting schemes to improve the survivability of these buildings.

1.2 Background

The study focused on flat-plate buildings constructed during forties and fifties (3,4,5,6). The design of flat-plate structures during that time was mostly empirical. Typically, the slab is divided each way into column strips, which served the purpose of beams between the columns; and middle strip, which can be regarded as suspended spans that were carried by column strips. For the simple regular flat-plates, the direct design method is used to calculate moments at critical sections, the total moment is then distributed between column strips and middle strips for the design of slab.

The shear stress is calculated at a section equal to slab thickness minus 1.5 inches away from the outside edge of column capital parallel to or concentric with the column when no drop panel are used. Shear strength of the critical section is then determined based on the amount of rebars passing over the column capital. If 50% or more of slab negative
reinforcement passed over the column capital, the allowable shear stress is limited to 0.03 $f_c'$, and if 25% or less of slab reinforcement passed directly over the column capital the allowable shear stress is reduced to 0.025 $f_c'$.

For a concrete compression strength of 3000 psi, the shear strength factors of 0.03$f_c'$ and 0.025$f_c'$ are $1.64\sqrt{f_c'}$ and $1.37\sqrt{f_c'}$, respectively. These values are rather conservative compared with the ACI 318-89 building code (12) allowable shear stress of $4\sqrt{f_c'}$ for a square column. The effect of moment transfer on connection shear strength is not accounted for directly. The interaction between moment and shear force appears to be reflected in the reinforcing detail, the allowable shear stress is reduced if smaller percentage of negative moment reinforcement is contained in the column capital. Some moment-transfer capacity is built in due to lower allowable shear stress. The design of slab flexural reinforcement was done using working stress method considering gravity load only.

Furthermore, the slab reinforcing detail was such that, (a) rebars were evenly spaced across column and middle strips, (b) spacing of rebars was limited to three times the slab thickness, and (c) the slab positive reinforcement, perpendicular to the slab edge at exterior supports, extended at least six inches into support/column and the slab negative reinforcement was anchored into support with a hook.

The basic procedure for design of flat-slab structures did
not change much between 1941 and 1956. However, in 1956 ACI Code (6), the two allowable limits of shear stress were further limited to 100 psi and 85 psi depending upon the amount of slab negative reinforcement passing over the column capital as discussed earlier. Since it was about that time concrete over 4000 psi strength began to be commonly used but not much research data was yet available to consider the existing shear stress limits valid for high concrete strength. In addition, the minimum length of slab reinforcement for straight and bent-up reinforcing detail were also clarified.

The allowable shear stress was changed in the 1963 ACI Code (7) to the $2\sqrt{f_c'}$ to reflect consensus that shear strength was proportional to square root of the concrete compressive strength instead of a direct relationship as described in the previous codes. Furthermore, the shear strength of the connections was to be checked for the most severe of the two conditions: (a) the slab acting as a wide beam unit, a potential diagonal crack extending in a plane across the entire width, and (b) two way action for the slab with potential diagonal cracking along the surface of an inverted truncated cone or pyramid around the column. These processes formed the basis of the shear design contained in the present code. Along with the recognition of the strength design method, the nominal shear stress for two-way action was calculated by

$$v_u = \frac{V_u}{\phi b_o d}$$  \hspace{1cm} (1.1)
With critical section located at a distance d/2 out from the periphery of column. The reinforcing detail and the overall design procedure, however, remained unchanged.

In the 1971 ACI Code (8), the maximum allowable shear stress was changed to $4\sqrt{f_c}$ on the critical section and the concept of moment-transfer through eccentricity of the shear about the centroid was recognized. Research in sixties at PCA laboratories demonstrated that the allowable shear stress factor of $4\sqrt{f_c}$ was unconservative for rectangular columns and also that 40% of the unbalanced moment at the connection was transferred by eccentricity of the shear varying linearly about the centroid of the critical section. These conclusions were, however, mostly based on tests of square columns. The proportion of moment transferred through shear was calculated by:

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3\sqrt{c_1/d}} \frac{c_2/d}{c_1+d}} \quad (1.2)$$

ASCE-ACI Task Committee 426 studied the research data and in 1974 reported the shear strength of reinforced concrete member and slabs (9). In this report the committee recognized the moment-transfer mechanism, the effect of the column aspect ratio on shear strength and the column side dimension to slab depth were realized important variables and were indirectly taken into account by assuming a pseudo critical section located at a distance d/2 from the column face. Based on this
report, the shear strength calculations were further refined in 1977 ACI Code (10) as

\[ \phi \left( 2 + \frac{4}{\beta_c} \right) \sqrt{f_{c'}} \leq \phi 4 \sqrt{f_c}, \] (1.3)

The shear design procedure described in the 1977 ACI Building Code remained unchanged in the 1983 (11) ACI Code. For flat-slab buildings in seismic zones, the reinforcing detail in the connection region was made more specific to maintain the integrity of the connection under load reversals. It was specified that (a) at least one of the column strip reinforcement bars be placed within a narrow effective width, (b) not less than one-quarter of top reinforcement at the support in the column strip should be continuous through the span, (c) continuous bottom reinforcement in the column strip should not be less than one-third of the top reinforcement at the support in the column strip. (d) not less than one-half of all bottom reinforcement at midspan should be continuous and should develop its yield strength at face of support, and (e) at discontinuous edges of the slab all top and bottom reinforcement at support should be developed at the face of support.

The shear design method in the current ACI-89 Code (12) is basically the same as the ACI 381-83, however, the reinforcing detail for slab-column connections is changed. At least two continuous bottom bars are anchored into connections and, for slabs in frames not braced against
sideways, the rebar embedded length is determined by analysis in order to resist the action of lateral load.

In a recent investigation of a collapsed warehouse due to high over loads, Vecchio and Collins (13) attributed high floor load capacities to membrane action in the slab. It was suggested that the compression of in plane membrane substantially increased both the shear and flexural capacities of the slab.

1.3 Objectives and Scope

The main objective of this study was to investigate the lateral load resistance of flat-plate connections designed and detailed to resist gravity load only and, on the basis of observed response, to develop analytical models suitable for predicting the response of flat-plate buildings under earthquake-type loading.

The experimental phase of the investigation consisted of testing connection reinforcing details, typical of gravity load design. This phase of the project was expected to yield data on the response of connections such as failure mechanism, lateral load resistance, stiffness, energy dissipation capacity, connection shear strength, and the ability to transfer moment between the slab and the column. The scope of the testing program was limited to the study of three variables which included the intensity of gravity loading, the configuration of slab reinforcement, and the presence of spandrel beams. Since continuity of the slab-column system
played an important role in the response of individual connections, the test specimens were configured as two-bay flat-plate subassemblies.
CHAPTER 2

EXPERIMENTAL PROGRAM

Most of the existing flat-plate buildings range in height from five to fifteen stories and may or may not have drop panels and spandrel beams. This investigation focused only on flat-plate construction without drop panels. Since spandrel beams are commonly present in flat-plate buildings, their effect on the response of connections was also studied in this investigation.

2.1 Prototype Building

The prototype structure for this investigation was a five story flat-plate building with three bays in the short direction and four bays in the long direction as shown in Appendix A Fig.A-1. The columns were typically 20 in. x 20 in. In cross section and were 20 ft. apart with each story as 10 ft. The compression strength of concrete was chosen as 3000 psi with reinforcement of Grade 40 steel. These properties can be considered typical of those used in early fifties.

The prototype building was assumed as an office building with a live load of 50 lb/ft². The total gravity load on floors, including the slab weight, was estimated as 195.5 lb/ft². The details of the design calculations, which are based on ACI-47 (4), are described in Appendix A. The slab thickness in this procedure is limited to ratio of L/36 or that obtained from shear considerations. The final thickness of 9 inches was chosen on the basis of shear strength. Moment
was determined using the direct design method and the slab reinforcement in the column and middle strips was chosen based on the working stress design procedure.

2.2 Specimen Design

The test specimens were half-scale slab-column connection subassemblies, each consisting of two exterior connections and one interior connection. The overall configuration of the test specimen and the individual connection details are shown in Figs. 2-1, 2-2 and 2-3, respectively. Since the study focused on the slab response in the connection region, the columns were designed to remain elastic with reinforcement consisting of 6 #7 Grade 60 bars. The gravity loading on the subassemblies was adjusted to result in the level of shear stress in the connection region as calculated for the prototype building. Connection reinforcing details as used during the construction of the specimens are shown in Figs. 2-4, 2-5 and 2-6. The top reinforcing ratio of column strip for both exterior and interior connections is 0.59%, for bent-up rebars the bottom reinforcing ratio of column strip for both exterior and interior connections is 0.22% and for straight the bottom reinforcing ratio is 0.37%. For bent-up rebars the bottom reinforcement is only 37.6% of the top reinforcement in column strip in the connection region, for straight rebars the bottom reinforcement is 63% of the top reinforcement in column strip in the connection region.

For a real half-scale modeling of the chosen prototype, the
FIGURE 2-1 The Overall Specimen Configuration
FIGURE 2-2 Reinforcement Detail of Specimens
FIGURE 2-3 Reinforcement Detail of Specimens

FIGURE 2-4 Interior Connection Reinforcement in DNY_4
FIGURE 2-5 Bent-Up Rebars at Exterior Connection in DNY_4

FIGURE 2-6 Reinforcement Detail of Spandrel Beam in DNY_4
specimens would have a span of 10 in., and slab width of in.
Due to constrains of testing frame, these dimensions were
reduced to 9.5 ft. and 6.5 ft., respectively. The columns in
the test subassemblies were terminated at the inflection
points which were assumed stationary at middle height of the
story above and below the slab. Keeping in view the scope and
objectives of this study, and the experience from previous
investigations of slab-column connections, three variables
were selected. These included the configuration of the slab
reinforcement in the connection region, presence of spandrel
beams and the intensity of gravity loading on the slab at the
time of lateral cyclic loading. These variables were determined
on the basis of detailing practised at the time of construction
of these buildings and on the basis of adverse loading condition
that these buildings may experience. Further details of the
variables and material properties are given in Table I.
2.3 Test Set-Up
The slab-column connection subassemblies were tested in a
steel reaction frame as shown in Figs. 2-7 and 2-8. The top
of columns were all connected to a rigid beam through load
cells, with the bottom of each column attached to the reaction
frame through a hinge. The lateral load was applied to the
rigid beam by a servo controlled actuator. The shear in each
column was independently measured along with the vertical
reaction at the center column which rendered the subassembly
statically determinate.
<table>
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<th>REBAR DETAIL</th>
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<th>$f_y$ (ksi)</th>
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FIGURE 2-7 Specimen DNY_2 in Testing Frame

FIGURE 2-8 Gravity Load Application to SLab in DNY_2
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FIGURE 2-9  Lateral Displacement Routine
adjacent to columns to measure rotation of the slab relative to the column along the loading direction. Yielding of slab reinforcement and loss of anchorage of reinforcement was determined from strain gages attached to slab bars at suitable locations. Targets were attached to the slab surface on a grid pattern which could be read using a level for determining deformed shape of the slab at various stages of the test. All of these measurement were recorded continually using an automated data acquisition and control system except for the recording of the crack development and deflection measurements which were taken while the test was briefly paused at regular intervals for visual observations.
CHAPTER 3

OVERALL SUBASSEMBLY RESPONSE

3.1 General Response

This section describes the overall response of the slab-column connection subassemblies. The behavior of subassemblies is presented in terms of lateral load carrying capacity, stiffness, energy dissipation, cracking pattern, and the observed mode of failure. The general behavior of the individual specimen is described first, followed by a specific discussion of response of interior and exterior connections. The overall responses of specimens is given in Table II.

SPECIMEN DNY_1: This specimen simulated the bent-up reinforcing detail in the slab and carried 30% of the uniformly distributed live load in addition to the design dead load. The yielding of slab top reinforcement first occurred near the face of the exterior column at a 0.75% drift. At 1% drift, the slab top reinforcement at interior column also yielded. The flexural cracks at the bottom of the slab, adjacent to exterior joints, grew to full slab width by 1.5% drift. Since only a small fraction of the slab bottom reinforcement extended into columns, the yielding of slab bottom reinforcement occurred as soon as the slab cracked in positive bending moment. Despite the limited anchorage length, the slab bottom reinforcement at the exterior column did not show any significant loss of bond or anchorage upto 2% drift. At 2.25% drift, however, the slab bottom reinforcement began
<table>
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<tr>
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<th>PEAK LOAD (kips)</th>
<th>PEAK DRIFT (%)</th>
<th>ULT. LOAD (kips)</th>
<th>ULT. DRIFT (%)</th>
<th>FAILURE MODE</th>
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<td>2.6</td>
<td>8.72</td>
<td>4.7</td>
<td>Punching</td>
</tr>
</tbody>
</table>
to pull out of the exterior connections resulting in a substantial increase in width of the slab bottom flexural crack. The specimen reached its peak lateral load at 3% drift. At this stage, the flexural cracks at top of the slab opened widely and concrete along the flexural crack at the bottom surface of the slab began to crush. The slab edge developed torsional cracks adjacent to the exterior joints and the anchorage of slab reinforcement in the exterior columns began to deteriorate rapidly. Because of the fully developed yield lines across the entire slab width, flexural hinges at the face of columns dominated the lateral response initially. Torsional and shear cracks in the slab adjacent to the exterior connections affected the response later in the test. At 4.5% drift, the exterior connections had suffered a combination of flexural and torsional cracks, while the interior connection experienced flexural yield lines adjacent to the column. The lateral load resistance of the specimen dropped gradually with each additional cycle.

