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16. Abstract (Limit: 200 words)

This report summarizes current experimental research at Cornell University on lightly reinforced concrete structures. Lightly reinforced concrete framing systems, designed primarily for gravity induced loads, with little or no attention given to lateral load effects, are characterized by the following critical details: 1) longitudinal column reinforcement not exceeding 2% with lap splices located immediately above floor levels in the zone of maximum lateral load moment; 2) widely spaced column ties; 3) little or no transverse reinforcement within the joint region; and 4) discontinuous positive moment beam reinforcement with a 6-inch embedment length into the column. This report includes a summary of the full scale experiments conducted on the behavior of lightly reinforced concrete building frame components subjected to reversing cyclic loads (simulated seismic effects). Thirty-four full scale interior and exterior beam-column joints have been tested to date. This extensive experimental program identifies the different damage mechanisms and studies the effect of critical details. The results are intended to provide for the calibration of simplified and more elaborate analytical models to evaluate frame behavior, and for the planning of repair or retrofit.

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NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

State University of New York at Buffalo

Seismic Behavior of Reinforced Concrete Frame Structures with Nonductile Details:

Part I — Summary of Experimental Findings of Full Scale Beam-Column Joint Tests

by

A. Beres, R.N. White and P. Gergely

Department of Structural Engineering

School of Civil and Environmental Engineering

Cornell University

Ithaca, New York 14853-3501

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NOTICE

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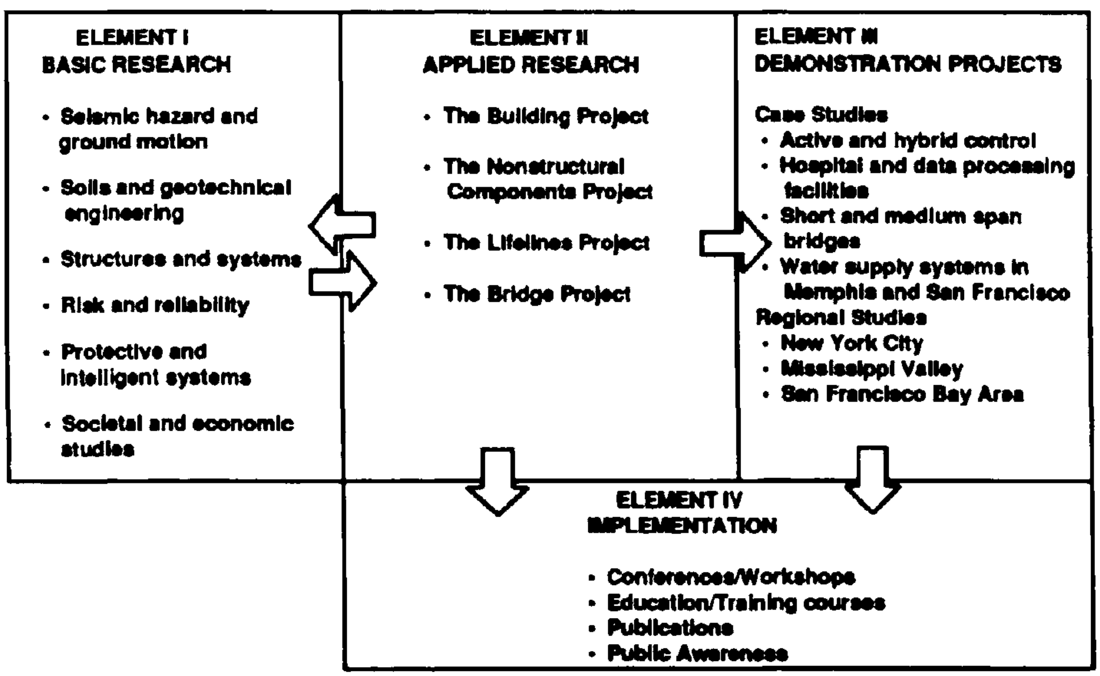
1 Graduate Research Assistant, School of Civil and Environmental Engineering, Cornell University
2 James A. Friend Family Professor of Engineering, School of Civil and Environmental Engineering, Cornell University
3 Professor of Structural Engineering, School of Civil and Environmental Engineering, Cornell University

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
State University of New York at Buffalo
Red Jacket Quadrangle, Buffalo, NY 14261

PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established to expand and disseminate knowledge about earthquakes, improve earthquake-resistant design, and implement seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures in the eastern and central United States and lifelines throughout the country that are found in zones of low, moderate, and high seismicity.

NCEER's research and implementation plan in years six through ten (1991-1996) comprises four interlocked elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten. Element III, Demonstration Projects, have been planned to support Applied Research projects, and will be either case studies or regional studies. Element IV, Implementation, will result from activity in the four Applied Research projects, and from Demonstration Projects.



Research in the **Building Project** focuses on the evaluation and retrofit of buildings in regions of moderate seismicity. Emphasis is on lightly reinforced concrete buildings, steel semi-rigid frames, and masonry walls or infills. The research involves small- and medium-scale shake table tests and full-scale component tests at several institutions. In a parallel effort, analytical models and computer programs are being developed to aid in the prediction of the response of these buildings to various types of ground motion.

Two of the short-term products of the **Building Project** will be a monograph on the evaluation of lightly reinforced concrete buildings and a state-of-the-art report on unreinforced masonry.

The **structures and systems program** constitutes one of the important areas of research in the **Building Project**. Current tasks include the following:

1. Continued testing of lightly reinforced concrete external joints.
2. Continued development of analytical tools, such as system identification, idealization, and computer programs.
3. Perform parametric studies of building response.
4. Retrofit of lightly reinforced concrete frames, flat plates and unreinforced masonry.
5. Enhancement of the IDARC (inelastic damage analysis of reinforced concrete) computer program.
6. Research infilled frames, including the development of an experimental program, development of analytical models and response simulation.
7. Investigate the torsional response of symmetrical buildings.

One of the major thrusts of the research at NCEER has been the evaluation of the performance of concrete frame structures that had been designed only for gravity loads. A variety of design details, common in most parts of the country, have been studied experimentally and analytically at several institutions. The main goal of these investigations has been the development of analytical tools for the prediction of the response of lightly reinforced concrete structures.

This is the first of a two-report series summarizing research on the seismic performance of reinforced concrete frame structures with nonductile details. This report describes the full-scale test series conducted on the behavior of interior and exterior beam-to-column joints. The experimental program covered a wide range of parameters, including different geometries and reinforcing configurations using thirty-four specimens. The results of these tests are intended to provide for the calibration of analytical models to evaluate frame behavior and to plan for repair and/or retrofit. The second report extends the test results to the behavior of building frames.

ABSTRACT

This report summarizes current experimental research at Cornell University on lightly reinforced concrete structures. Lightly reinforced concrete framing systems, designed primarily for gravity induced loads, with little or no attention given to lateral load effects, are characterized by the following critical details *(a)* longitudinal column reinforcement not exceeding 2% with lap splices located immediately above floor levels in the zone of maximum lateral load moment, *(b)* widely spaced column ties, *(c)* little or no transverse reinforcement within the joint region, and *(d)* discontinuous positive moment beam reinforcement with a 6-inch embedment length into the column.

This report includes a summary of the full scale experiments conducted on the behavior of lightly reinforced concrete building frame components subjected to reversing cyclic loads (simulated seismic effects). Thirty-four full scale interior and exterior beam-column joints have been tested to date. This extensive experimental program identifies the different damage mechanisms and studies the effect of critical details. The results are intended to provide for the calibration of simplified and more elaborate analytical models to evaluate frame behavior, and for the planning of repair or retrofit.

A companion NCEER report (Part II) that extends the test results to the behavior of building frames will be published in 1993. This report will also treat the evaluation of nonseismically detailed frames subjected to seismic loads.

ACKNOWLEDGMENTS

This research was sponsored by the National Center of Earthquake Engineering Research with funding from the National Science Foundation and the New York State Science and Technology Foundation.