**SPECIMEN DNY_2:** This specimen had reinforcing detail similar to that used in specimen DNY_1 except that full dead and live load was applied to the slab during the test. The increase in the gravity load had a very significant effect on the mode of failure. Unlike specimen DNY_1 which predominantly experienced a flexural mode of failure, the specimen DNY_2 failed due to punching shear at interior connection at 2% drift. The specimen sustained the first cycle at this drift
level but failed in punching during the repeat cycle at the same drift value. In anticipation of this type of failure, special arrangement was made to contain the total collapse of the slab by placing supports a few inches clear of the slab near the interior column. Yielding of the slab reinforcement at interior and exterior connections occurred at 0.75% and 0.90% drifts and at loads 67% and 80% of the peak loads, respectively. Because of the heavy gravity load the flexural cracks in the connection region were visible only on the top surface of the slab adjacent to each column.

**SPECIMEN DNY_3:** The reinforcement in the slab of this specimen consisted of straight bars, instead of the bent-up bars used in specimens DNY_1 and DNY_2. This type of reinforcing detail has also been commonly used in flat-plate buildings of vintage under consideration in this study. The bottom bars in the slab extended into the column and the slab, therefore had more resistance to positive bending adjacent to the column. Since the amount of top reinforcement in the slab was the same as in other specimens, the load at which the slab reinforcement yielded was not significantly different from the yield load of DNY_1 and DNY_2. The yielding of slab reinforcement initiated at 1% drift and the lateral load resistance peaked at about 2% drift. At this drift level the exterior connection region had suffered significant cracking due to shear, bending and torsion. The behavior of interior connection was dominated by flexural yielding of the slab.
At 5% drift, both exterior connections had lost the ability to transfer moment between the column and the slab.

**SPECIMEN DNY_4:** This specimen had the same configuration and detail as the specimen DNY_1 except that an 8in. x 12in. spandrel beam was added to the slab at the exterior connections. Such spandrel beams have been commonly used in flat-plate buildings to support exterior walls. The presence of spandrel beam increased the torsional stiffness of the slab and thus increased the moment transfer capacity of the connection significantly and, also changed the mode of failure. Unlike other specimens in which torsional cracks developed in the slab edge at the exterior connections, a major flexural crack developed at the face of the edge beam which extended the full width of the slab. The yielding of the slab reinforcement began at 0.5% drift and the specimen reached its peak load at 2% drift. At 2.25% drift, the slab bottom bars extending into the edge beam lost their anchorage. The failure of the specimen occurred at 5% drift when the interior connection failed in punching shear.

### 3.2 Cracking Pattern

All specimens, except DNY_2 which was subjected to full dead load plus live load, were uncracked under the service gravity loads (DL+0.3 LL) before the start of the test. On increasing the slab load to full dead and live load, minor cracks developed in specimen DNY_2 at the face of the interior connection. Under lateral loading, flexural and torsional
cracks formed in the slab exterior connections. These cracks extended and increased in width while new cracks developed at higher drift levels. By 2.5% drift the crack pattern stabilized. Further increase in drift only widened the existing cracks with very few new cracks.

3.3 Load-Deformation

The observed load-deformation response of the four specimens is shown in Figs. 3-1 through 3-4. Except for the specimen DNY_2, which was subjected to higher gravity load and thus failed in punching shear, the load-deformation response generally appears to be very similar. The envelope of the hysteresis loops in each loading direction is approximately bilinear with some tendency of gradual strength reduction. The loops are severely pinched, indicating a rapid loss of stiffness and a low energy dissipation capacity. In specimens with dominant flexural yield lines, energy was dissipated mostly by yielding of the slab reinforcement and opening of the flexural cracks. Slippage of reinforcing bars, local crushing of concrete, and friction along the major cracks also contributed to some dissipation of energy. The load-deformation response generally resembled that of an underreinforced flexural member but without a well-defined yield point. The specimen DNY_3 experienced an unexpected initial large displacement of 1.5% drift due to a malfunction in the servo-control system which caused lower stiffness of the specimen during the regular test cycles. The specimen
FIGURE 3-1  Hysteresis Loops of DNY_1

FIGURE 3-2  Hysteresis Loops of DNY_2
FIGURE 3-3  Hysteresis Loops of DNY_3

FIGURE 3-4  Hysteresis Loops of DNY_4
DNY_2 because of the higher gravity load, sustained only a limited number of cycles, due to reduction in deformation capacity under increased gravity shear (14).

3.4 Strength

The peak load and corresponding drift of specimens are given in Table II and the envelope of load-drift loops is shown in Fig. 3-5. The slab top reinforcement in DNY_2 yielded first at interior connection. The yielding of slab in all the other specimens occurred first at exterior connections. Because of the presence of the transverse beam, yielding of slab reinforcement at exterior connection of specimen DNY_4 occurred only at 0.5% drift. The peak load in positive direction were within 10% of the average peak load of 10.4 kips. The specimen DNY_2 and DNY_4 lost their strength when interior connections failed in punching. The ascending part of the loading curve in the negative direction for all specimens was very similar due to the loss of stiffness in the previous half cycle. The average peak load value in the negative direction was approximately 5% smaller than the average peak load reached in the positive direction.

The overall lateral load resistance of the specimens depended very much on the performance of slab-column connections (15,16). When the connection failed prematurely in punching shear, as in the case of specimen DNY_2, the slab could not develop its full flexural capacity and the observed strength of specimen was lower. The specimens DNY_1, DNY_3 which had
FIGURE 3-5 Load-Deformation Envelope of Specimens
flexural yielding of slab as the primary mode of lateral load resistance and DNY_4 which suffered the punching failure at 4.7% drift level were able to develop their full strength and sustained that load for at least upto 3% drift despite the loss of anchorage of the slab bottom bars at exterior connections.

The loss of strength under repeated cycles was evaluated by repeating load cycles at 1.5%, 2.0% and 4.5% drifts. Prior to reaching the peak load, the reduction in strength during the first repeated cycle was 5% to 8% and during the third repeated cycle the drop in strength was 7% to 11%. After the peak load, the reduction in strength increased to 10% to 18%. Typical loops from repeated cycles are shown in Fig. 3-6. Small displacement cycles of 1% drift were repeated at predefined intervals to evaluate the low amplitude stiffness and energy dissipation degradation of specimens. A typical comparison between the two loops, one at initial 1% drift and the other at 1% drift following 2% drift, is shown in Fig. 3-7. Both the energy dissipation capacity and the lateral load resistance during the second small drift cycle are significantly smaller.

3.5 Stiffness

Flat-plate connections are inherently flexible, and experience a rapid degradation of stiffness under cyclic loading (17). The observed degradation of stiffness during the positive and negative loading directions of the four test
FIGURE 3-6  Load vs. Drift Response Under Repeated Cycles

FIGURE 3-7  Comparison of Displacement Cycles
specimens is shown in Fig. 3-8. The stiffness of a specimen here is defined as the secant stiffness representing slope of the line connecting origin to the peak-displacement point during a given half cycle. The initial stiffness and the change in stiffness varied among the specimens up to 2% drift. Beyond this drift level, however, the stiffness of all specimens became very similar and it also degraded in an identical manner. The specimen DNY_2, which was subjected to a higher gravity load, had the least initial stiffness compared to other specimens. This is attributed to initial cracking of the slab under increased gravity load. The specimen with the edge beam had the highest initial stiffness in both positive and negative loading directions due to the restraining effect of the slab edge beam. Differences in reinforcement detail between the specimens DNY_1 and DNY_3 did not affect the stiffness. Cracking of the slab from loading in one direction, however, resulted in lower stiffness during loading in the opposite direction, which was always slightly lower. Approximately 50% of the initial stiffness was lost during the first 1% of drift. Additional 20% was lost when the specimens reached two percent drift. Only 10% of the original stiffness remained by the time the specimens reached 5% drift.

3.6 Ductility

The deformation capacity of connections has been traditionally expressed in the form of a ductility factor which is defined as a ratio of the ultimate displacement to
FIGURE 3-8  Stiffness Degradation of Specimens
the displacement at first yield of reinforcement. The slab-column connections, however, there is no well defined yield point. A common definition of an assumed yield point is illustrated in Fig. 3-9(a). In this definition, the intersection of a secant line connecting the origin with 3/4 peak-load point on the primary curve and the horizontal line through the peak-load is assumed as the yield point. The ultimate displacement is defined as a point on the strength envelope corresponding to strength loss of not more than 20% of the peak load (18,19,20). Table III shows the ductilities of the subassemblies based on this definition. The specimen DNY_4 with the edge beam had the largest ductility of 5.88, and the specimen DNY_2 with highest gravity load had the least ductility of 2.0. The reinforcing detail in the slab appear to have affected the deformation capacity. The specimen with bent-up bars had a ductility of 4.28 which is greater than the ductility of 3.75 for DNY_3.

3.7 Anchorage Failure

At interior connections, the slab bottom reinforcement either extended a limited distance into the interior column or stopped short of the column depending upon whether straight or bent-up reinforcing detail is used. At the exterior connections, the slab bottom reinforcement continued into the column a distance of 6". Furthermore, the amount of slab bottom reinforcement continuing into column is so small that under reversal of loading, the reinforcement would yield as
FIGURE 3-9  Ductility of Specimens
<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>P_{iy} (kips)</th>
<th>D_{iy} (%)</th>
<th>P_{ey} (kips)</th>
<th>D_{ey} (%)</th>
<th>P_{y} (kips)</th>
<th>D_{y} (%)</th>
<th>D_{u} (%)</th>
<th>D_{u}/D_{y}</th>
</tr>
</thead>
<tbody>
<tr>
<td>DNY_1</td>
<td>8.7</td>
<td>1.0</td>
<td>8.35</td>
<td>0.75</td>
<td>8.2</td>
<td>1.05</td>
<td>4.50</td>
<td>4.28</td>
</tr>
<tr>
<td>DNY_2</td>
<td>6.46</td>
<td>0.75</td>
<td>7.69</td>
<td>0.88</td>
<td>7.2</td>
<td>1.00</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>DNY_3</td>
<td>6.52</td>
<td>1.0</td>
<td>7.90</td>
<td>1.00</td>
<td>7.5</td>
<td>1.20</td>
<td>4.50</td>
<td>3.75</td>
</tr>
<tr>
<td>DNY_4</td>
<td>9.10</td>
<td>0.75</td>
<td>6.20</td>
<td>0.50</td>
<td>8.2</td>
<td>0.80</td>
<td>4.70</td>
<td>5.88</td>
</tr>
</tbody>
</table>

P_{iy}: load at yielding of interior connection
D_{iy}: drift at yielding of interior connection
P_{ey}: load at yielding of exterior connection
D_{ey}: drift at yielding of exterior connection
P_{y}: assumed yield load
D_{y}: assumed yield drift
D_{u}: ultimate drift
soon the cracking moment is reached. The ability to transfer moment between the slab and the column under positive bending, depended a great deal on the anchorage performance. At exterior connections, the reversal of loading resulted in the loss of anchorage of slab bottom reinforcement after yielding of the bars. Until that point, the slab bottom reinforcement assisted in limiting the crack width at slab bottom in the connection region. However, once the anchorage failed, the moment transfer capacity under positive bending became negligible. When this occurred, the lateral load capacity dropped and the load redistributed to other columns. The effect of local anchorage failure at a connection did not have an obvious effect on the overall response of the subassemblies. After the loss of anchorage, the lateral load resistance was provided by the connections with slab under negative bending.

3.8 Moment Redistribution

The moment distribution in the slab along the column line explains both the cracking pattern and the interaction among the different connections. The slab reinforcing detail, presence of the spandrel beam, and the intensity of the gravity load all affected the distribution of moment among the connections. The ultimate shape of the moment diagram was influenced by the change in stiffness at individual connections and their ultimate modes. The discussion below will focus on several aspects of moment redistribution among connections.

As noted earlier, two gravity load intensities were used
to simulate dominant lateral load and dominant gravity load conditions. Fig. 3-10 shows the moment diagrams for gravity load alone and gravity load combined with lateral load. As shown in Fig. 3-10(a) and 3-10(b) the moment diagrams resulting from gravity load alone, before the application of lateral load were symmetrical and the specimens remained mostly in elastic state (21,22). Under this loading, the negative moment at interior connections was three times larger than the negative moment at exterior connections. The 50% of gravity load was distributed to interior column. The combined gravity load and lateral load moment diagram for specimen DNY_1 at 2% drift is shown in Fig. 3-10(c). The negative moments in connections in Fig. 3-10(c) caused by gravity load became smaller in comparison with the moment in Fig. 3-10(a) due to the reduction in stiffness of connections. Under action of lateral load, the 65% of overall lateral load was distributed to interior column, the rest was taken by two exterior columns.

The distribution of moment at 0.5% and 2% drifts for specimen DNY_1 is shown in Figs. 3-11(a) and (b). At 0.5% drift the response was essentially elastic. Forty two percent of the lateral load was carried by the interior column and the rest resisted by the exterior columns. The maximum positive moment was near the center of both spans. As the drift level increased, the positive moments near the column exceeded the cracking strength of the slab. By 2% drift, the slab top reinforcement near the columns had yielded and the subassembly reached its
FIGURE 3-10  Observed Moment Distribution in DNY_1 and DNY_2
maximum lateral resistance. The moment diagram at this stage reflects the effect of cracking in the slab under reversed loading and of yielding of the slab top reinforcement. Because of the limited capacity of the slab in positive bending, the moment carried by the left exterior connection decreased considerably and it redistributed to the interior connection on the column side with slab top reinforcement in tension. Early loss of stiffness at the edge connections transferred a significant portion of the gravity and lateral load to the interior connection. At 2% drift, the interior connection carried 65% of the lateral load while the left exterior connection carried only 7.5% of the lateral load. The positive moment immediately to the interior column was always below the cracking moment. Hence the cracks in the bottom of slab in the right span occurred closer to the midspan where the moment exceeded the cracking strength.

The effect of higher gravity load on moment distribution at 2% drift is illustrated in Fig. 3-11(c). Since the specimen DNY_2 was subjected to full dead and live load, the interior connection experienced high shear. A few interesting observations can be made from the moment response of this subassembly: (a) gravity load dominated the response and caused early cracking of slab at top near the connections, (b) the maximum positive moment occurred near the midspan, (c) because of the higher initial negative moments in the connection regions, the cracking of the slab under positive
FIGURE 3-11 Observed Lateral Force Distribution in Specimens
moment could only occur a certain distance away from the column face. At 2\% drift, the moment at exterior connections were thus smaller than the moment at the interior connection. Approximately, 59\% of the lateral load was carried by the interior column and the remaining was equally shared by exterior columns.

The effect of loss of anchorage on the distribution of moments can be seen in Fig. 3-11(d) which shows the observed moments in specimen DNY_3 at 3\% drift. The loss of anchorage of slab bottom reinforcement at left edge connection significantly reduced the moment transfer capacity of the connection with lateral load redistributing to the interior connection. At this drift level, the edge connection acted more like a hinge support and the maximum positive moment in the left span was only 62\% of the cracking moment of the slab in the midspan. The cracking moment in the right span occurred at 7 in. from the face of the interior column.

The presence of spandrel beams greatly improved the moment transfer capacity at edge connections in negative bending. However, the positive moment transfer at edge connection was unaffected by the presence of the spandrel beam as illustrated in Fig. 3-11(e).

3.9 Deflection of Slab

The deflection of slab due to lateral and gravity loads was measured during each test. These deflections were measured 8 in. and 24 in. away from the column line along the longitudinal
direction, respectively. The deflected shape of slab at different drift levels relative to the initial deflected shape due to gravity load alone is shown in Fig. 3-12 through Fig. 3-15.

The effect of initial gravity load on the deflected shape of the slab can be seen by comparing the slab deflections of specimens DNY_1 and DNY_2 at 2% drift as shown in Fig. 3-12. The specimen DNY_1 which carried dead load plus 30% of the live load had an antisymmetrical deformed shape indicating a dominating effect of the lateral load. The point of contraflexure was close to the midspan and the deflections were relatively small. By increasing the gravity load to full dead plus 100% of live load, the inflection point disappeared from the right span and moved close to the interior connection in the left span. The exterior connection at 2% drift acted more like a hinged support. The moment transfer at the interior connection occurred only on the side of the column with slab top in tension as indicated by negative deflection of the slab. The maximum deflection was almost six times the deflection observed in DNY_1. The initial cracking due to heavier gravity load therefore had a very significant effect on moment transfer and deflection in the slab.

Fig. 3-13 compares the deformed shape of the slab of specimen DNY_2 at 0% drift and at 2% drift. The effect of initial cracking in the slab at the interior connection and resulting initial deflections are to be noted. Lateral load increased
FIGURE 3-12  Effect of Gravity Load on Slab Deflection

FIGURE 3-13  Slab Deflection Due to Lateral Load
the deflection in the first span along the loading direction and reduced the deflection in the second span but the effect was not very significant. The higher gravity load obviously controlled the response in this specimen.

The presence of spandrel beam in specimen DNY_4 attracted more moment at edge connections which affected the deformed shape of the slab. As shown in Fig. 3-14, larger deflection change occurred near the edge connections. Because of the restraining effect of the spandrel beams, the deflection and rotation of the slab at interior connection of DNY_4 was smaller than that of the DNY_1 on both sides of the column.

The effect of anchorage loss and larger effective slab width due to the presence of spandrel beam can be seen by comparing the deflected shape of the specimens DNY_3 and DNY_4 at 4% drift as shown in Fig. 3-15. In the right span, the anchorage loss in the bottom of exterior connection of both specimens resulted in a very similar deflected shape. However, since a larger slab width participated at the edge connection of DNY_4 due to the presence of the spandrel beam, the rotation and deflection of the slab left edge was smaller compared to that observed in DNY_3.
FIGURE 3-14  Effect of Spandrel Beam on Slab Deflection

FIGURE 3-15  Comparison of Slab Deflection in DNY_3 and DNY_4
CHAPTER 4
RESPONSE OF INTERIOR CONNECTIONS

The behavior of interior slab-column connections observed during the tests is presented in this chapter. The discussion is focused on the response of slab in the connection region, failure mode, unbalanced moment transfer capacity of the connections, rotation of the slab relative to column and the degradation of stiffness with load reversals. A summary of the results is given in Table IV.

4.1 Cracking Pattern

Flexural cracks on the top surface of the slab first developed as cracks parallel to the column face which extended over a limited width in the connection region. As the drift level increased, these cracks grew both in width and length and became more inclined to the direction of loading. The final configuration of the cracking pattern on the top surface of the slab for all specimens is shown in Fig. 4-1. Except for the cracks adjacent to the column face, the other flexural cracks tended to radiate away from the column periphery.

The cracks on the bottom face of the slab were more dependent on the slab reinforcing detail and the intensity of the gravity load. Because of the presence of the negative moment, the net positive moment developed a certain distance away from the column face. The termination of the bottom reinforcing bars short of the interior column in specimen DNY_1 and DNY_4, therefore, did not affect the cracking pattern. The flexural
### TABLE IV TEST RESULTS OF INTERIOR CONNECTIONS

<table>
<thead>
<tr>
<th>INTERIOR JOINT</th>
<th>$M_m$ (k-in)</th>
<th>$M_{mf}$ (k-in)</th>
<th>$M_{un}$ (k-in)</th>
<th>$R$ (kips)</th>
<th>$R/V_o$</th>
<th>FAILURE MODE</th>
</tr>
</thead>
<tbody>
<tr>
<td>DNY_1</td>
<td>320</td>
<td>275</td>
<td>418</td>
<td>16.0</td>
<td>1.06</td>
<td>Flex</td>
</tr>
<tr>
<td>DNY_2</td>
<td>270</td>
<td>233</td>
<td>296</td>
<td>19.8</td>
<td>1.54</td>
<td>Punch</td>
</tr>
<tr>
<td>DNY_3</td>
<td>360</td>
<td>284</td>
<td>428</td>
<td>12.0</td>
<td>0.96</td>
<td>Flex</td>
</tr>
<tr>
<td>DNY_4</td>
<td>332</td>
<td>269</td>
<td>390</td>
<td>12.5</td>
<td>1.13</td>
<td>Punch</td>
</tr>
</tbody>
</table>

$M_m$: Measured maximum moment at centroid of the connection.  
$M_{mf}$: Measured maximum moment at the face of the column.  
$M_{un}$: Measured maximum unbalanced moment at connection.  
$R$: Reaction measured at the base of the column.  
$V_o$: $\sqrt{f'c'bs}d$. 


FIGURE 4-1 Cracking Pattern of Interior Connections
FIGURE 4-2  Punching Failure of Interior Connection in DNY_2
crack in specimen DNY_2 which carried a high gravity load, developed even farther away from the column face. The final failure mode is shown in Fig. 4-2. In straight bar detail, as in DNY_3, half of the slab bottom reinforcement was cut short of the column face and the remaining half continued into the column. This detail created a potentially cracking plane when the reinforcement was terminated. Due to a malfunction in the servo-control mechanism, specimen was subjected to a large initial displacement which caused a flexural crack on the bottom of the slab at the column face. The second flexural crack occurred at the cut-off point for the slab reinforcement in capital.

4.2 Strain Distribution

The distribution of strain in the slab top reinforcement at interior connections is shown in Fig. 4-3. The presence of spandrel beam at exterior connections and the configuration of slab reinforcing detail do not appear to affect the distribution of strain in the slab top reinforcement of interior connections (23,24). Unlike the seismic detail the reinforcement within the column strip is uniformly distributed. The slab top reinforcement therefore yielded over a wider region of C1+5h centered on the column line. The measured unbalanced moment was transferred at interior connection in use of C1+5h as the effective width of the slab. After the slab cracked under positive moment, the transfer of unbalanced moment took place only on the side of column
FIGURE 4-3 Strain Distribution at Interior Connection of Specimens

FIGURE 4-4 Strain Distribution at Interior Connection in DNY_4
with slab top reinforcement in tension. This further enlarged the slab effective width to approximately C₁+7h as shown in Fig. 4-4.

4.3 Failure Mode

The interior connection of DNY_2 had punching failure at 2% drift due to the high gravity load as shown in Fig. 4-2. The interior connection of DNY_4 failed at a high drift level of 5%. The interior connections of DNY_1 and DNY_3 experienced flexural failures. The location of cracks of slab bottom adjacent to the interior column was affected by the magnitude of the gravity load. Because of only small amount of slab bottom steel or no steel extended into the column, the flexural capacity of slab in positive bending is limited to the cracking strength of the slab.

4.4 Moment-Rotation

The moment-rotation response of interior connections was derived from the forces measured at the top and bottom of the interior column and the rotation of the slab over a distance of 6 in. from the column face. Fig. 4-5 and 4-6 show the moment-rotation loops for the specimens DNY_1 and DNY_2, respectively. Because of the small amount of reinforcement or no reinforcement at the bottom of the slab in the connection region, the moment transfer capacity was limited to the cracking moment of the slab in this load direction. Due to the presence of bending moment resulting from gravity load, the cracking on the bottom of the slab occurred a certain
FIGURE 4-5  Moment-Rotation of Interior Connection of DNY_1

FIGURE 4-6  Moment-Rotation of Interior Connection of DNY_2
distance away from the column face. The rotation of the slab therefore represents mostly the rotation of the slab over a distance 6 in. adjacent to the column face. The difference in the initial stiffness between two loading directions is caused by initial cracking of the slab in the connection region due to the superimposed gravity load (25). The bilinear shape of the envelope in positive bending direction with higher strength was the typical of all specimens. The moment capacity in the reversed direction depended upon the intensity of the gravity load level and thus the location of the cracking in the slab.

4.5 Stiffness Degradation

The loss of connection stiffness under lateral loading can be critical to the performance of flat-plate buildings. Because of the significantly different reinforcing detail in the top and bottom of the slab in the two loading directions, the rate of loss of stiffness in the two directions was also quite different. Fig. 4-7 shows the loss of stiffness experienced by specimens at different drift levels during the test. The stiffness is assumed as moment per unit rotation of the slab in the connection region.

The initial stiffness in the loading direction that causing tension in the slab top reinforcement was significantly higher than the initial stiffness in the loading direction causing tension in the bottom of the slab. The rate of loss of stiffness in specimens with the same gravity load was roughly
FIGURE 4-7  Stiffness Degradation of Interior Connections
the same. Almost 75% to 85% of the initial stiffness was lost by 2% drift. The loss of stiffness beyond this drift level was more gradual. Because of the initial cracking of the slab in DNY_2 due to higher gravity load, the initial stiffness of this specimen was lower. The stiffness of DNY_2 degraded in a manner similar to other specimens upto 1% drift. Beyond this drift, the stiffness of DNY_2 dropped rapidly due to the punching failure.

The stiffness degradation when slab bottom was in tension was similar developing with the rotation to the other loading direction.

4.6 Moment Transfer Capacity

The moment transfer capacity of the slab in the two loading directions was different due to the different amounts of slab top and bottom reinforcement (26). As such the two cases are discussed separately.

Slab Top in Tension: The moment causing tension in the slab top reinforcement was resisted mostly within the slab width equal to the column strip. The strain in the reinforcement within this width was much higher compared to the strain in the reinforcement in the rest of the slab. The design of interior connections required 75% of the total reinforcement to be placed within the column strip. The assumption of column strip as the effective width for calculation of bending moment gives a conservative estimate of the bending capacity. The actual moment transferred by the slab and the theoretical
flexural strength of the column strip are given in Table IV. The observed moments are about 10% higher than the flexural capacity of column strip. The slab in specimen DNY_2 did not reach its full flexural strength due to early punching failure of the connection. The moment transfer capacity in the positive moment direction is limited to flexural strength of the slab over the column strip.

**Slab Bottom in Tension:** Moment transfer capacity in the negative moment direction depended a great deal upon the reinforcing detail. In the bent-up reinforcing detail, all the slab bottom reinforcement was either bent-up or stopped short of the column. The slab, therefore, had no reinforcement at the bottom face adjacent to the column. In the region between the cut-off point and the bent-up point, the amount of reinforcement was less than the minimum reinforcement requirement. Even though the flexural cracking did not occur at the face of the column due to the presence of the gravity load, the maximum moment-transfer capacity was limited to the flexural cracking strength of the slab. In straight reinforcing detail of the slab, 50% of the slab reinforcement was terminated at 6 in. from the column face and the remaining reinforcement extended 3 in. beyond the column center line. The amount of reinforcement in the continuing into the column is typically less than the minimum reinforcement requirement and the moment-transfer capacity of the slab is therefore
limited to the slab cracking strength. Once the slab reached its cracking moment, the ability of the slab to resist moment with slab bottom in tension became negligible.
CHAPTER 5
RESPONSE OF EXTERIOR CONNECTIONS

This chapter describes the behavior of exterior slab-column connections. The response of connections is described in terms of the observed cracking pattern, strain distribution in the slab reinforcement, failure mode, stiffness and moment-rotation relationship. The ability to transfer unbalanced moment both in the positive and negative moment directions, and the effective width of the slab are discussed in greater detail.