Advice in different phases of the NCEER program was received from numerous practicing engineers. Advice on existing building details from Jacob Grossman of Robert Rossenwasser Associates of New York City, Raymond A. DiPasquale of DiPasquale and Associates of Ithaca, Glenn Bell of Simpson, Gumpertz, and Herger, Inc. of Boston, and Thomas Sabol of Engelkirk, and Sabol, Inc. of Los Angeles is greatly appreciated. The contribution of Stephen P. Pessiki, former graduate student at Cornell, is also acknowledged. His Ph.D. thesis set the stage for this study.

Special thanks are due to Tim Bond and David Farmer of the George Winter Laboratory Technical Services, and to Paul Jones of Cornell Civil Engineering Machine Shop. Thanks are also extended to the many undergraduate laboratory assistants who helped at various phases of the experimental work.

A note of thanks is given to the faculty members and the graduate students of the State University of New York at Buffalo who are collaborators in the NCEER Lightly Reinforced Concrete Structures Research Group. Their contributions in computer simulation and experimental evaluation of reinforced concrete frames provided substantial support for the efforts at Cornell.

Opinions, findings, and conclusions expressed in this study are those of the authors only and do not necessarily reflect the views of the sponsors.

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SECTION 1

INTRODUCTION

1.1 Motivation

There are many thousands of multistory reinforced concrete frame structures in the United States that were designed without regard to any significant lateral forces. The lack of seismic considerations resulted in non-ductile reinforcing details that were in sharp contrast to those used in modern seismic design. Also, the amount of reinforcement was often below the minimum values specified in current building codes. Therefore, the lateral load resistance of these existing structures is considered suspect for even moderate earthquakes. The research reported here was primarily motivated by the fact that lessons learned about frame behavior in regions of high seismicity were judged to be inadequate to fully support seismic evaluations of reinforced concrete frames in other regions.

During this century, many buildings predominantly designed for gravity loads have been built in zones of low and moderate seismicity. This has been common not only in the US but in developing countries as well. At the same time older structures having similar details can be found in regions of high seismicity (such as California); these structures were built prior to onset of modern seismic design. ACI Building Codes prior to the 1971 edition did not contain seismic provisions. Evaluation, and subsequent enhancement of these structures might be warranted for various reasons, such as change in the building occupancy, moderate damage occurring in previous earthquakes, or mandatory upgrade (e.g., for essential and critical facilities, such as emergency and communication centers, schools, and hospitals).

To develop reliable seismic evaluation techniques for this broad class of frames, a comprehensive research program has been underway at Cornell University under the

- Incorporate the generalized experimental data into complex and simplified computational tools to predict building response during earthquakes.
- Evaluate the seismic behavior of existing buildings and establish performance criteria to determine whether these (LRC) structures need to be retrofitted.

The findings and recommendations of the combined reports are intended to serve as guidelines for practicing engineers as well as to provide information for building code developers, especially in zones of low and moderate seismic intensity.

1.3 Scope and Organization of the Report

The scope of this investigation was limited to frames without infill-walls in buildings of regular geometry. A brief background review is provided in Section 2 on the identification of potentially critical design details and the relevant code requirements.

The extensive experimental program, discussed in Sections 3, 4, and 5, is the continuation of the test series reported in 1990 by Pessiki, Conley, Gergely, and White. Because of space limitations, the detailed test results from the new experiments are not included here but are documented in a supplementary report [Beres, White, and Gergely, 1992]. The repair and retrofit phases of the experimental program were reported in detail separately [Beres, El-Borgi, White, and Gergely, 1992]. Results of the small-scale model building studies were published by El-Attar, White, and Gergely, [1991].

SECTION 2

BACKGROUND

2.1 General Characteristics of Existing Reinforced Concrete Frames Designed Primarily for Gravity Loads

Presently, building codes in many regions of moderate and low seismicity require only limited adherence to provisions for seismic resistance. The overwhelming majority of existing structures built in these zones have been designed without any compliance to these requirements. In addition, some provisions are felt to inadequately address key questions crucial to the safe transmission of the seismically induced inertia forces to the ground. The inherent shortcomings of gravity load design philosophy imply high susceptibility of the frame structures, examined in this report, to anticipated seismic risk.

Details found in existing frame structures violate the principles of modern seismic design practice, such as:

- Avoid brittle type failures, e.g., concrete crushing, column rebar buckling, joint shear failures.
- Use ductile details that provide for large inelastic deformations, limit the maximum deformations (interstory drift) to safeguard the secondary elements and reduce P-Δ effects.
- Avoid the formation of a collapse mechanism. Assure proper failure hierarchy (strong column - weak beam).
- Limit the damage, such that the gravity loads will be safely transmittable to the ground.

Lack of satisfaction of the above criteria is expected to impose high potential for building damage and might result in catastrophic consequences.

5. Discontinuous positive beam reinforcement with a short embedment length into the column.
6. Construction joints below and above the beam-column joint.
7. Columns having bending moment capacity close to those of the beams.

2.3 Relevant Code Requirements

This section is a brief summary of the ACI-318 Building Code (1956-1989) provisions pertinent to the examined reinforced concrete frame details. Since the 1956 version of the Code very little modification has been implemented to enhance reinforcing details applicable to this project; only the 1971 Code included a large number of changes tightening some provisions and relaxing others. Otherwise, the relevant specifications were fairly consistent until 1989 when certain requirements became more restrictive.

2.3.1 Longitudinal Reinforcement of the Column with Lapped Splices

The minimum 1% and the maximum 8% longitudinal steel ratio provisions have been consistently present in all reviewed editions of the ACI Code. Some local building codes had relaxed the minimum requirement to 0.5%, probably to permit use of fewer bars in the upper levels where the gravity type of loading does not impose high demand on the columns.

The longitudinal bars in the columns of multistory buildings are most often lap spiced. Although it is generally recommended not to locate splices at points of maximum stress it was common practice to place the splice region just above the joint. Minor modifications were made in each edition of the ACI Code concerning the minimum required lapped length. The minimum length specified ranges from $20d_b$ to $30d_b$ depending on the yield strength of the steel and the bond strength. Starting in 1971, a 0.83 reduction factor can be applied for lap-spliced bars in columns confined by minimum spacing requirements that is usually the case.

2.3.2 Confinement of the Concrete

The discussion about concrete confinement in the ACI Code is limited to the seismic design provisions. It is assumed that for gravity action, the utilization of highly confined compression zones is not necessary, since only small plastic hinge rotations are expected near the supports.

Maximum tie spacing allowed is the least of the following: $16d_{b,main\ bar}$, $48d_{b,tie}$, the smallest column dimension. ACI Code (1956-1989) provisions also require that transverse reinforcement spacing be no more than half the effective depth of the member. However, this is not mandatory when the factored shear is less than 50% of the shear capacity of the concrete, as often happens in columns where the shear forces (including those from wind effects) are small. There was no provision to place ties at the end of the splice, but the first ties above the floor level were placed at a maximum distance of half the regular tie spacing since the '71 Code.

Until 1963, the ACI Code required support of the intermediate main reinforcement by 90 degree corners of a tie; this would likely increase the confinement. Since 1963 it is only specified that "ties shall provide lateral support at each corner and longitudinal bars shall be no more than six inches from lateral supports." Note that even #2 ties were permitted before 1971 when the minimum tie diameter was increased to #3 for up to #10 main bars and #4 for larger or bundled longitudinal bars.

2.3.3 Transverse Reinforcement in the Joint

When designing frames by the ACI Code, the connection regions are assumed to be rigid and the detailing is presumed adequate to transfer moments, axial and shear forces through the joint. There are no appropriate provisions to address the joint shear strength. The seismic design section of the recent ACI Codes does contain a provision for using a minimum area of shear reinforcement within the joint panel, but this provision may be disregarded if lateral action is of no concern.

The ACI Code provision for the continuation of the column transverse reinforcement through the joint addresses minimum confinement issues rather than the joint shear capacity. From 1971, ties within the joint panel are required, with the exception of beams framing into the column from all sides of the joint, or if analysis or experiment shows that strength reserve is adequate, which is frequently the case for buildings designed primarily for gravity loads.

The seismic design provisions of the current ACI Code requires taking the joint shear capacity as 12-20 $\sqrt{f_c'}\, b_j h$ (where $b_j h$ is the effective joint cross section area), depending on the confinement provided by the beams. These values are independent of the axial load and the transverse reinforcement.