5.1 Cracking Pattern

All specimens were subjected to gravity load before the application of lateral load. For specimen DNY_1, DNY_3, DNY_4, the gravity load included the full dead load plus 30% of the live load, while the specimen DNY_2 was subjected to full design dead and live loads. Under gravity load alone, only specimen DNY_2 experienced initial cracking. These cracks first appeared at the face of exterior columns as flexural cracks and extended as torsional cracks from inside corners of the columns to the slab edge at about 45 degree angle. During small displacement cycles, these flexural and torsional cracks elongated further with only few new cracks. By 1% drift, the flexural crack at bottom of the slab adjacent to the exterior column had extended the full width of the slab in specimens DNY_1 and DNY_4. Since only a small amount of slab bottom reinforcement extended into exterior column, the
FIGURE 5-1  Cracking Pattern of Exterior Connections

FIGURE 5-2  Flexural Yield Line at the Face of Spandrel Beam
flexural capacity of the slab at exterior connection with slab bottom in tension is limited at best to its cracking moment capacity. For straight reinforcement detail in DNY_3, the flexural and torsional cracks at exterior connections at both top and bottom present the similar pattern at 2% drift. In specimen DNY_2 which had a higher gravity load, the major flexural crack at the bottom of the slab occurred at 22 in. from the exterior column face at 2% drift. These cracks opened wider as the drift increased. The final cracking pattern at exterior connections of the four specimens are shown in Fig. 5-1. The cracking pattern at the top of the slab in specimens without a spandrel beam was very similar. Since amount of the slab bottom reinforcement extending into the exterior connections with bent-up reinforcing detail is relatively small, the cracks at the bottom of the slab adjacent to the exterior connections were especially severe. The higher gravity load in DNY_2 controlled the cracking of the slab which predominantly occurred on its top face. Because of the spandrel beam, the torsional cracks at exterior connections in DNY_4 were very minimal, however, the flexural crack on the face of the spandrel beam was significant as shown in Fig. 5-2.

5.2 Strain Distribution

The distribution of strain in the slab reinforcement when slab top in tension at first yielding is shown in Fig. 5-3. The specimen DNY_4 which had an edge beam had a more uniform
FIGURE 5-3  Strain Distribution of Exterior Connections

FIGURE 5-4  Strain Distribution at Peak Load of Exterior Connections
strain distribution compared with the other specimens. The distribution of strain in the slab at peak load for each specimen is shown in Fig. 5-4. All of the slab reinforcement yielded in specimen DNY_4 while only the rebars within a certain distance centered on the column line yielded in the other specimens. Based on the configuration of the cracking pattern at the edge connection and the variation of the observed strain, the reinforcement within a distance equal to $C_1 + C_2$ as indicated in Fig. 5-4, appears to have been fully developed in resisting the bending moment in specimens without a spandrel beam. The strain in reinforcement outside this region were below yield level and varied with each specimen. The specimen DNY_2 for example failed in punching shear before the slab could develop its full flexural strength and the strain in reinforcement was therefore significantly lower than in specimens DNY_1 and DNY_3.

5.3 Failure Mode

The failure of exterior connections consisted of two modes; flexural failure or flexural plus torsion failure depending upon whether a spandrel beam in present and anchorage failure of bottom reinforcement in exterior connections.

When slab top in tension specimen DNY_1 and DNY_3 showed the flexural plus torsion failure in exterior connections, specimen DNY_4 had flexural failure at exterior connections due to the presence of spandrel beam. When slab bottom is in tension, anchorage failure occurred in exterior connections.
of DNY_1, DNY_3 and DNY_4. The exterior connections in Specimen DNY_2 did not reach failure because of the early punching failure at interior connection.

5.4 Moment-Rotation

The anchorage of slab top and bottom reinforcement into exterior connections played a key role in the moment-rotation response of the exterior connections (27). The small amount of slab bottom reinforcement in the connection region resulted in unsymmetrical hysteresis loops, and the anchorage failure caused excessive rotation of the slab under positive bending. Typical hysteresis loops of exterior connections are shown in Figs. 5-5 and 5-6. Here, the positive direction means slab bottom in tension.

The moment-rotation response for exterior connections is also affected by the intensity of the gravity load and the presence of the spandrel beam. The effect of gravity load on rotation depended on the location of the flexural crack relative to the column face. When slab bottom in tension, the flexural cracks at bottom of the slab occurred adjacent to the column face, anchorage of slab bottom reinforcement is lost. Under heavier gravity load, flexural cracking occurred a certain distance away from the column face. In that case, the slab bottom reinforcement may not lose its anchorage. As observed from Figs. 5-5 and 5-6, the slab reached its cracking strength under positive bending and lost the anchorage of the slab bottom reinforcement, and it rotated without any
FIGURE 5-5  Moment-Rotation Response of 
Exterior Connection of DNY_1

FIGURE 5-6  Moment-Rotation Response of 
Exterior Connection of DNY_2
significant bending resistance. DNY_2 had a response typical of this situation. When flexural cracking at the bottom of the slab occurred a certain distance away from the exterior column face, the slab bottom reinforcement was not so severely stressed as in the other specimens. The moment-rotation response under negative bending, which causes tension in the slab top reinforcement is very similar to that of an underreinforced flexural member. The moment-rotation envelope is approximately bilinear with a well-defined yield point. The slope of the second segment varied among the specimens.

The moment-rotation response in the positive moment direction under dominant lateral load is of particular interest. Specimens DNY_1, DNY_3 and DNY_4 fall in this group. As shown in Fig. 5-5, the moment-rotation relation under positive bending has three distinct stages. The first stage corresponds to cracking of the slab, the second stage reflects the yielding of the slab reinforcement, and the third stage occurs when the anchorage of the slab bottom reinforcement is lost. The hysteresis loops are, therefore, highly unsymmetrical and very much dependant upon the strength of the connection under reversed bending.

5.5 Stiffness Degradation

The variation of stiffness of the four exterior connections during the loading history is shown in Fig. 5-7. These plots show the secant stiffness based on the rotation of the slab over a distance of 6 in. from the column face. Due to
FIGURE 5-7  Stiffness Degradation of Exterior Connections
significantly different flexural strength of the slab in the two loading directions, and different configuration of the slab top and bottom reinforcement, the degradation of stiffness in the two directions differed considerably.

In the loading direction which caused tension in the slab top reinforcement, the specimen DNY_2 with full dead load and live load had the highest initial stiffness followed by the specimen DNY_4 with a spandrel beam and DNY_1 with bent-up reinforcement. The specimen DNY_3 with straight reinforcement had the least initial stiffness of all specimens. About 70%-75% of the initial stiffness was lost by 1% drift in all specimens, the rate of loss of stiffness decreased beyond 1% drift, with less than 10% of the initial stiffness remaining by the end of the test for specimens DNY_1, DNY_3, and DNY_4. The loss of stiffness in the negative bending direction resulted mostly from early yielding of the slab reinforcement.

Rapid degradation of stiffness in the positive moment direction resulted primarily from the anchorage failure of the slab bottom reinforcement. As soon as the slab reached its cracking strength, the slab bottom reinforcement yielded and stiffness dropped to less than 10% of the initial stiffness. With few additional reversals, the anchorage of slab bottom reinforcement was completely lost and the stiffness of exterior connections in the positive moment direction degraded to zero for all practical purposes. The specimen DNY_2 which supported the heavier gravity load was an exception. In this specimen,
the cracking moment in the slab occurred a certain distance away from the column face and hence the loss of stiffness was more gradual.

5.6 Moment Transfer Capacity in Negative Bending

Two distinct moment transfer mechanisms were observed in the exterior connections. These are the spandrel beam case where the flexural yield line developed across the full width of the slab and the no-beam case where the moment transfer occurred through a flexural-torsional mechanism \((28,29)\). The parameters such as slab reinforcement configuration had little effect on the moment transfer response. The relative strength of the slab edge in torsion and the flexural strength of the slab controlled the mode of moment transfer between the slab and the column. Figure 5-8 shows the two mechanisms observed during testing of specimens. The first case, shown in Fig. 5-8(a) represents the spandrel beam case (DNY_4) where the cracking torsional capacity of the spandrel beam, \(T_C\), is greater than the net slab flexural capacity \((M_S - M_{C1+C2})/2\), where \(M_S\) is the flexural capacity of the entire slab width and \(M_{C1+C2}\) is the flexural capacity of the slab over \(C_1+C_2\) width as illustrated in Fig. 5-9. The moment in DNY_4 is transferred across the full width of the slab. The second type of mechanism develops with a weak spandrel beam or no spandrel beam as in specimens DNY_1, DNY_2 and DNY_3. In this case, the torsional cracking strength of the slab edge is smaller than the flexural capacity of the corresponding slab
FIGURE 5-8  Critical Section at Exterior Connections

FIGURE 5-9  Moment-Transfer at Exterior Connections
width. The moment transfer occurs through a combination of flexural-torsional mechanism as shown in Fig. 5-8(b).

With a strong spandrel beam, as in specimen DNY_4, the full width of the slab was effective as indicated by the distribution of strain in slab reinforcement. The measured value of the moment transferred at the connection and the theoretical flexural capacity of the entire slab width are given in Table IV. The average ratio of the measured and theoretical moments is 1.05 which strongly suggests the full participation of slab in exterior connections with a strong spandrel beam.

The moment transfer at exterior connections without a spandrel beam depends greatly on the torsional strength of the slab edge (30,31). The crack pattern, failure mechanism, and the strain distribution in the slab (Fig. 5-4) of specimens DNY_1, DNY_2 and DNY_3 suggest moment transfer to occur mostly over a slab width of \( C_1 + 2C_2 \), where \( C_1 \) and \( C_2 \) are the column dimension perpendicular and parallel to the loading direction. The total moment transfer capacity, therefore, consists of two components: (a) the flexural capacity of the slab width equal to or less than \( C_1 + C_2 \), (b) the torsional capacity of the slab edge on both sides of the column i.e.

\[
M_t = M_{c_1+c_2} + 2T_c
\]  
(5.1)

\( M_{c_1+c_2} \) is the slab contribution and \( T_c \) is the torsional cracking strength of the slab edge as shown in Fig. 5-9.

Since the slab edge is not reinforced with closed stirrups
for the steel to be effective in resisting torsion, the torsional capacity of the slab edge is limited to its cracking torsional strength, \( T_c \), which may be calculated by

\[
T_c = \alpha \sqrt{f_c' \frac{\Sigma x^2 y}{3}}
\]

(5.2)

where \( \alpha \) factor has a value between 4.0 and 7.0. ACI Building Code (12) conservatively adopted 2.4 (32,33) which corresponds to a torque equal to about 40% of the cracking torque of a beam without web reinforcement and implied \( \alpha \) factor of 6.0. To calculate the theoretical capacity of the slab edge, \( \alpha \) is assumed as 5.0 (34,35). A comparison of the total calculated moment transfer capacity and the moment transfer observed at the connections is given in Table V. This approach results in theoretical capacity to be approximately 90% of the measured moment transfer capacity.

5.7 Moment transfer Capacity in Positive Bending

The amount and anchorage of slab positive reinforcement at the exterior connections affected both the failure mechanism and the moment transfer capacity under positive bending. In specimens DNY_1, DNY_2 and DNY_3, which had bent-up bar configuration in the slab, 1/3 of (reinforcing ratio of bottom rebar over entire slab width is 0.11%) the slab bottom reinforcement at midspan extended into edge connections. This amount of reinforcement is less than the minimum reinforcement requirement for flexural capacity to be higher than the cracking moment capacity. The moment transfer under positive
bending is, therefore, limited to the cracking strength of the entire slab width.

With straight bar configuration in the slab, as in DNY_3, all of the slab bottom reinforcement extended to the slab edge. Thus the cracking pattern and the mechanism of moment transfer under positive bending was very similar to the one under negative bending because of the flexural-torsional mechanism of moment transfer. The maximum effective width was limited to $C_1+C_2$ as in the case with slab top reinforcement in tension. Hence the same procedure described previously could be used, with the total moment transfer capacity being equal to the flexural strength of the slab width equal to $C_1+C_2$ plus the torsional strength of the slab edge.

Depending upon the magnitude of the gravity load, the cracking positive moment of the slab in the connection region may occur a certain distance away from the column face. Under full dead and live loads, as in specimen DNY_2, the net positive moment at the face of the column was less than the cracking moment. The location where moment in the slab reached the cracking strength was approximately 20 in. from the column face. The moment transfer capacity under positive bending is therefore limited to the cracking strength of the full slab width for slabs with bent-up reinforcement detail. Since the amount of slab bottom reinforcement was relatively small, the
positive moment transfer capacity of the connection degraded rapidly. Based on this approach the test results compare very well with the calculated capacities as shown in Table V.

5.8 Anchorage Performance

Approximately 1/3 to 1/2 of slab bottom reinforcement at midspan was extended into exterior connections in specimen DNY_1, DNY_2 and DNY_4. The embedment length of these rebars, measured from the column face, was 6 in. To evaluate anchorage performance, two strain gages were attached to the bar, one at the column face (point 1) the other at 1.5 in. (point 2) from end of the bar. The observed variation of strain for specimen DNY_1 at these two location is shown in Fig.5.10. Before the slab reached its cracking strength at the column face, the strain at point 2 was nearly zero. When the slab developed a main flexural crack main at 1.5% drift, the point 1 reached the yield strain, and the strain level increased gradually.

Under continued load reversals, the bar lost its anchorage at 2.25%. The deterioration of bond at the column face extended farther into the column as drift increased. At this stage, the bond was completely lost and the bar could only resist a small force due to friction between the steel and concrete surface. The slippage of the bar resulted in a considerable increase in the rotation of slab relative to the column and a rapid loss of stiffness. The rotation of slab at 2.25% drift was approximately 2.5 times the rotation at first yield.
<table>
<thead>
<tr>
<th>Test Results</th>
<th>Theoretical Results</th>
<th>Neg. M.</th>
<th>Pos. M.</th>
<th>Test/Theory</th>
</tr>
</thead>
<tbody>
<tr>
<td>M&lt;sub&gt;eff&lt;/sub&gt;</td>
<td>Neg. M.</td>
<td>Pos. M.</td>
<td>M&lt;sub&gt;eff&lt;/sub&gt;</td>
<td>M&lt;sub&gt;eff&lt;/sub&gt;</td>
</tr>
<tr>
<td>(k-in)</td>
<td>Width</td>
<td>Width</td>
<td>(k-in)</td>
<td>(k-in)</td>
</tr>
<tr>
<td><strong>EXT.</strong></td>
<td><strong>JOIN</strong></td>
<td><strong>DNY_1W</strong></td>
<td><strong>DNY_2E</strong></td>
<td><strong>DNY_2W</strong></td>
</tr>
<tr>
<td>M&lt;sub&gt;neg&lt;/sub&gt;</td>
<td>158</td>
<td>C1+2C2</td>
<td>125</td>
<td>C1+2C2</td>
</tr>
<tr>
<td>M&lt;sub&gt;pos&lt;/sub&gt;</td>
<td>177</td>
<td>C1+2C2</td>
<td>132</td>
<td>C1+2C2</td>
</tr>
<tr>
<td>Ratio</td>
<td>0.85</td>
<td>0.90</td>
<td>0.97</td>
<td>0.94</td>
</tr>
</tbody>
</table>

- **M<sub>neg</sub>**: Negative moment (slab top in tension)
- **M<sub>pos</sub>**: Positive moment (slab bottom in tension)
- **M<sub>eff</sub>**: Moment transfer capacity of entire slab width
- **M<sub>eff</sub>**: Moment transfer capacity of exterior column
- **Neg. M.**: Moment transfer capacity around exterior column in slab bottom
- **Pos. M.**: Moment transfer capacity in the column face
- **Ratio**: Ratio of test to theoretical values

* : No cracks around exterior column in slab bottom
FIGURE 5-10 Variation of Anchorage Strain With Drift
CHAPTER 6
SHEAR STRENGTH OF CONNECTIONS

The punching failure of slab-column connections has been the cause of many catastrophic failures in flat-plate buildings (36). Particularly, in older buildings where design was based on gravity load consideration only, the punching strength of slab-column connections under earthquake type of loading can be of serious concern as there is no mechanism to protect against progressive collapse of floors. This chapter addresses the shear strength of slab-column connections subjected to earthquake type loading.

6.1 Interior Connections

Among the specimens tested, the specimens DNY_2 and DNY_4 experienced punching failure at interior connections. The higher intensity of gravity load in DNY_2 and the presence of the spandrel beam in DNY_4 were the key factors that caused punching failure in these specimens. The other two specimens had flexural mechanism at the interior connections.

Moments at the interior connections calculated on the basis of measured reactions are given in Table IV. Since the specimens DNY_1 and DNY_3 did not fail in punching shear capacity, the slab was able to develop its full flexural capacity, and the connections could transfer a higher unbalanced moment. Axial force in the column measured as reaction at its base is also given in Table IV. This force represents the direct shear component in the connection region. The direct shear $P$, is
normalized with respect to the factor $\sqrt{f_c \cdot b_o \cdot d}$, where $b_o$ is the perimeter of the critical section at distance $d/2$ from the column face and $d$ is the effective depth of the slab. The resulting ratio are shown in the last column of Table IV. The specimen DNY_2 with a direct shear ratio 1.52 had a very clear early punching failure. Specimen DNY_4 with a direct shear ratio of 1.13 was at the borderline with shear failure occurring late in the test at 5% drift. From these observations, the direct shear at interior connections appears to be a key factor affecting the mode of failure. For symmetrical structures, the direct shear may be approximately by gravity load shear (37,38). Hence, for the convenience of calculations, the direct shear is replaced by gravity load shear, $V_g$, which can be easily determined from gravity loads.

The shear component generated by the unbalanced moment is determined by using the linear shear stress model used in the current ACI Building Code [12]. However, the effective width of the slab for moment transfer must be established, first. It is to be noted that in the gravity load design of the class of flat-slab buildings under consideration, the transfer of unbalanced moment is not considered in the design process and the slab top reinforcement in the connection region is distributed uniformly throughout the width of the column strip. The observed distribution of strain in the slab reinforcement, shown in Fig. 4-3, suggests a slab width most effective in transferring moment to be equal to the column
<table>
<thead>
<tr>
<th>INTERIOR JOINT</th>
<th>$\nu_g$ (psi)</th>
<th>$\nu_{un}$ (psi)</th>
<th>$\nu$ (psi)</th>
<th>$\alpha \sqrt{f'_c}$ (psi)</th>
<th>$\frac{\alpha \sqrt{f'_c}}{\nu}$</th>
<th>$\frac{M_{mf}}{M_n}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>DNY_1</td>
<td>76.2</td>
<td>63.0</td>
<td>139.2</td>
<td>264</td>
<td>1.89</td>
<td>1.07</td>
</tr>
<tr>
<td>DNY_2</td>
<td>94.4</td>
<td>120.0</td>
<td>214.4</td>
<td>209</td>
<td>0.98</td>
<td>0.85</td>
</tr>
<tr>
<td>DNY_3</td>
<td>57.1</td>
<td>64.6</td>
<td>121.7</td>
<td>201</td>
<td>1.68</td>
<td>1.10</td>
</tr>
<tr>
<td>DNY_4</td>
<td>59.6</td>
<td>99.1</td>
<td>158.0</td>
<td>154</td>
<td>0.99</td>
<td>0.92</td>
</tr>
</tbody>
</table>

$v_g$: Shear stress caused by gravity shear force.  
$v_{un}$: Shear stress caused by unbalanced moment transfer.  
$v$: $v_g + v_{un}$.  
$\alpha \sqrt{f'_c}$: Shear strength.  
$M_{mf}$: Measured maximum moment at face of column.  
$M_n$: Calculated moment capacity.
width plus five times the slab depth \((C_2+5h)\). Most of the unbalanced moment at 2\% drift was transferred through this slab width.

The shear strength of the interior connection is determined on the basis of shear capacity observed in specimen DNY_2 which experienced a distinct punching shear failure at the interior connection. Following the procedure contained in the present ACI Building Code, the shear strength is assumed to be a combination of gravity shear plus a shear component from unbalanced moment transfer as given by

\[
V = \frac{V_u}{A_c} + \frac{\gamma_v M_{un} C_{ab}}{J_c}
\]  

(6.1)

Where \(\gamma_v\) represents the portion of the unbalanced moment transferred through eccentric shear with the remaining unbalanced moment transferred through bending. The flexural capacity of the slab effective width \(C_2+5h\) is about 64\% of the unbalanced moment transferred at interior connection of DNY_2. The portion of the unbalanced moment transferred through shear could therefore be assumed approximately as 40\%, the same as used in the present ACI Building Code, i.e. \(\gamma_v=0.40\). The ultimate shear \(V_u\) is taken as the column reaction \(P\) given in Table IV. Using the usual geometric properties as defined in the present ACI Building Code [12], the maximum shear stress on the critical section is obtained as

\[
\nu = \frac{19.8 \times 1000 + 0.4 \times 296 \times 1000 \times 6.9}{209.76 + \frac{6784}{6784}} = 214.4 \text{ psi}
\]
which expressed as a function of $\sqrt{f_c'}$ is equal to $3.5\sqrt{f_c'}$. The shear strength of the interior connection for specimen DNY_2 could therefore be taken as $3.5\sqrt{f_c'}$.

Higher gravity load in specimen DNY_2 resulted in a narrower effective slab width for unbalanced moment transfer. However, under normal service load conditions, as in DNY_4, the strain distribution shown in Fig. 4-4 suggests a larger slab width effective in transferring the unbalanced moment. In this case, the slab effective width can be estimated as $C_{2+7h}$. Unlike in DNY_2, the flexural cracking occurred adjacent to the column face which reduced the effective shear area in DNY_4. Therefore, more moment is transferred through bending in DNY_4 compared to the DNY_2. The $C_{1+7h}$ width of the slab at 5% drift is found to transfer 75% of the unbalanced moment with the remaining 25% transferring through eccentric shear. The $\gamma_v$ factor for the case where bottom flexural cracking developed at the face of the column, is therefore 0.25 instead of 0.40. Following the procedure above, shear strength of interior connection of DNY_4 can be calculated from Equation (6.1) as

$$\gamma_v = \frac{12.5 \times 1000 + 0.25 \times 390 \times 1000 \times 6.9}{209.76 + 6784} = 158.7 \text{ psi.}$$

which in term of $\sqrt{f_c'}$ becomes $3.0\sqrt{f_c'}$ psi.

The difference in the shear strength of the two connections $3.0\sqrt{f_c'}$ vs. $3.5\sqrt{f_c'}$ suggests that the shear capacity of the connections is not a constant quantity but it is affected by the previous loading history including the drift level,
intensity of the gravity load, and yielding of the slab reinforcement in the connection region. The connections that experience large deformation reversals lose the in-plane confinement due to flexural cracking in the connection region and excessive yielding of the slab top reinforcement. Smaller gravity load allows larger drift levels while the larger drift levels result in reduced shear capacity. The observed relationship between the shear strength factor $\alpha$ in $\alpha \sqrt{f_c'}$ and the corresponding ultimate drift is shown in Fig. 6-1. The shear capacity at a given drift level may be related to the drift level as

$$\alpha = 3.8 - 0.16 \Delta$$  \hspace{1cm} (6.2)

where $\Delta$ is the drift value (%).

The shear strength factor $\alpha$ of 3.8 represents a conservative estimate of punching shear capacity under pure gravity load and it reduces by 16% for each 1% increase in the drift level.

As mentioned earlier, the deformation capacity of the connection is dependent upon the level of gravity load. The observed relationship between drift and gravity shear ratio $V_g/\sqrt{f_c'}b_o d$ is shown in Fig. 6-2. The specimen DNY_2 which was subjected to 100% dead plus live load reached a drift of 2% with a gravity shear ratio of 1.54. The specimen DNY_4, which had full dead load plus 30% of the live load reached 5% drift under gravity shear ratio of 1.13. It is apparent that the level of gravity load has a profound effect on the deformation
capacity of the interior connections. Based on the gravity shear vs. drift response of these two specimens and the previously established shear capacity of $3.8\sqrt{f_{c'}}$ under pure gravity load, the drift capacity may be related to the gravity shear by

$$
\Delta = \left( \frac{V_g}{\sqrt{f_{c'}b_o d}} \right)^2 - \left( \frac{6.0}{\sqrt{f_{c'}b_o d}} \right) + 0.73
$$

(6.3)

This relationship implies that drift capacity increases rapidly for gravity shear ratio of less than 1.50.

A relationship between shear strength factor $\alpha$ and gravity shear ratio $R = V_g/\sqrt{f_{c'}b_o d}$ can be established by combining Equations (6.2) and (6.3) as

$$
\alpha = 3.8 - 0.16 \left( \left( \frac{12.2}{\sqrt{f_{c'}b_o d}} \right)^2 - \left( \frac{6.0}{\sqrt{f_{c'}b_o d}} \right) + 0.73 \right)
$$

$$
\alpha = 3.68 - \frac{1.95}{\left( \frac{V_g}{\sqrt{f_{c'}b_o d}} \right)^2} + \frac{0.96}{\left( \frac{V_g}{\sqrt{f_{c'}b_o d}} \right)}
$$

(6.4)

The relation between $\alpha$ and gravity shear ratio $R$ is graphically shown in Fig. 6-3. Equation (6.4) can be further approximated by a linear relationship of

$$
\alpha = 2.31 + 0.39 \frac{V_g}{\sqrt{f_{c'}b_o d}}
$$

(6.5)

with the following bounds:

$0 \leq \Delta \leq 7.0$
FIGURE 6-1  Shear Strength vs. Drift Relationship

FIGURE 6-2  Gravity Shear Ratio vs. Drift Relationship

FIGURE 6-3  Shear Strength vs. Gravity Shear Ratio Relationship
\[ 1.0 \leq \frac{V_o}{\sqrt{f_{c}^{'}} b_c d} \leq 3.8 \]

\[ 2.7 \leq \alpha \leq 3.8 \]

The shear stress factor \( \alpha \) has commonly accepted bound of 2.0 for these beam shear and 4.0 for slab punching shear. Within this limits, the linear approximation appears to be reasonable. To ensure a drift of at least 2\%, the gravity shear ratio must be kept less than 1.54 which is equivalent to shear stress factor of 3.23.

The calculated values of direct shear and shear resulting from unbalanced moment for all interior connections are given in Table VI. The calculated lower shear stress and high moment transfers in DNY_1 and DNY_3 are in conformity with the observed flexural mode of failure in these specimens.

6.2 Exterior Connections

The behavior of exterior connections was typically characterized by a combination of bending and torsional action in all specimens (19). Exterior connections lost their stiffness quite rapidly, hence the punching shear failure under lateral load was not possible. The shear acting on the critical section of an exterior connection, measured as column reaction during the test, includes shear resulting from both gravity and lateral loads. The gravity load shear did not change much during the test from that measured at the initial condition under pure gravity load (37,38). The increase in
shear therefore resulted mostly from the lateral load. The variation of total shear with drift is shown in Fig. 6-4. The shear is normalized with respect to $\sqrt{f_c' b_o d}$, where $b_o$ is perimeter of the critical section assumed at distance $d/2$ from the column face. The first point of each curve at zero drift corresponds to shear caused by gravity load only. The gravity shear ratio at the initial condition varied between 0.6 and 0.9.

The specimen DNY_2 which resisted heavier gravity load is of particular interest since it experienced punching failure at the interior connection. The shear under lateral load increased by 50% at 2% drift at which the interior connection failed in punching shear. At this drift level the shear stress reached $1.3\sqrt{f_c'}$ which is relatively low to cause a punching failure.