2.3.4 Discontinuous Beam Reinforcement with Short Embedment Length

Moment reversals induced by large lateral load impose the risk of pullout of the embedded, discontinuous positive flexural beam reinforcement. The resulting hinging action of the beams may cause large deformations and loss of load capacity. Anchorage requirements were quite lenient in the previous ACI codes. Until 1971, only a quarter of the positive reinforcement had to be extended into the supports and a minimum six inch embedment was specified. The 1971 and the following versions of ACI-318 included a clause that required adequate embedment length to develop the nominal yield strength of the discontinuous bars, when the beam in question is part of a lateral load resisting system. However, this provision could be completely ignored if lateral loads were negligible and gravity loads governed.

Only the latest, (1989), edition has provisions for continuity. Based on ACI-318 (Ch. 12.2.2), the required minimum development length for flexural reinforcement is the larger of $0.04A_b f_y/\sqrt{f_c'}$ or $0.0004 d_b f_y$.

2.3.5 Construction Joints Below and Above the Beam-Column Joint

The code provisions concerning construction joints concentrate on surface preparation requirements including laitence removal, cleaning, wetting and application of cement grout before placing new concrete, without specifying joint location.

2.3.6 Relative Flexural Strength of Beams and Columns

The relative strength of beams and columns have a fundamental influence on the damage hierarchy. Since 1979 the ACI Code has provisions with the intent to enforce a beam sidesway collapse mechanism for buildings designed in high seismicity regions. The flexural moment capacity of the columns has to exceed by at least 20% that of the beams, where 1.25 overstrength factor (multiplier) should be taken into account to calculate the bending moment capacity of the beams.

2.4 Review of Previous Related Studies

Technical literature on the behavior of reinforced concrete frames became very rich during the past three decades. Most of the work carried out over this period has focused on the improvement of design procedures. Studies on existing non-seismically designed buildings before the NCEER initiative were scarce and mostly confined to post-earthquake reconnaissance and rehabilitation studies. In this subsection, only the main thrust of the seismic research on the behavior of reinforced concrete frames is summarized. In the following sections, each major topic is accompanied by a review of state of the art knowledge.

The first extensive monograph on the seismic design of reinforced concrete structures was published by Blume, Newmark, and Corning, in 1961. In the early sixties "groundbreaking" studies on reinforced concrete beam-to-column connection regions were carried out at the

PCA labs by Hanson, and Conner [1967]. These experiments showed the fundamental benefits of proper seismic detailing.

The first seismic design recommendations were based on these and other studies [e.g., Wight, and Sozen, 1973] conducted until the mid seventies. These guidelines applied similar principles to those in the design of flexural members, e.g., the concrete and the transverse reinforcement in the joint were assumed to act collectively to resist the shear forces within the joint, and the amount of transverse reinforcement was to be provided to resist shear stresses beyond the shear capacity of the concrete in the joint. These principles were reflected in the first edition of the ACI-ASCE 352 Committee, Joints and Connections in Monolithic Concrete Structures, Recommendations in 1976. The Recommendations were based on test series conducted on isolated beam-column joints without transverse beams or slabs.

In buildings constructed according to these guidelines, it was found that joints are often congested and difficult to construct. Additional experimental results became available by extensive testing programs conducted in the US during the seventies and eighties mostly at U. Texas at Austin, U. Michigan at Ann Arbor, U. California at Berkeley, U. of Illinois at Urbana, and at research institutions of New Zealand, Canada, and Japan. These studies addressed a wide range of parameters and as a result, a conceptual change concerning joint shear strength capacity, and addressing confinement effects, was implemented in the 1985 edition of the 352 Committee report. Additional refinements were included in the latest edition, published in 1991.

Since the first comprehensive and widely accepted explanations on the behavioral aspects of the reinforced concrete beam-to-column joints by Park, and Paulay in 1975, and Paulay, Park, and Priestley [1978], many complex analytical models were born elaborating on important details, such as, the effect of joint shear reinforcement and concrete compression struts on the joint shear capacity. Attention of researchers shifted to unexplored areas of slab contribution, eccentric beams, etc. Small-scale models of multistory, multibay building structures were tested on shake-tables.

Computerized nonlinear dynamic analysis techniques were enhanced since the first computer program DRAIN-2D, which found wide acceptance in the research community, was developed by Kanaan, and Powell in 1973. Besides the several offsprings of the original code, new software packages like IDARC by Park, Reinhorn, Kunnath [1987] were developed. Though these programs are now capable to address 3-D problems, flexible floors, etc., they haven't found their way yet to the consulting offices, partly because of lack of guidelines for the modeling of complex structural configurations, and partly because of the unfriendly user-interface.

The first experimental studies directly targeting beam-column components were conducted at Cornell University and at the State University of New York at Buffalo. Full-scale components were tested by Pessiki et al. [1990] and reduced-scale specimens by Winters, Hoffmann, Symans, and Wood [1991], Aycardi, Mander, and Reinhorn [1992], Choudhuri, Mander, and Reinhorn [1992]. A 1/6 scale two-story one bay and a 1/8 scale three-story three-bay building model was tested on shake table by El-Attar et al. [1992a,b]. Similar to the latter one, a 1/3 scale three-story three-bay structure was tested without and with retrofit by Bracci, Reinhorn, and Mander [1992a,b,c]. Repair and retrofit experiments were also conducted at full-scale by Beres et al. [1992].

The other major area of the NCEER research efforts is the enhancement of existing nonlinear time history analysis software. Refined versions of IDARC were developed by Kunnath, Reinhorn, and Lobo [1992], Lobo, Reinhorn, and Kunnath [1992], and El-Borgi, White, and Gergely [1991]. Similar efforts on DRAIN-2D led to SARCF-II by Rodriguez-Gomez, Chung, and Meyer [1991] and DRAIN-2DX by Allahabadi [1987], and Powell, and Prakash [1992]. These analytical tools were used by several investigators for the analyses of lightly reinforced concrete frames, e.g., Shahrooz and Muvdi [1991], Hoffmann, Kunnath, Mander, and Reinhorn [1992]. Because of the lack of user-friendly pre- and post-processors (currently under development for both IDARC and DRAIN-2DX) the task of ensuring error-free, easy input and the rapid visualization of arbitrary result quantities is extremely important in the context of reinforced concrete structures, where the complex input and output data is voluminous. Elaborate dynamic inelastic analyses, incorporating P-Δ effects, are still considered by most structural designers too time-consuming and cumbersome. Therefore, simplified evaluation methods are also considered in Part II.

SECTION 3

EXPERIMENTAL PROGRAM

Similar specimen configuration and testing methodology were used for the entire full-scale component testing program at Cornell involving experiments on bare, repaired, and retrofitted beam–column joints. This section contains the relevant details of the tests on subassemblies without any strengthening. A brief overview of the testing plan is provided in Section 3.1, followed by short summaries of the specimen geometry, configuration and fabrication details in Section 3.2. The loading arrangement and the measurement and test control systems are discussed in Sections 3.3 and 3.4. Finally, the test parameters are described in Section 3.5.

3.1 Overview of the Testing Program

Thirty–four virgin beam-column subassembly specimens have been tested at Cornell to date. Results from the tests of the first ten specimens were published previously by Pessiki et al. [1990]. Details of the remaining tests are reported by Beres et al. [1992] in a companion report that complements the present work.

Typical details of interior and exterior joints are shown in figures 3-1 and 3-2 respectively. The variables examined in this program are as follows:

(a) Six interior joint specimens had continuous positive beam reinforcement through the beam–column joint panel. These specimens were detailed to investigate the influence of the amount of joint reinforcement and column bar arrangement on the behavior of joints with spliced and unspliced vertical column rebars.

(b) Fourteen interior joint specimens had discontinuous positive beam reinforcement extending 6 inches into the columns. Variables studied included the size of embedded

3.2 Specimen Geometry, Materials and Fabrication Details

The most important specimen characteristics were:

- 14" x 24" beams with 2–#6 or 2–#8 (continuous or discontinuous) positive moment bars and with #3 stirrups at 5" spacing.
- 16" x 16" columns with 1% or 2% reinforcement and #3 ties at 14" and 16" spacing respectively (with the first tie placed 7" and 8" above the joints as specified in past ACI Codes); extra #3 ties at the lower bending point of the offset vertical reinforcement. With the exception of four specimens no ties were placed within the joint panel zone.
- 1.5" concrete cover over ties and stirrups.
- Nominal material strengths were $f_c' = 3500$ psi and $f_y = 60$ ksi.
- Some specimens had post–tensioned transverse beam stubs to simulate the presence of lateral confinement from transverse beams, framing in from out of plane.