The increase in shear due to lateral load in the other three specimens was smaller. As the exterior connection developed a flexural-torsional mechanism, the load redistributed to interior connection and the net shear at the exterior connection decreased. Since none of the exterior connections failed in punching, the punching shear strength of the exterior connections could not be measured. However, it is clear that if the gravity load shear is limited to $0.8\sqrt{f_c' b_o d}$, at least 3.5% lateral drift can be achieved without a punching failure. With gravity shear of $0.9\sqrt{f_c' b_o d}$, the drift of at least 2% is achievable.
FIGURE 6-4 Measured Shear Force Variation With Drift at Exterior Connections

FIGURE 6-5 Measured Shear Force With Unbalanced Moment at Exterior Connections
The variation of observed shear and moment at the exterior connections of specimens is shown in Fig. 6-5. Shear is normalized with respect to $\sqrt{f_c \cdot b_o d}$ and moment is normalized with respect to flexural capacity of the exterior connection as defined in Chapter 5. The relationship between shear and moment of all specimens appears to follow a similar trend. In all cases, the moment increased faster than the shear did. The difference in the initial shear and moment are due to different gravity load and different initial stiffness. The specimen DNY_4 with spandrel beam carried more initial shear due to higher stiffness at the exterior connection and the specimen DNY_2 had higher initial shear and moment due to larger gravity load. As noted earlier, none of the exterior connections failed in shear and the moment transfer was affected by the rate of stiffness degradation. The specimen DNY_2 did not reach its moment capacity because of the punching failure at the interior connection. Once the interior connection punched, the load redistributed to the exterior connections and the shear increased rapidly. But, in none of these specimens, the shear could reach a high enough level to cause a punching failure. The net shear due to gravity load and moment transfer reached as high as $1.3\sqrt{f_c \cdot b_o d}$ in specimen DNY_2 which had the highest gravity load. It can therefore be concluded that for initial gravity shear at exterior connections of $0.9\sqrt{f_c \cdot b_o d}$ and less, the response is governed primarily by bending and torsion of the slab edge.
Under positive bending, the net shear on the critical section is reduced due to the opposing effects of gravity shear and the lateral load. Also, the net moment at the column face depends on the relative magnitude of gravity load and lateral load moments. As observed during the testing of specimens, the possibility of a failure due to a net positive moment at the column face under normal loads is relatively small. Even if the slab cracked in positive bending, the net shear area is sufficient to resist the low level of shear at exterior connections.

6.3 Analytical Model For Connection Strength

The prediction of connection strength in the previous section was based on the linear shear stress model specified in the current ACI Building Code. Based on the test results, the allowable shear stress and moment transfer factors were adjusted to reflect the non-ductile detail of the slab-column connections. Attempt is made in this section to further modify the linear shear stress model to more realistically represent the unsymmetrical cracking pattern at the interior connections by changing the effective shear area of the critical section (39).

Interior Connections: Test results have shown that the moment transfer predominantly occurred on the column face subjected to negative moment with slab top reinforcement in tension. On the opposite face of the column, the slab under positive moment has at best a moment transfer capacity equal
to the cracking strength of the slab. Depending on the magnitude of the gravity load, the flexural cracks under positive bending were observed to occur between 0.25d and d distance from the face of the column where d is the slab effective depth. The presence of such a crack reduces the net area available for shear resistance on that face. Consequently, most of the shear and moment at interior connection is transferred through the remaining three faces of the slab critical section only. The eccentric shear stress model for connections could therefore be modified to reflect this behavior of non-ductile slab-column connections.

In order to simplify the shear model, it is assumed that (a) the flexural crack on the bottom of the slab is located at a distance d/2 from the column face, and (b) the shear and bending capacity of the cracked slab under positive bending is negligible. The modified slab critical section is shown in Fig. 6-6. The critical section may be considered as fixed on three faces and hinged on the face with slab under positive moment. A finite element analysis of plate with such boundary conditions and subjected to gravity load and moment at the center indicates that only 8% of the direct shear and 7% of the shear from the moment is transferred through the hinged edge. Conservatively, the shear and moment transfer at the slab edge subjected to positive bending may be ignored. The net effective critical section for shear stress calculations may therefore be assumed to consist of three sides as shown
FIGURE 6-6 Shear Stress Distribution With Hinge Support on One Side of Critical Sections

FIGURE 6-7 Proposed Critical Section for Shear at Interior Connections
in Fig. 6-7. The eccentric shear model can then be applied to this critical section to determine the maximum shear stress in the connection region.

As discussed earlier, the effective width for unbalanced moment transfer is taken as \( c_1+5h \), with 25% of the unbalanced moment being transferred through eccentric shear. The geometric properties of such a critical section are very similar to that of the exterior connections used in the ACI Building Code (12). The area of the critical section is

\[
A_c = d(2(c_1 + d) + c_2 + d) \tag{6.6}
\]

\[
J_c = d \left( \frac{2a^2}{3} - (2a + b)x_2^2 \right) + \frac{ad^3}{6} \tag{6.7}
\]

where

\( a = c_1 + d \)

\( b = c_2 + d \)

and \( x_2 \) is distance from center of critical sections to the edge.

Shear stress can then be calculated in a usual manner by

\[
\nu = \frac{V_s}{A_c} + \frac{\gamma_b M_{wa} C_{ab}}{J_c} \tag{6.8}
\]

The calculated values of shear stress by this procedure are compared with the measured values in Table VII. The \( \nu \) of DNY_4 is a little larger than the shear strength determined by the linear relation shown in Fig. 6-1. The maximum shear stress \( S = \nu \) value for DNY_1, DNY_3 is less than the shear
strength and the bending dominated the response.

**Exterior Connections:** The response of exterior connections was typically controlled by a combination of bending plus torsion of slab edge. The unbalanced moment transfer strength of exterior connections is:

\[ M_{un} = M_{c1+c2} + 2M_t \]  \hspace{1cm} (6.9)

\( M_{c1+c2} \) is the moment strength of the slab width \( c1+c2 \) centered on column line and \( M_t \) is torsional strength of the slab edge.

However, if a spandrel beam is present which changes the moment transfer to a pure flexural mechanism, then the unbalanced moment transfer capacity is equal to the flexural strength of the full slab width.

Since none of the exterior connections failed in punching, the shear strength of exterior connections could not be predicted on the basis of test results. From the test observations, the likelihood of a punching failure occurring at the exterior connections appears to be quite small.
CHAPTER 7
HYSTERETIC MODEL OF CONNECTIONS

Analytical modeling of non-ductile slab-column connections for evaluating the seismic response of flat-plate buildings needs to account for the behavior of connections observed during tests. The model must reflect stiffness degradation, moment-rotation characteristics, and the unsymmetrical nature of moment-response in the two loading directions. An analytical model for non-ductile connections is proposed that accounts for the above mentioned response parameter.

7.1 Background

In inelastic analysis, proper consideration of a hysteretic model is one of the critical factors in successfully predicting the dynamic response of the structure under earthquake motions. While most research on hysteresis models is focused on beam-column connections, the research on hysteresis models of slab-column connections is rather limited. Hwang and Moehle (40) investigated the moment-rotation hysteresis loops of slab-column connections based on the experimental work, and indicated that the elastic (uncracked) moment-rotation stiffness of slab-column connections is a linear function of column dimension and slab span (41). Yun Ding (42) modified the Clough model (43) and adapted the moment-rotation of beam-column connections to slab-column connections. Bending strength and stiffness variation, as well as the failure mode of slab-column connections under cyclic loading, were included
in his model.

7.2 Envelope of Hysteresis Loops

**Interior connections:** The envelope of moment-rotation loops as observed during tests is shown in Figs. 7-1 through 7-4. The moment that causes the tension in the slab top reinforcement is defined as positive and the measured rotation represents the rotation of the slab over 6 inches relative to the column face. The reinforcement amount and detail in the connection region had a significant effect on the envelope in the two loading directions.

Unlike in beam-column connections, the ascending part of the curve in slab-column connections has no distinct cracking point. When the slab-top reinforcement begins to yield, the ascending part of the curve begins to diverge from the initial slope due to reduction in the stiffness of the connections. With the increase of lateral displacement, the concrete in the compression area begins to crush and the peak level is reached. After the peak load is reached, the strength begins to degrade, as indicated by the negative slope of the envelope at larger drift levels. Overall, the envelope in the positive loading direction approximates a bilinear response due to a relatively small amount of the reinforcement ratio in the slab.

In the reversed moment direction, the combination of negative moment caused by the gravity load with positive moment from lateral load determined the moment-rotation response. In that
FIGURE 7-1 Moment-Rotation Envelope of DNY_1

FIGURE 7-2 Moment-Rotation Envelope of DNY_2
FIGURE 7-3  Moment-Rotation Envelope of DNY_3

FIGURE 7-4  Moment-Rotation Envelope of DNY_4
respect, cracking strength of the slab and the location of the flexural crack relative to the column face played an important role. Once the slab was cracked under moment which caused tension at the bottom of the slab, no further moment could be transferred and the slab rotation became more like that of a hinge. For a simplified representation of moment-rotation in this direction, a linear relation is used with the slope same as that in the positive direction and peak strength limited to cracking strength of the slab.

**Exterior connections:** As shown in Figs. 7-5 through 7-8, the moment-transfer capacity for the case with slab-top reinforcement in tension was much higher than in the other direction. The response in this direction might be modeled as a bilinear relationship. When the slab bottom reinforcement was in tension, all specimens showed a similar behavior which could be modeled as in the case of interior connections.

When the top was in tension the moment-rotation envelope presented different forms for each specimen. On ascending part of DNY_1 a "yield platform" appeared at 80% maximum moment as shown in Fig.7-5. With the development of deformation, the moment resistance smoothly reached the peak point. With the increase of deformation after peak load, the moment resistance descended. Because most deformation was generated in the yield platform, the bilinear relation is proposed to model the variation of moment with rotation.

The envelopes of moment-rotation for DNY_3 and DNY_4 in
FIGURE 7-5  Moment-Rotation Envelope of DNY_1

FIGURE 7-6  Moment-Rotation Envelope of DNY_2
FIGURE 7-7  Moment-Rotation Envelope of DNY_3

FIGURE 7-8  Moment-Rotation Envelope of DNY_4
positive direction were similar to a trilinear relation due to early crack opening. The first break point appeared at about 50% of maximum moment. Due to cracking, the bending stiffness of the connection was reduced to 1/2 - 1/3 of initial stiffness. Whereupon the reduction of moment transfer capacity developed smoothly with the increase of rotation deformation. In order to simplify the envelope, the trilinear is replaced by the bilinear in which only one line connected between origin and yield point is placed. The secant stiffness is used as the ascending stiffness conservatively.

When the bottom was in tension, the cracking strength of critical section dominated because of under reinforcement. Once the crack strength was reached, the moment transfer was dropped significantly. Even though a few reinforcements may resist a certain amount of moment, anchorage failure caused the reinforcements to lose action completely under a couple of cyclic loadings. Therefore, the moment-rotation behavior on this direction is close to a linear relation.

7.3 Proposed Moment-Rotation Model

The testing envelopes of moment-rotation represent the rotation deformation under total moment. The basic characteristics include: (a) an approximate bilinear primary curve in the loading direction that causes negative moment in the slab, (b) a maximum moment capacity equal to cracking moment of the slab on effective width in the opposite loading direction, beyond which the moment capacity drops to nearly
zero, (c) stiffness degraded with increased deformation level, and (d) lower unloading stiffness.

A modified model is suggested to express the moment-rotation behavior of the connections as shown in Fig.7.9. The five rules in hysteresis systems are as follows:

Rule (1) Elastic stage.

Loading: \( M < M_Y \) Stiffness \( K_1 \)=slope of OA

\[ M = M_Y \] Goto rule 2

Rule (2) \( M = M_Y \) Stiffness \( K = 0 \)

Unloading \( K_2 \)=slope of BC. Goto rule (3)

Rule (3) If \( M = 0 \) \( M_b > M_C \) Goto rule (5)

If \( M = 0 \) \( M_b \leq M_C \) Goto rule (4)

(\( M_b \) is historically maximum moment when bottom in tension.)

Rule (4) Reloading from C to D

Unloading Goto rule (1).

Rule (5) \( M = 0 \) reloading from point E

The several parameters involved in this model, such as ascending stiffness \( K_1 \), yielding strength \( M_Y \), and descending stiffness \( K_2 \) are directly related to the geometric behavior of a slab-column connection, material strength, reinforcement ratio and detail, loading type, and history.

7.4 Parameter Determination

Loading Stiffness \( K_1 \): The effective stiffness of connections, as the secant stiffness of the moment-rotation curve, is a function of parameters, as mentioned above (43). These parameters should be directly or indirectly reflected in the
FIGURE 7-9 Proposed Hysteresis Model
model.

The relationship between the secant stiffness and the applied moment for the connections of test specimens is shown in Figs. 7.10 through 7.13. The secant stiffness is normalized with respect to the bending stiffness of the entire slab width while the moment is normalized with respect to the maximum moment reached at the connections.

As noted in stiffness vs. moment plots, the relationship has two distinct regions. In the initial portion the stiffness drops rapidly. The second part is relatively flat with smaller loss of stiffness. The exterior connections in DNY_2 lost 55% of its initial stiffness when the net applied moment was 60% of the moment strength of the connections. The loss of stiffness in DNY_4 was more dramatic due to the development of a major flexural crack at the face of the spandrel beam. It lost 75% of the initial stiffness when the net moment increased to 40% of the moment transfer capacity of the connection. After these moments were reached, the stiffness stabilized with a relatively small rate of stiffness loss. For interior connections, the stiffness-moment relation also has two distinct regions. For DNY_1 and DNY_2, these break points occurred approximately at 55% and 65% moment ratios, respectively.

Based on the mathematical analysis between the stiffness and parameters, the expression of secant stiffness (21, 22, 23) is derived. The effectiveness of gravity load on the
FIGURE 7-10  Stiffness-Moment Response of DNY_4

FIGURE 7-11  Stiffness-Moment Response of DNY_1
FIGURE 7-12  Stiffness-Moment Response of DNY_2

FIGURE 7-13  Stiffness-Moment Response of DNY_4
stiffness is presented as a factor $R_e$.

For interior connections or exterior connections without a spandrel beam:

$$K = R_e \left( 2.0 - \frac{l_c}{I_g} \left( 7.6 \frac{M}{M_o} - 4 \left( \frac{M}{M_o} \right)^2 \right) \right) K_o$$  \hspace{1cm} (7.1)

For exterior connections with a spandrel beam:

$$K = R_e \left( 1.5 - \frac{l_c}{I_g} \left( 5.4 \frac{M}{M_o} + 7 \left( \frac{M}{M_o} \right)^2 - 3.0 \left( \frac{M}{M_o} \right)^3 \right) \right) K_o$$  \hspace{1cm} (7.2)

$$0 \leq K \leq K_o$$  \hspace{1cm} (7.3)

$$0 \leq M \leq M_o$$  \hspace{1cm} (7.4)

$K$: Secant stiffness

$K_o$: Bending stiffness of gross section of entire slab width

$R_e$: Gravity load factor

$M$: Current bending moment

$M_o$: Bending strength of the connection

$I_{cr}$: Cracking moment of inertia of effective slab width.

$I_i$: Inertia moment of whole slab width.

Gravity load factor varies with initial gravity load ratio, according to our test result,

For interior connections, if

$$\frac{v_g}{\sqrt{f_{ce} b_s d}} \geq 1.5, \hspace{0.1cm} R_e = 0.5.$$  

For exterior connections without a spandrel beam, if

$$\frac{v_g}{\sqrt{f_{ce} b_s d}} \geq 0.9, \hspace{0.1cm} R_e = 0.5.$$  

For exterior connections with a spandrel beam, if

$$\frac{v_g}{\sqrt{f_{ce} b_s d}} \geq 1.2, \hspace{0.1cm} R_e = 0.5.$$
Otherwise, \( R_e = 0.8 \).

This proposed model has taken into account the effect of applied moment on the variation of stiffness. Indirectly it also considered the most important factors in the performance of flat-plate systems, such as sizes of plates and column, reinforcement ratio and arrangement, bending moment, and moment inertia.

Based on the equations above, the corresponding secant stiffness can be determined once the bending moment is known. The \( K_1 \) stiffness in the bilinear model of moment-rotation proposed is related to the yield moment at which the ascending part ends. The yielding moment as mentioned before varies between 80% and 90% of the bending capacity of this connection. 85% of bending moment capacity of the connection is regarded as the yield moment to determine the \( K_1 \) values by Eq.7.1.

**Unloading Stiffness \( K_2 \):** The typical unloading part of moment-rotation loops is shown in Fig.7.14. \( K_2 \) of each loop is taken as the slope between points \( P_1 \) and \( P_2 \). The \( P_1 \) shown as the peak point of the typical loop and the point \( P_2 \) is the point of 20% or 10% of the peak value depending on the location of break-point in the declining part. The actual \( K_2 \) value depends on the loading history. With the increase of loading cycles, the \( K_2 \) decreased. \( K_2 \) varied from 0.5\( K_1 \) to 0.8\( K_1 \), and the average value was around the 0.7\( K_1 \). The straight line between two points is able to match the declining curve closely. The \( K_2 \) as a constant, \( K_2 = 0.7K_1 \), is proposed.
FIGURE 7-14 Unloading Stiffness $K_2$
CHAPTER 8

INELASTIC RESPONSE OF NON-DUCTILE FLAT-PLATE BUILDINGS

Flat-plate structures are commonly analyzed by equivalent frame methods. Plate elements in the structure are modeled by equivalent beams. Thus the analysis of a plate-column frame is thus similar to that of a beam-column frame (42,44) except for the possibility of punching failure at connections. The hysteresis model should also represent the moment-rotation relationship of slab-column connections. In order to study the response of non-ductile flat-plate buildings under the action of earthquake motion, two typical structures were analyzed by a modified DRAIN-2D program (45). The nonlinear dynamic analysis program was modified to account for the slab-column connection behavior in a previous study (42). The modified part includes the effect of punching failure and the slab-column connection hysteresis relationship. The flat-plate structure analyzed with this program is shown in Fig.8.1. The design of this structure is given in Appendix A. The shear and flexure capacities of connections were determined by the methods described in earlier chapters. The moment-rotation model, as shown in Fig.7.9, was incorporated into the program to reflect the observed behavior of connections. The ground motion was selected from earthquake records of the Mexico City earthquake of 1985 (1) shown in Fig.8.2.

During the analysis, the moment and shear at the ends of
FIGURE 8-1 Analytical Model of a Flat-plate Building
Mexico City 1985 Earthquake

FIGURE 8-2 Earthquake Record Used in Analysis
elements could be found at every time step. If the negative bending moment exceeded the yield capacity without punching at a connection, a plastic hinge was inserted in the slab near the column face. When the positive bending moment exceeded the slab cracking moment, moment transfer capacity was lost in this direction and a concrete hinge was inserted in the slab bottom. However, if the connections reached their punching capacity, the joint was considered to have lost its moment transfer capacity entirely. In the case of flexural yielding, the joint must be checked for both yielding and punching shear during the next time step. However, if punching occurred, the yielding and shear of connections were not examined during the next time step.

The following assumptions are made in modeling the structure:

(a) the floor mass is associated only with the horizontal displacements at the floor levels,

(b) each span of slab between two columns is represented by a single modified reinforcement concrete beam element,

(c) element lengths from center to center of joints are assumed, but the rigid zones of joints are taken into account so that the moments and shear forces computed represent those at column faces instead of at the center of columns,

(d) p-delta effect is considered for columns, and

(e) the mass proportional damping is assumed as 5% of
critical damping.

As used in the testing program, the live load levels of 30% and 100% of the full live load were studied in the analysis.  

8.1 Flexural Failure Mode  

As the test results indicated, flexural mode dominated the response under dead load, plus 30% of the live load inspection of the slab reinforcing arrangement. The following example illustrates such a response.

Since the building was designed to resist gravity load only, the reinforcement is not sufficient to resist the lateral load plus the gravity load. When the building roof displacement reached a drift of 0.33%, most connections reached their flexural yielding strength, forming plastic hinges formed as shown in Fig.8.3. The corresponding deformed shape is shown in Fig.8.4. Due to the formation of flexural hinges above the second floor, the maximum relative drift occurred between the second and third floor. The formation of plastic hinges in the slab resulted in significant drop in lateral stiffness. Deformed shape following the formation of flexural hinges is shown in Fig.8.5(a). The relative drift of each floor is shown in Fig.8.5(b), it indicated that the drift value of the lower floor increased greatly after the formation of flexural hinges. The connections of anchorage failure of bottom reinforcements after the formation of plastic hinges is shown in Fig.8.6. The anchorage failure caused the rotation deformation to increase significantly, corresponding to a
FIGURE 8-3 Flexural-hinge Locations of 0.33% Drift

FIGURE 8-4 Lateral Displacement at 0.33% Drift
FIGURE 8-5(a) Floor Displacement at 0.82% Drift

FIGURE 8-5(b) Relative Drift of Floors at 0.82% Drift
FIGURE 8-6 Hinge Locations at 0.82% Drift

FIGURE 8-7 Final Failure Mode at 1.5% Drift
small or almost zero moment resistance. The anchorage failure is defined as a concrete hinge. When the roof drift reached 1.5%, all connections formed hinges. This failure feature is shown in Fig.8.7. The deformed shape comparison of two structures, one old building designed by gravity load only, and one designed by designed by current ACI Code (12), is shown in Fig.8.8(a). The roof displacement of the old building is 1.8 times that of the current building. The relative drift between each floor is shown in Fig.8.9. Maximum relative drift 3.5% occurred between the third floor and fourth floor. The formation of plastic hinges and anchorage failure in connections resulted in the rapid increase of lateral displacement. When roof drift reached 5%, the failure of columns occurred.

8.2. Punching Failure Mode

The remaining building was analysed with full dead plus live load. The shear ratio $V_g/\sqrt{f'_c}b_0d$ was 1.7. This building configuration and member properties remain the same as in the previous example except for the shear ratio (previous example $V_g/\sqrt{f'_c}b_0d=1.1$). Due to the increase of gravity load punching failure occurred in connections.

When the building roof drift reached 0.3%, all connections except 6 and 24 yielded as shown in Fig.8.10(a). The comparison of deformed shape between 30% and full live load buildings is shown in Fig.8.10(b). The displacement of full live load building was greater than that of 30% live load building.
FIGURE 8-8 Displacement Comparison With New Building

FIGURE 8-9 Relative Drift of Floors at Drift 3.5%
FIGURE 8-10(a) Plastic Hinge Positions (full live load)
FIGURE 8-10(b) Displacement Comparison
With 30% Live Load Building

FIGURE 8-11 Punching Failure Developing
When the building roof drift was only 0.48%, the first punching failure happened in connections 15 and 16, as shown in Fig. 8.11. The punching failure of connections resulted in complete loss of the bending and shear capacities of the connections. The gravity load resisted by this connection had to be transferred to adjacent connections. Connections 13 and 14 failed in punching immediately following the failure of connections 15 and 16. Due to no continuous bottom reinforcement passing through the column, the whole floor dropped and entire building collapsed.

Heavier gravity load greatly reduces the lateral resistance and the deformation capacity (46). The punching failure occurred before connections reached moment transfer capacity and bottom reinforcement anchorage failure. Since the earlier appearance of punching failure, the building roof drift only reached 0.5%, causing an obvious drop in lateral deformation capacity.
CHAPTER 9
CONCLUSION

9.1 Summary

An experimental investigation was carried out to study the seismic resistance of non-ductile slab-column connections. Four two-bay flat-plate subassemblies with the typical of gravity load design detail were tested under earthquake type loading. Based on the observed response of subassemblies and individual connections, conclusions were drawn on the strength, stiffness and deformation capacity of subassembly, interior and exterior connections. Conclusions on different aspects of the investigation follow:

9.2 Subassembly Responses

(1) Lateral load resistance of a subassembly depends on the failure mode of connections. In the case of a punching failure, the lateral resistance of a subassembly was limited mainly by the punching strength of interior connections. When flexural yielding in the slab dominates the response, the lateral resistance mostly came from the flexural strength of slab under negative bending at interior and exterior connections. The contribution of the slab under positive bending at connections was limited to its cracking strength and became negligible after the slab has reached its cracking capacity.

(2) The presence of spandrel beam increased initial lateral stiffness while an incease in the gravity load reduced the initial stiffness. When drift reached 2%, the 70% of the
initial stiffness was lost. Beyond the 2% drift, the stiffness of the subassemblies degraded in a similar manner and was independent of the specimen configuration.

(3) The intensity of the gravity load on the slab had the most effect on the lateral drift capacity of the subassembly. Under normal service load condition (DL+0.3LL), the subassemblies were able to reach a lateral drift of at least 4.5%. At full dead load and live load, the lateral drift capacity was reduced to 2%. The presence of an edge beam and bent-up reinforcing detail in the slab both appeared to improve the ductility of the slab-column subassemblies.

(4) Under normal service load conditions, the lateral load is resisted by a flexural yielding mechanism with yield lines developing across the full width of the slab at the interior connections and by a flexural-torsional mechanism of the slab edge at the exterior connections. Because of the low reinforcement ratio in the slab, the load-deformation response of the subassemblies is approximately bilinear.

(5) The lack of adequate anchorage or failure of anchorage of the slab bottom reinforcement in the connection region did not have any obvious effect on the overall lateral resistance of the subassemblies. However, in the event of a punching failure at the interior connection, the slab top reinforcement was not able to sustain the gravity load on the slab.

(6) The share of the lateral load carried by the interior column increased as the stiffness of the exterior connections
degraded. Under full dead and live load action, approximately 60% of the lateral load was carried by the interior column at 2% drift.

9.3 Interior Connections

(1) The mode of failure of non-ductile interior slab-column connections under seismic loading depends largely on the level of gravity shear applied to the connection. For a gravity shear ratio \( \frac{V_g}{\sqrt{f'_c b_s d}} \) of 1.0 or less, the mode of failure was primarily flexural. However, with a gravity shear ratio of 1.5, the mode of failure changed to that of punching shear.

(2) The punching shear capacity of the connection decreased with the increasing drift level and yielding of the slab reinforcement. Based on the eccentric shear stress model of ACI Building Code, Shear strength of the interior connection varied between \( 3.5 \sqrt{f'_c} \) at 2% drift to \( 3.0 \sqrt{f'_c} \) at 4.7% drift.

(3) The unbalanced moment transfer occurred only on the side of the connection with slab top reinforcement in tension. Moment transfer capacity on the side of the connection with slab under positive bending is limited to the cracking strength of the slab. With non-ductile reinforcing detail, the negative flexural strength of the column strip gave the best estimate of the unbalanced moment-transfer capacity of the interior connection.

(4) The connection stiffness degraded rapidly, losing 75 to 85% of the initial stiffness by 25% lateral drift. Provided the gravity load is small and the punching failure does not
occur, the connection is able to sustain at least 80% of the lateral load through at least 4% drift, though at a considerable loss of stiffness.

(5) A modification of the critical section used in the eccentric shear stress model is proposed. This model recognizes the observed limited capacity for shear and moment transfer between the column and the slab under positive bending after the slab has reached its cracking moment. The suggested three-sided section appears to better predict the shear strength of the interior connections.

9.4 Exterior Connections

(1) None of the exterior connections failed in punching shear. The limited torsional capacity of the slab edge acted more like a fuse and transferred the load to the interior connection, thus protecting the exterior connection from a punching failure. The exterior connections were able to sustain a gravity shear ratio $\frac{V_g}{\sqrt{f_{t_e}^* b_s d}}$ of 0.75 through 4-5% drift without a punching failure. This, however, did not represent the maximum shear capacity, as none of the connections reached their punching strength.