Emphasis was placed on reproducing actual construction procedures. Specimens were cast in a vertical position using internal vibrators to simulate bleeding and settlement of the fresh concrete mix. Construction joints were created just under and above the beam by the three separate pouring lifts. Surface preparation methods and curing procedures matched those of typical field practice.

Concrete was provided by a local ready-mix supplier. Compressive strengths of cylinders, made at each cast, were obtained after curing in both "field" and "standard" environments. The strength values showed fairly large variation over the course of this study as detailed in Section 3.5. Tolerance deficiencies experienced (when reinforcing steel bending was made by local manufacturers) added to the list of parameters to be examined.

3.3 Loading Arrangement

The following discussion pertains directly to interior joint specimens (a similar procedure was used for exterior ones).

To simulate seismic action, the cruciform shaped specimens were loaded in a computer-controlled testing facility constructed at Cornell. Figure 3-3 shows two elevation views of the custom built testing frame, while figure 3-4 shows an idealization of the force and reaction systems. The use of full-scale tests was necessitated by the uncertainties inherent in reduced scale modeling of complex details, such as the splice and the embedment region. Detailed information about the experimental setup is provided in the report of Pessiki, Conley, Bond, Gergely and White [1988]. The system described there was used during the course of the testing program with several enhancements involving the measurement devices, the data acquisition system and the control software.

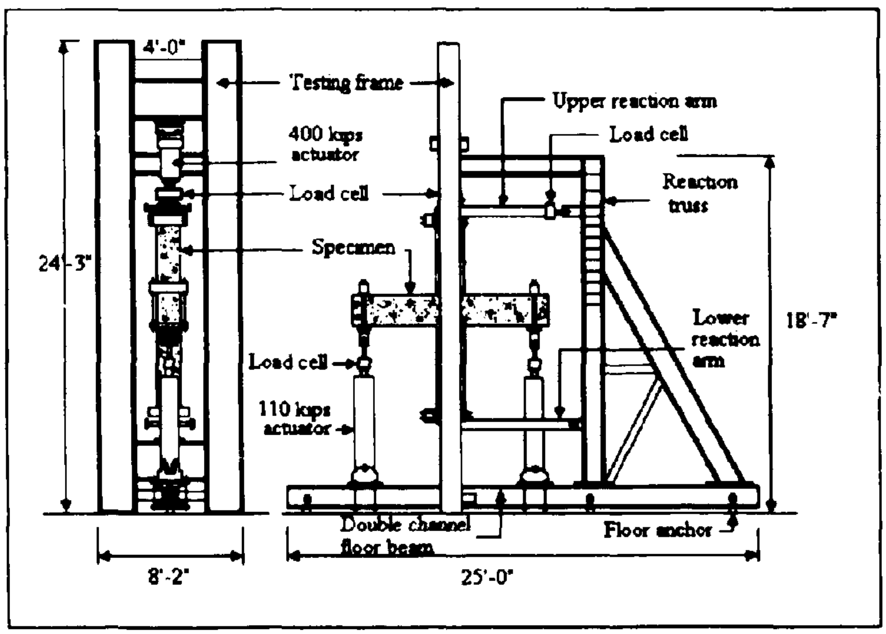


FIGURE 3-3 Two Elevation Views of the Testing Frame (after ref. Pessiki et al. 1988.)

The specimen configuration and the loading arrangements simulated forces and deformations of the joint component representing simultaneously acting gravity and lateral loads in a real structure. Each end of the column members of the isolated substructure was held in place with stiff horizontal reaction arms. Forces exerted by the vertical actuators attached to the ends of the beam members produced cantilever bending of the beams and antisymmetrical double curvature in the column.

Although the seismic loading was assumed to act in the plane of the frame components, lateral confining effects induced by the gravity loaded transverse beams were also incorporated. This was achieved by casting short transverse beam stubs on the sides of the joints. The joint confinement provided by the compressed part of the beam cross section was simulated by applying an average 450 psi compressive stress over a 14 inch wide and 8 inch high area with a manually controlled hydraulic prestressing mechanism.

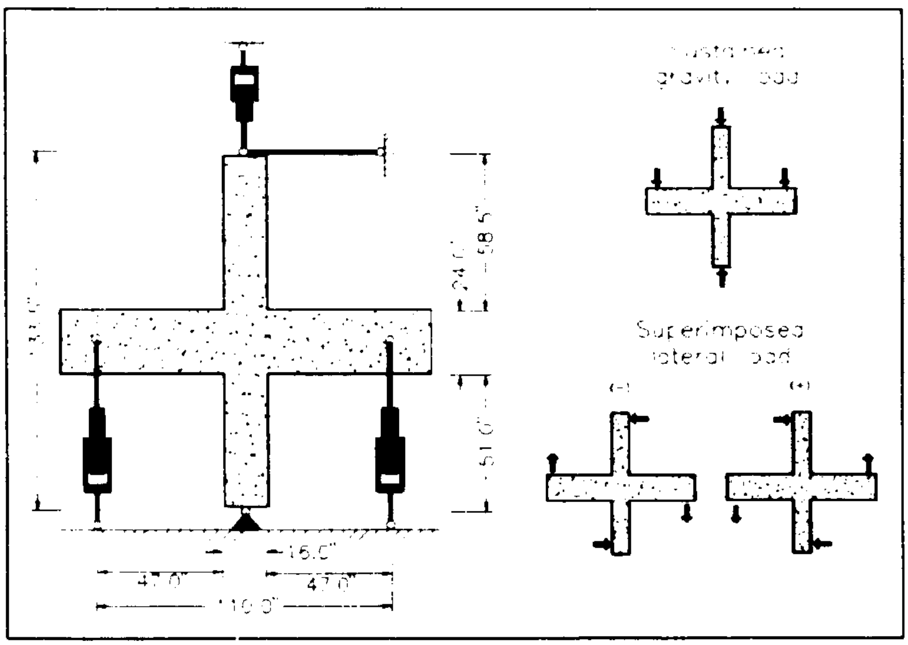


FIGURE 3-4 Idealization of the Force and Reaction System

The slowly applied reversed cyclic load was controlled by the values of the shear forces acting on the beams, with the "reference" value of 20 kips (25 kips for interior joints with continuous positive reinforcement) representing constant dead and service loads on each beam. The preset load-history, demonstrated in figure 3-5, consisted of sets of three cycles applied to the beam ends at paired force levels of 30 and 10 kips, 40 and 0 kips, 50 and -10 kips (negative denotes upward force), and 60 and -20 kips. Low-level cycles (30 and 10 kips) were applied after each third cycle. Loading beyond peak resistance was displacement-controlled by the gradually increasing values of positive beam rotation measured over a distance of 11 inches from the joint. The algebraic sum of the beam forces and the compressive axial force on the top of the column were kept constant throughout the test. During the first six tests (interior joints with continuous positive beam reinforcement) a different loading pattern was applied. The entire load history was directed in displacement control. This control was based on estimated yield rotation values and ductility factors.

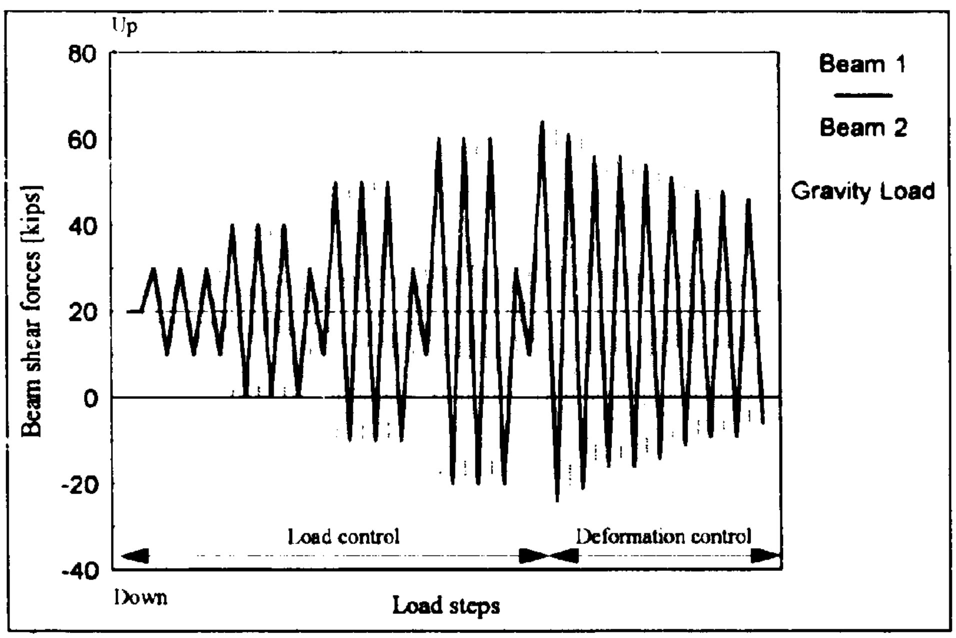


FIGURE 3-5 Typical Load History

3.4 Measurement and Control Systems

A computer program was written to semi-automatically control the load application and data acquisition tasks during a test. This interactive software allowed the operator to have full control over the applied load or displacement history. Multiple levels of operator intervention was provided for altering the speed of execution, displayed graphical output, and numerical information.

Each of the three (two at exterior joints) independent hydraulic servo–controlled actuators was directed by the control program via MTS Controller System that monitored the individual closed loops in terms of displacement. Forces were measured with load cells at the three actuators and at the top reaction arm. Force and displacement values were displayed at each load increment to provide interaction possibilities for the operator.

Despite several attempts to lubricate the machined bearing surfaces connecting the top column actuator to the specimen, the force measurements at the reaction arm were judged not to be sufficiently accurate to represent the column shear force because of the friction exhibited at the hinges. This was quite apparent at low shear force levels. Therefore, the measured force values reported here were replaced by values calculated based on equilibrium.

Member rotations were computed from measurements made with linear displacement transducers. These transducers measured relative displacements of points of member cross sections adjacent to the joint over a distance of 11" in the beam(s) and 13.5" in the columns. Interstory drift was calculated as the total column height multiplied by the amount of rotation the entire specimen must undergo to restore the displaced positions of the end(s) of the beam(s) corresponding to gravity load alone. Some specimens had additional instrumentation such as strain gages or inclinometers.

3.5 Summary of the Individual Test Parameters

A wide range of variables were studied, including reinforcing steel arrangement, gravity load level, concrete confinement, and strength. Parameters of the tested interior and exterior connections are tabulated specimen-by-specimen in Appendix A.

All specimens had similar dimensions with the exception of one interior joint where the upper column length was increased by 8 inches and the lower column length was decreased by the same 8 inches. Typical dimensions of exterior and interior specimens were described earlier in Section 3.2 and shown with reinforcing details in figures 3-1 and 3-2. At the origination of the testing program [Pessiki et al, 1990] the upper columns were chosen to be 8.5 inch longer than the lower columns to impose higher bending moment at the splice region.

The longitudinal reinforcement ratio in the column was in the range of 1 to 2% with varying numbers of bars, such as, 4-#7 (1%), 4-#10 (2%), 6-#8 (2%), and 8-#7 (2%). All specimens (with one exception, where the longitudinal bars were continuous) were made with lap splices of $30d_b$ (38 inches for #10, 30 inches for #8, and 27 inches for #7). Because of the splice, the column bars were offset in the plane of the applied load.

In the beams, the negative moment longitudinal reinforcement, either 4-#9 (4.0%), or 2-#8 and 2-#6 (2.5%), was always continuous through the interior connections. For exterior joints, bent-down negative moment reinforcement was used with 6" bending radius and 12" extension as shown in figure 3-2. The positive rebars were either continuous [2-#9] or discontinuous [2-#6 (0.9%) or 2-#8 (1.6%)] with 6" embedment in the joint panel zone.

As transverse reinforcement #3 bars were used with 5 inch spacing in the beams and 14 or 16 inch spacing in the column. The first column tie was placed at half the regular spacing above the top of the beams. All ties and stirrups were closed loops with 90 degree bends. The concrete cover to the transverse reinforcement was 1.5 inches in the beams and columns.

Material strength values exhibited large variations. The measured yield strength of the generally used Grade 60 reinforcing steel ranged from 66.9 to 83.2 ksi. Two specimens were cast with Grade 40 steel, but according to calculations, the 49.5 ksi actual yield strength was not reached during the tests. Therefore, independent of their actual yield strength, all rebars were assumed to be behaving linearly elastically. In some specimens made with Grade 60 bars, with beam stubs and high axial load, the steel stress in the embedded bars was estimated to be up to 15% higher than the yield stress for Grade 40 bars.

The compressive strength of concrete cylinders, kept near the specimens to expose them to similar environment (ambient temperature and humidity), showed even greater scatter. While most of the compressive strength results fell in the 3000 to 4000 psi range, a few concrete batches exhibited large deviation from the targeted 3500 psi, ranging from 2140 to 5720 psi. This made it necessary to normalize the strength results by using a multiplier $\sqrt{3500/f_c^{'}}$, where $f_c^{'}$ is in psi and 3500 psi is the nominal design strength.

Sustained load values representing gravity loads (dead and occupancy loads) were applied at the beginning of each test. Axial force on the columns was either 100 or 350 kips, which translates to about $0.11A_cf_c^{'}$ and $0.39A_cf_c^{'}$ respectively. The initial shear forces on the beams were 20 kips for specimens with discontinuous positive reinforcement and 25 kips for those with continuous reinforcement.

SECTION 4

OBSERVED DAMAGE PATTERNS OF BEAM-COLUMN JOINTS

This section summarizes the experimental observations in detail. The thirty-four specimens are divided into three categories. First, interior joint specimens are discussed in two separate groups based on the continuity of the positive moment beam reinforcement. This is followed by the behavior description of the exterior joints. In the latter class, only specimens with "discontinuous" positive moment beam reinforcement were tested. Examples of characteristic cracking patterns and hysteresis plots are provided and damage development patterns are discussed.

4.1 Interior Joints with Continuous Reinforcement

This subsection is the recapitulation of the relevant findings published in an earlier NCEER report by Pessiki et al. [1990]. The interior connection specimens with continuous reinforcement were detailed to study the capacity of the splice zone and the joint panel. The load history applied to these six specimens was slightly different from that for other specimens. The cyclic loading was controlled throughout the test according to gradually increasing rotations measured at the upper column adjacent to the joint panel. The column axial force level was constant at 350 kips in all specimens.

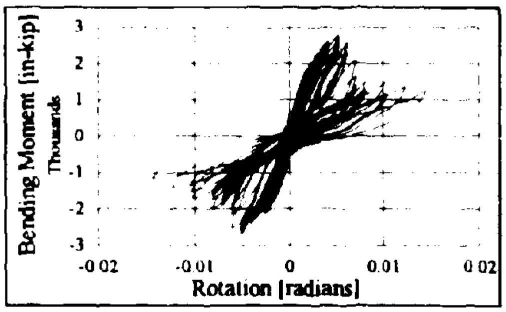
In specimens without ties in the joint, damage was confined to the joint panel and to the upper column in the zone below the first column tie above the joint. Most of the energy dissipation and stiffness loss that occurred in the columns was also attributed to the deterioration of the latter zone very close to the joint, and the joint panel itself, as demonstrated with two hysteresis plots shown in figures 4-1(a,b). All specimens had extensive shear cracking in the joints at failure (figure 4-1(c)). In specimens that had no ties within the joint, the loss of strength is attributed to the low shear capacity provided by the concrete. This was also manifested during the final cycles by the large deformations of the beams caused by the loosened embedment of the continuous longitudinal reinforcement. Unfortunately, no numerical data is available about the deformation of the beams, and the relative contribution of beam rotations to the total deformation of the specimens (interstory drift) cannot be quantified for this group of specimens.