(2) The unbalanced moment-transfer under negative bending at exterior connection was limited to the flexural capacity of effective slab width. If a strong edge beam was present, the connection was able to develop flexural strength of the entire width of the slab. In the positive moment direction, the moment transfer was limited to the cracking strength of
the slab.

(3) The exterior connections lost their stiffness much faster than the interior connections losing approximately 70% to 75% of the initial stiffness in the negative moment direction by 1% drift. In the positive moment direction, the stiffness became negligible after the slab reached its cracking strength. Under repeated load reversals, the anchorage of slab bottom reinforcement was completely lost by about 2.25% lateral drift.

9.5 Analysis Model

(1) The anchorage failure of bottom rebars in the exterior connections made this column lose the resisting capacity in this loading direction.

(2) The proposed stiffness calculation conservatively indicates the variation of connection stiffness with the development of applied moment.

(3) The proposed model of moment-rotation, which indicates the typical deformation features of connections, is suitable for both exterior and interior connections.

9.6 The Analysis of Buildings

(1) If \( \frac{\nu_p}{\sqrt{f_c} \cdot b_o d} = 1 \) in interior connections, the plastic hinges in most connections were generated when building top drift was only 0.33%; and when building top drift reached 1.5%, all connections presented plastic hinges and most exterior connections had anchorage failure. The significant reduction of lateral stiffness and too big deformation caused by plastic
hinges and anchorage failures are the typical feature of flexural failure.

(2) The comparison between the old building and the new building indicated that when the old building top drift reached 1.5\% drift, the new building top drift only reached 0.83 percent, subjected to the same earthquake load.

(3) If \( \frac{\nu_g}{f_{lc} b_d} \) larger than 1.54 in interior connection, the higher gravity load caused the early appearance of plastic hinges. The punching failure of interior connections at 0.48\% of building top drift resulted in a whole floor collapse and, in turn, the building falling down.
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17. Cano, M. T., and Klingner, R. E., "Comparison of Analysis
Concrete Buildings with a View to Drift Control," 


APPENDIX A  SUBASSEMBLY DESIGN

This building shown in the Fig.A.1. is a five story building which is two-way solid flat-plates, three spans and columns without drop panels. The building is supposed to be an existing office building located in the New York city. Only the gravity load was considered. Design work mostly followed "Building Code Requirements for Reinforced Concrete (ACI 318-47) ".

THE DESIGN OF A FLAT-PLATE STRUCTURE WITHOUT DROP PANELS

MATERIAL

Concrete $f_c' = 3000$ psi, Reinforcement Grade 40 $f_y = 40,000$ psi

LOAD

Assuming a slab thickness of 9 inches and unit weight of reinforced concrete 150 lbs/ft$^3$,

| Self weight | 112.5 lb/ft$^2$ |
| Partition | 20.0 lb/ft$^2$ |
| 1" granolithic finish | 13.0 lb/ft$^2$ |
| Total dead load | 145.5 lb/ft$^2$ |
| The live load of office building | 50.0 lb/ft$^2$ |
| Dead load + Live load | 195.5 lb/ft$^2$ |

SLAB THICKNESS

1) The slab thickness $h > L/36$

Span length, $L = 20'$

$L/36 = 6.67''$ so $h = 9''$ O.K.

2) The shear strength

The critical section is defined as thickness of slab $h$ minus
1.5", outside edge of columns as shown in Fig.A.2. The shear stress caused by shear force in critical section is

\[ \nu_c = \frac{V}{bjd} \]

V: total shear force in critical section.
d: depth=h-1.5".
b: perimeter of critical section.
j: safe constant 0.88.
C_l: column size.

The shear stress should not exceed: (1) 0.03f_c' when at least 50 percent of bars for negative M_c pass directly over column capital. (2) 0.025f_c' when at least 25 percent of bars for negative M_c pass directly over column capital. If without a column capital, C_l+2h is taken as a column capital region. Here the 50 percent of the bars is assumed to pass the C_l+2h.

For interior connections

\[ V=20 \times 20 \times 195.5 / 1000 = 78.2 \text{ kips} \]
\[ b=4(20+2\times7.5)=140" \]
\[ d=9.0-1.5=7.5" \]

Shear stress 84.6 psi < 0.03f_c' = 90 psi O.K.

For exterior connections

\[ V=20 \times 10 \times 195.5 / 1000 = 39.1 \text{ kips} \]
\[ b=3 \times 20+4 \times 7.5=90" \]

Shear stress 65.82 psi < 0.03f_c' = 90 psi O.K.

For corner connections

\[ V=10 \times 10 \times 195.5 = 19.55 \text{ kip} \]
b=2x20+2x7.5=55''

Shear stress 53.85 psi < 0.03f_c' = 90 psi  O.K.

Finally, the thickness of slab h=9'' is selected.

**TO DETERMINE FLEXURE REINFORCEMENT**

The system of moment coefficients recommended by the code is used here due to satisfactory of limitation listed below:

(1) Slab must be rectangular and monolithic with columns.

(2) There should be three or more rows of panels in each direction.

(3) Maximum ratio of length to width of panel=1.33.

(4) Dimensions of adjacent panels should not vary by more than twenty percent of shorter span.

(5) Slabs may be solid or ribbed.

The total bending moment in the direction of either side of panels for a total uniform load w on the panel is

\[ M_o = 0.09wL \left[ 1 - \frac{2C}{3L} \right]^2 \]

\[ M_o = 0.09 \times 20 \times 20 \times 195.5 \times 20 (1 - 2x38/(3x240))^2 = 112.61 \text{ k-ft} \]

For interior panel
Column strip  
Negative moment=0.46M_o=51.8 k-ft  
Positive moment=0.22M_o=24.8 k-ft

Middle strip  
Negative moment=0.16M_o=18.0 k-ft  
Positive moment=0.16M_o=18.0 k-ft

For exterior panel

Column strip  
Exterior negative=0.41M_o=46.2 k-ft

Positive moment=0.28M_o=31.5 k-ft

Interior negative=0.5M_o=56.3 k-ft

Middle strip  
Exterior negative=0.1M_o=11.3 k-ft  
Positive moment=0.2M_o=22.5 k-ft

Interior negative=0.176M_o=19.8 k-ft

The bending reinforcement is determined by working stress method

\[ A_s = \frac{12M_o}{f_s \cdot J \cdot d} \]

\[ f_s = 20 \text{ ksi (allowable stress of steel)} \]
\[ d = 7.5" \]

Notice: if the slab simply rest upon a wall without any edge restraint, the bars in the strips perpendicular to the exterior wall must be changed for typical exterior panels. But, for this problem, the slab is monolithic with edge column, so moment value on exterior panel does not need to be changed. The \( A_s \) value and rebar type for each critical section listed below:
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S: Straight reinforcement.
B: Bent-up reinforcement.

Moment unit: k-ft and $A_s$ (area of rebar) unit: in$^2$.

The reinforcement arrangement is shown in Fig.A.3. The detail around the connections is shown in Fig.A.4.
FIGURE A-1  Prototype Building
Critical Section at an Interior Connection

Critical Section at an Exterior Connection

FIGURE A-2 Critical Section at Interior and Exterior Connections
FIGURE A-3  straight and bent-up reinforcing detail
## APPENDIX B  PROGRAM INPUT

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STOP
APPENDIX C  DRAIN-2D INPUT EXPLANATION

USER'S GUIDE TO INPUT DATA FOR MODIFIED R/C BEAM ELEMENTS

The input data defining the problem to be solved consists of the following parts:

A. PROBLEM INITIATION AND TITLE.
B. STRUCTURE GEOMETRY INFORMATION
C. LOAD INFORMATION
D. TIME HISTORY OUTPUT SPECIFICATION
E. ELEMENT SPECIFICATION

If a structure involves the modified R/C beam elements, the format of input data of part A to part B is the same as that of the original version [25], but the input element specification data for the group of modified R/C beam elements must follow the format described in the following.

E9(a) Control Information for Group (715) - One line.
See Note 1 for the rules on specifying this type of elements.

Column  5: Punch 9 (to indicate that group consists modified R/C beam elements.

6 - 10: Number of elements in group.

11 - 15: Number of different element stiffness types. See Section E9(b).

16 - 20: Number of different end eccentricity types. See Section E9(c).
21 - 25: Number of different yield surfaces for cross sections. See Section E9(d).

26 - 30: Number of different fixed end force patterns. See Section E9(e).

31 - 35: Number of different punching shear property types. See Section E9(f).

E9(b) Stiffness Types (I5, 3F10.0, 3F5.0, 2F10.0, 2F5) - One line

Column 5: Stiffness type number.

6 - 15: Section property EI.

16 - 25: Section property EA.

26 - 35: Section property GA.

36 - 40: Flexural stiffness factor Ki.

41 - 45: Flexural stiffness factor Kj.

46 - 50: Flexural stiffness factor Kij.

51 - 60: Strain hardening modulus for end i, as a portion of Young’s modulus.

61 - 70: Strain hardening modulus for end j, as a portion of Young’s modulus.

71 - 75: Unloading parameter Ai. See Note 2 for explanation.

76 - 80: Unloading parameter Aj. See Note 2 for explanation.

E9(c) End Eccentricity Types (I5, 4F10.0) - One line

Column 5: End eccentricity type number.

6 - 15: Horizontal eccentricity at end i.

16 - 25: Horizontal eccentricity at end j.

26 - 35: Vertical eccentricity at end i.
36 - 45: Vertical eccentricity at end j.

E9(d) Cross Section Yield Surface Types (I5, 5X, 2F10.0)
   - One line

Column 5: Yield surface number.

11 - 20: Positive yield moment, \( M_{y^+} \).
21 - 30: Negative yield moment, \( M_{y^-} \).

E9(e) Fixed End Force Pattern Types (2I5, 7F10.0) - One line. Omit if there are no fixed end forces.

Column 5: Pattern number.

6 - 10: Axis code, as follows.

   Code = 0: Forces are in the element coordinate system.

   Code = 1: Forces are in the global coordinate system.

11 - 20: Clamping force \( F_i \).
21 - 30: Clamping force \( V_i \).
31 - 40: Clamping force \( M_i \).
41 - 50: Clamping force \( F_j \).
51 - 60: Clamping force \( V_j \).
61 - 70: Clamping force \( M_j \).
71 - 80: Load reduction factor.

E9(f) Punching Shear Property Types (I5, 4F10.0) - One line.

Column 5: Punching shear property number.

6 - 15: Punching shear area \( A_c \) in Eq. 3-1, equal to \( 2d(c_1+c_2+2d) \).
16 - 25: Parameter \( J/c \), where \( c = (c_1 + d)/2 \). See Eq. 3-1. For edge connection, punch 1.

26 - 35: \( r_Y \), fraction of unbalanced moment transferred by eccentricity of shear at the interior connection. \( r_Y \) can be obtained from the following equation:

\[
r_{y} = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{c_1 + d}{c_2} d}}
\]

See Section 1-3 for explanation of notation \( c_1, c_2 \), and \( d \). For the edge connection, punch 1.

36 - 40: Shear strength.

E9(g) Element Generation Commands (12I5, 2F5.0, 2I4, 2I1)

- One line for each generation command. Element must be specified in increasing numerical order. Lines for the first and last elements must be included.

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<tr>
<td>16 - 20:</td>
<td>Node number increment for element generation. If zero or blank, assumed to be equal to 1.</td>
</tr>
<tr>
<td>21 - 25:</td>
<td>Stiffness type number.</td>
</tr>
<tr>
<td>26 - 30:</td>
<td>End eccentricity type number.</td>
</tr>
<tr>
<td>31 - 35:</td>
<td>Yield surface number for end i.</td>
</tr>
<tr>
<td>36 - 40:</td>
<td>Yield surface number for end j.</td>
</tr>
</tbody>
</table>
41 - 45: Code for including geometric stiffness. Punch 1 if it is to be included. Leave blank or punch zero if it is to be ignored.

46 - 50: Time history output code. If a time history of element results is not required for the elements covered by this command, punch zero or leave blank. If a time history printout, at the intervals specified on D1, is required, punch 1.

51 - 55: Fixed end force pattern number for static dead loads on element. Leave blank if there are no dead loads.

56 - 60: Fixed end force pattern number for static live loads on element. Leave blank if there are no live loads.

61 - 65: Scale factor to be applied to fixed end forces due to static dead loads. Leave blank if there is no dead load.

66 - 70: Scale factor to be applied to fixed end forces due to static live loads. Leave blank if there is no live load.

71 - 74: Punching shear property number for element end i.

75 - 78: Punching shear property number for element end j

79: Connection type code for element end i. If the connection at element end i is an interior connection, punch 1. If it is an edge connection, punch 2.

80: Connection type code for element end j. See explanation above.

NOTE 1 The following rules must be conformed when the modified R/C beam elements are specified.
(1) The element end must be at a plate-column connection, that is, one bay of plate cannot be divided into two or more elements.

(2) The modified R/C beam elements must be numbered from bay to bay first, then from level to level.
APPENDIX D    DRAIN-2D LISTING OF MODIFIED SUBROUTINE

<table>
<thead>
<tr>
<th>SUBROUTINE</th>
<th>MODIFICATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subroutine INEL9</td>
<td>Punching Shear Property</td>
</tr>
<tr>
<td>Subroutine STIF9</td>
<td>Connection Stiffness</td>
</tr>
<tr>
<td>Subroutine RESP9</td>
<td>Moment-Rotation Hysteresis Loop</td>
</tr>
<tr>
<td>Subroutine RESP9</td>
<td>Punching Check</td>
</tr>
</tbody>
</table>