The increased number of longitudinal reinforcing bars (8-#7) in the columns resulted in a more concentrated diagonal crack pattern in the joint panel compared to the distributed cracking exhibited at specimens with 4-#10 bars. The only specimen that had no splice showed no damage in the column other than minor flexural cracking and crushing of concrete at large deformations.

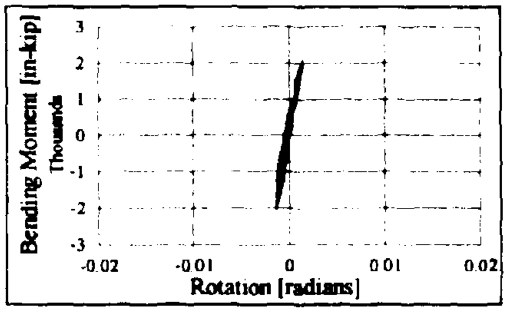
Load capacities belonging to different damage modes were very close. Providing 2-#3 ties in the connection region distributed the cracks within the joint panel, shifted the failure zone to the splice region, and decreased the rate of strength loss. It did not increase the peak resistance significantly because of the weakness of the lightly confined splice zone.

In columns made with eight No. 7 bars and ties within the joint, loss of cover over the splices contributed to the eventual failure initiated by bowing of the offset splice reinforcement. Buckling of the lightly confined column longitudinal bars resulted in the sudden loss of load bearing capacity of the columns.

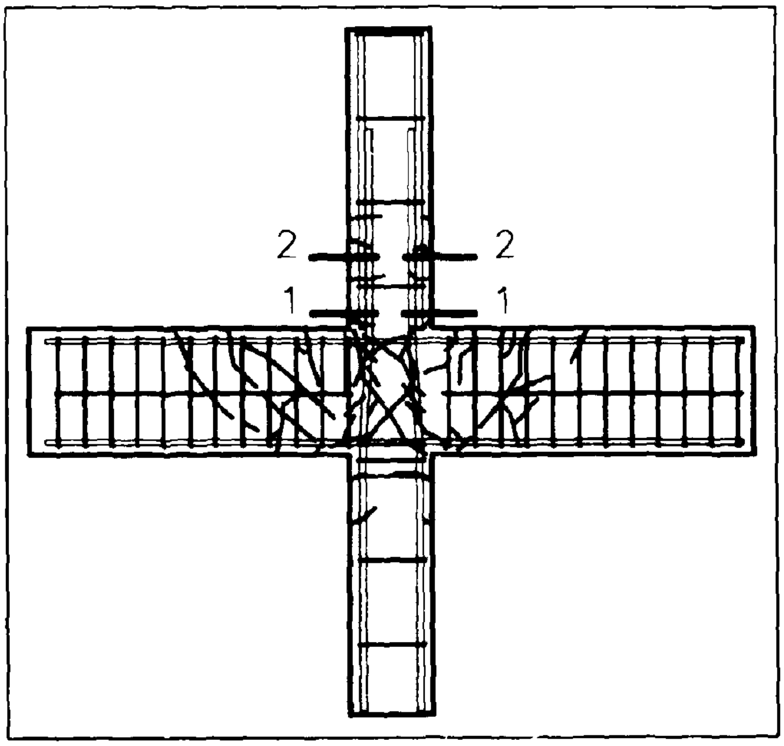
The dominating damage modes were either excessive shear cracking in the joint panel or buckling failure at the splice zone. The joint shear stresses at peak load (computed based on the guidelines of ACI-ASCE 352R) were between 11.8 and 13.6 $\sqrt{f_c'}$, with negligible influence of column bar size and arrangement, as opposed to the maximum allowed 15 $\sqrt{f_c'}$ for seismically designed joints (where f_c' is the compressive strength of the concrete at the joint–panel zone in psi, and this type of joint in classified as type 2, exterior joint by ACI–ASCE 352R) as discussed later in Section 4.3.1.



a. Moment–Rotation at Zone 1.



b. Moment–Rotation at Zone 2.



c. Cracking Pattern

FIGURE 4-1 Typical Interior Joint with Continuous Reinforcement (Specimen I-02)

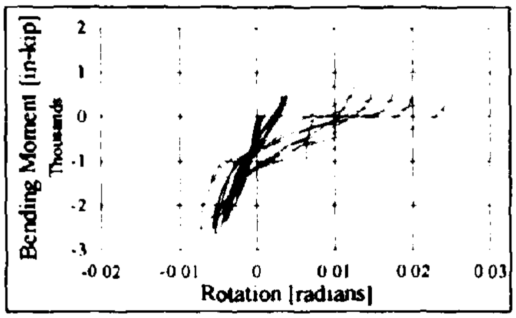
4.2 Interior Joints with Discontinuous Reinforcement

Fourteen specimens were constructed with discontinuous bottom beam reinforcement embedded 6 inches into the column. Figures 4-2(a,b) show plots of bending moment versus rotation measured close to the joint of a typical specimen. The individual hysteresis loops are markedly different from those for more thoroughly reinforced joints for several reasons. The hysteresis loops are not symmetrical since *(a)* the beam reinforcement was not symmetrical, *(b)* the reversing load cycles produced the superposition of the symmetrical gravity loads and the antisymmetrical loads simulating the lateral action, and *(c)* the bottom beam reinforcement tended to pull out at increased positive bending moment levels.

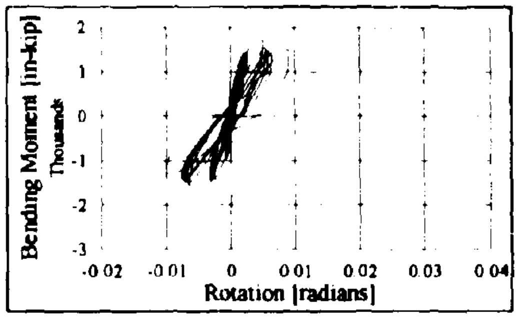
Failure of the typical specimen was initiated by pullout of the discontinuous beam reinforcement from the beam-column joint. Crack development and loading history is summarized in table 4-1. At early stages of the load history, cracks appeared on the face of the joint near the embedded bars. These cracks progressed as the test continued, eventually merging with diagonal cracks formed also at lower load levels at the top corners of the joint panel due to the downward forces on the beams. The final crack pattern is shown in figure 4-2(c). The lack of joint shear strength capacity was aggravated by the additional distress in the vicinity of the short embedment length of the bottom beam bars. This resulted in gradual diagonal crack opening and further loss in strength and stiffness during the subsequent cycles.

In a few cases, the dominant cracking pattern was different. Cracks propagating from the positive reinforcement vertically along the beam-joint interface caused most of the total specimen deformation. Frequently, the opposite joint faces showed an unsymmetrical combination of cracking, with diagonal cracks dominating at one side and large vertical cracks opening along the beam joint interface at the other.

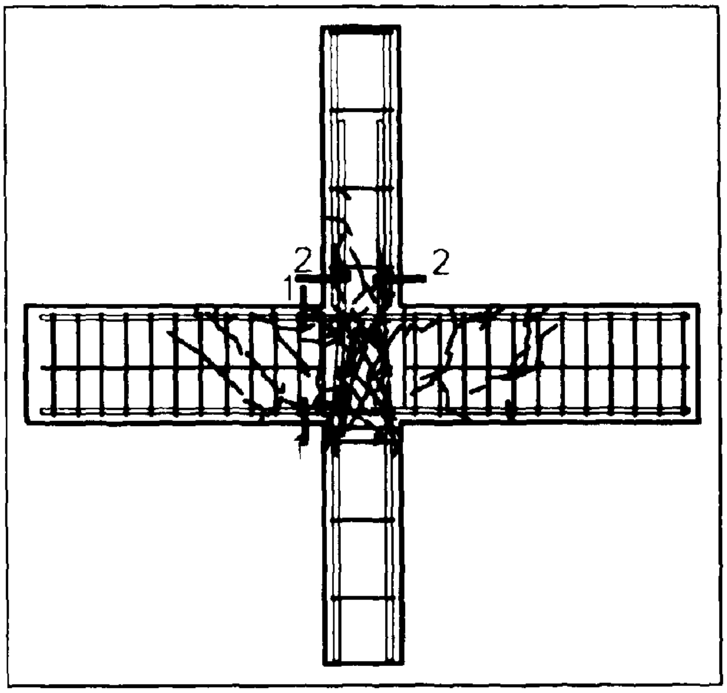
Spalling of concrete cover over a distance of 3 to 4 inches above and below the joint, and vertical cracking up to the first tie, occurred in the top column but the splices in general performed well. In some specimens (mainly with #7 bars in the column), splitting cracks along the splices occurred at final stages of the load history. In these cases, the concrete cover was lost, exposing the buckled column bars.



a. Moment–Rotation at Zone 1.



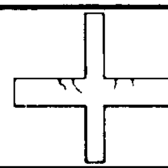
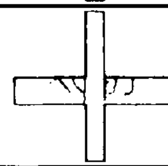
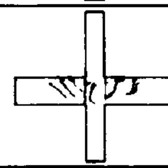
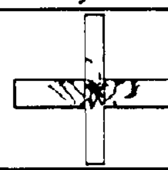
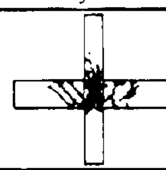
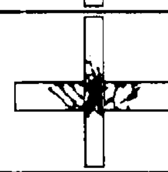
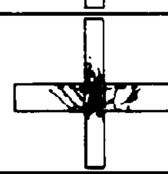
b. Moment–Rotation at Zone 2.



c. Cracking Pattern

FIGURE 4-2 Typical Interior Joint with Discontinuous Reinforcement (Specimen I-11)

TABLE 4-1 Load History of Specimen I-11

Cycle numbers	Control parameters	Lateral Load Direction	Peak values	Cracking pattern
0	Beam force (Sustained, gravity load)		+20, +20 kips	
1, 2, 3	Beam shear forces	+ -	+30, +10 kips +10, +30 kips	
4, 5, 6	Beam shear forces	+ -	+40, 0 kips 0, +30 kips	
7	Beam shear forces	+ -	+30, +10 kips +10, +30 kips	Low-level cycle
8, 9, 10	Beam shear forces	+ -	+50, -10 kips -10, +50 kips	
11	Beam shear forces	+ -	+30, +10 kips +10, +30 kips	Low-level cycle
12 13	Positive beam rotations	+ - + -	+0.013, +0.013 rad +0.015, +0.015 rad	
14 15	Positive beam rotations	+ - + -	+0.018, +0.018 rad +0.020, +0.020 rad	
16	Positive beam rotations	+ -	+0.028, +0.024 rad	

Maximum joint shear stresses at the peak upward forces were up to 40% less than in interior specimens with continuous positive reinforcement. The column axial force was the most significant variable. Specimens loaded with larger axial force (350 kips) exhibited up to 30% increase in load capacity. They had increased energy dissipation capacity and higher overall specimen stiffness in the initial cycles.

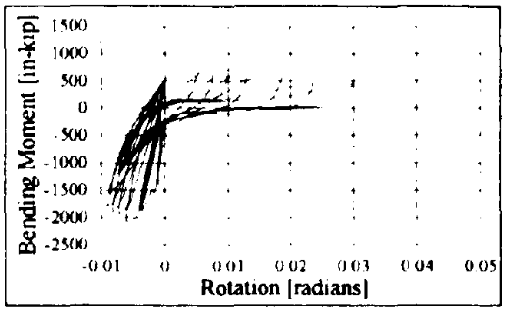
At peak strength the embedded rebars experienced 42 to 58 ksi stress, which was always below the yield stress of the Grade 60 bars. The size of the embedded reinforcement (3/4 and 1 inch diameter) did not significantly influence these values, though the rate of strength loss was larger in specimens with the smaller bars. The beams did not experience any significant distress except very close to the column.

Some specimens had transverse beam stubs to simulate the lateral confinement that would be provided by beams framing in perpendicular to the primary frame. The beam stubs produced no marked effect on strength capacity, stiffness degradation or the total dissipated energy. The presence of the beam stubs partly shifted the damage to the column, thereby altering the energy dissipation ratios among the members.

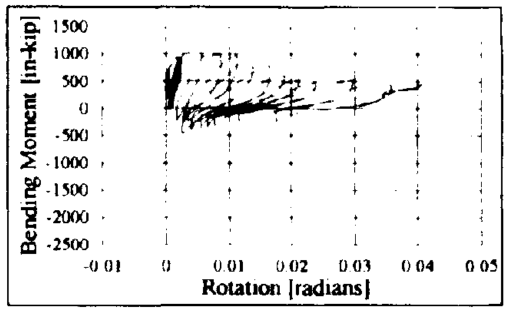
4.3 Exterior Joints with Discontinuous Reinforcement

Fourteen specimens were tested to study the behavior of exterior joint region. The same load history described in Section 3.3 was applied to simplify comparison with results from the interior joints.

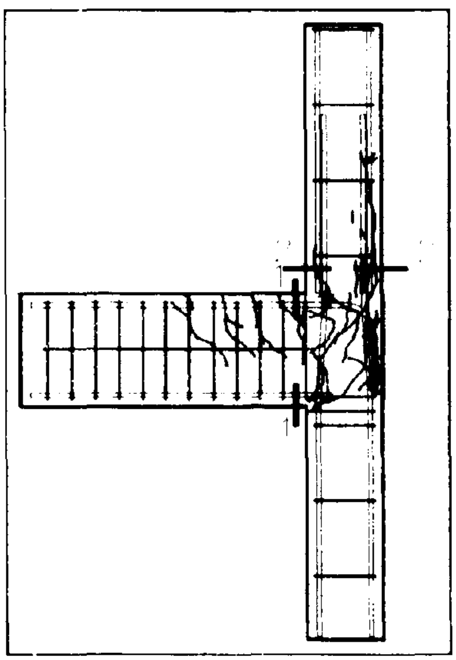
Moment-rotation plots for a typical specimen without transverse beam stubs are given in figures 4-3(a,b). Crack development and loading history is summarized in table 4-II. In contrast to the interior joints, downward loading on the beams had a major contribution to the failure of the exterior joints. Initial cracks appeared at the upper corner of the joint panel close to the beam during early load cycles. Under increasing loads, diagonal cracks developed perpendicular to the bent down reinforcement, causing a significant drop in the specimen stiffness. Finally, these cracks progressed diagonally across the joint both into the splice region and the embedment zone. The load carrying capacity dropped suddenly as cracking extended along the entire length of the splice, leading to buckling of the longitudinal column bars.



a. Moment–Rotation at Zone 1.



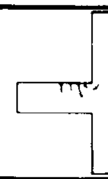
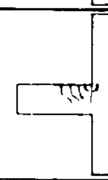
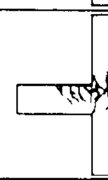
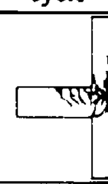
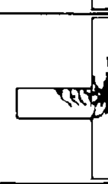
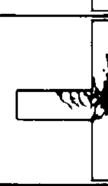
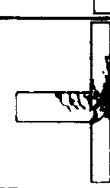
b. Moment–Rotation at Zone 2.



c. Cracking Pattern

FIGURE 4-3 Typical Exterior Joint with Discontinuous Reinforcement (Specimen E-01)

TABLE 4-II Load History of Specimen E-01

Cycle numbers	Control parameters	Lateral Load Direction	Peak values	Cracking pattern
0	Beam force (Sustained, gravity load)		+20 kips	
1, 2, 3	Beam shear force	+ -	+30 kips +10 kips	
4, 5, 6	Beam shear force	+ -	+40 kips 0 kips	
7	Beam shear force	+ -	+30 kips +10 kips	Low-level cycle
8, 9	Beam shear force	+ -	+44 kips -10 kips	
10 11	Column, beam Positive rotations	+ - + -	+0.011, -0.000 rad +0.014, +0.005 rad	
12 13	Column, beam Positive rotations	+ - + -	+0.016, +0.008 rad +0.019, +0.010 rad	
14 15 16 17	Column, beam Positive rotations	+ -	+0.021, +0.013 rad +0.024, +0.019 rad +0.030, +0.024 rad +0.036, +0.030 rad	

In some specimens, vertical flexural cracks developed at the interface of the beam and the joint panel. Additional load cycles induced a large opening of the construction joint above the beam, and drove the cracks along the splice. The final cracking pattern for Specimen E-01 is shown in Figure 4-3(c). Under negative bending, the prying action of the bent-down negative beam reinforcement initiated cracking along the vertical extension portion of the hook and often produced full separation of the concrete cover layer opposite the beam (extending from the lower construction joint to the splice region). The applied positive bending of the beams caused further deterioration of the embedment zone, but the pull-out action was not as dominating as in the interior joints.

Specimens with transverse beam stubs showed a similar failure mechanism; however, cracking was less severe. Pulling out of the bottom beam bars initiated at about the same load as intensive cracking at the splices. Transverse confinement did not increase the peak load capacity but provided a more gradual strength degradation. Specimens tested at the higher level of column axial force (350 kips) or those having 2-#3 ties within the joint showed about 15-25% higher strength, as was the case for interior specimens.

In summary, failure occurred by a combination of excessive diagonal shear cracking followed by splice failure in the top column, spalling of the concrete cover due to the prying action of the bent-down negative beam reinforcement, and to a smaller extent, pullout of the embedded positive beam reinforcement. There was negligible damage exhibited in the lower column or in the beam.

SECTION 5

COMPARATIVE PARAMETRIC STUDY

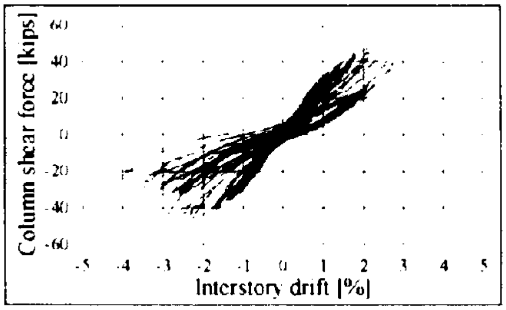
The following study is primarily based on the recorded column shear force and interstory drift data reflecting the capacity of the entire subassembly. Typical hysteretic plots for the three groups of specimens are shown in figures 5-1(a,b,c). The strength values used in the calculations were normalized to 3500 psi concrete strength based on the compressive test results of site-cured cylinder samples of the concrete cast in the beams and joints.

Results on interior joints with continuous positive beam reinforcement were detailed by Pessiki et al. [1990]. Therefore, the focus here is on beam-column connections with discontinuous reinforcement. Discussion in this report is limited to the highlights of the main findings. Detailed results for joint shear factors, stiffness degradation and energy dissipation are provided in a supplementary report [Beres et al. 1992].

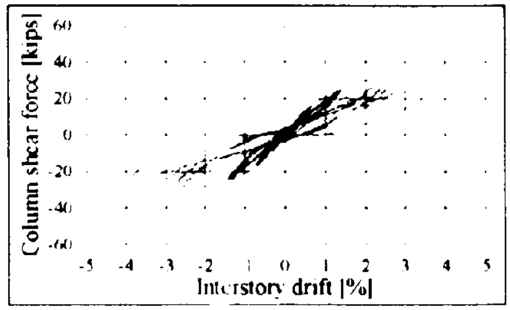
5.1 Strength and Ductility

In this section, the experimental results are examined first in terms of the column shear forces followed by a discussion of joint shear strength capacity. Figures 5-2 and 5-3 show the strength degradation patterns (envelopes of hysteresis graphs) for interior and exterior specimens with discontinuous reinforcement, grouped according to the presence of the transverse beam stubs.

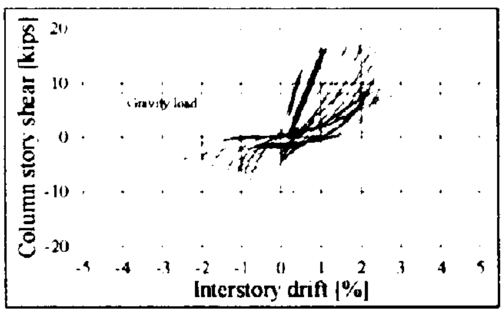
In the interior joints, the peak strength values occurred at about 1.5-2% interstory drift showing about 30% scatter in the peak capacities. The higher column axial force ($0.39A_cf_c'$) resulted in higher peak strength values with a usually more rapid strength degradation (details of comparison studies are shown in the supplementary report). There was little difference in the subassembly strength based on other factors, such as the presence of the transverse beam or the amount of reinforcement



(a) Interior Joint, Continuous Positive Bars

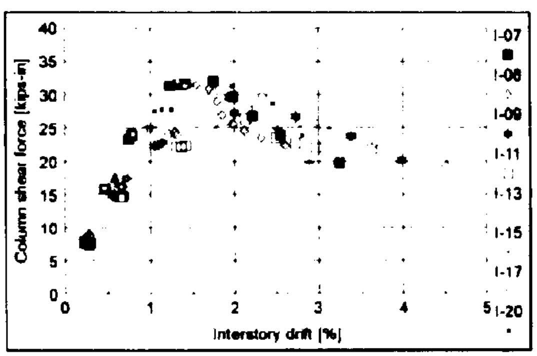


(b) Interior Joint, Discont. Positive Bars

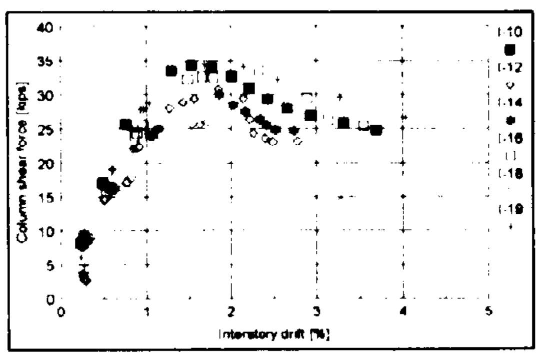


(c) Exterior Joint

FIGURE 5-1 Typical Column Shear Force vs. Interstory Drift Plots

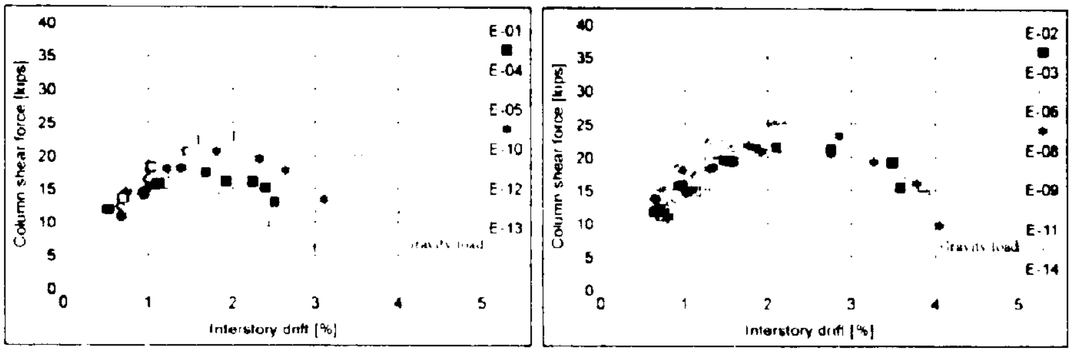


a. Without Transverse Beam Stub



b. With Transverse Beam Stub

FIGURE 5-2 Strength Deterioration of Interior Joint Specimens



a. Without Transverse Beam Stub **b. With Transverse Beam Stub**

FIGURE 5-3 Strength Deterioration of Exterior Joint Specimens

For exterior joints the peak strength values were reached at about 1.5-2.7% interstory drift with about 40% scatter of the peak capacities. Higher column axial force or the presence of 2-#3 ties within the joint produced higher maximum strength capacities and a more gradual strength degradation. The presence of transverse beams also resulted in slower strength degradation but no increase in capacity.

The interior joint specimens reached higher total peak column shear strength capacity values than the exterior joint specimens as shown in figure 5-4. However, in the exterior joints, the gravity loads induce shear forces in the joint taking a substantial portion of the total shear capacity. Consequently, the shear capacity available for the lateral loading of interior joints is markedly higher than that of exterior connections with the given proportions. The maximum strength values of the exterior connections occurred at slightly higher values of interstory drift. Considering the initial drift of 0.3-0.6% for the exterior joints at gravity load, in a frame subjected to lateral loads, both interior and exterior joints should reach their peak strength approximately at the same additional drift value. Therefore, assuming a rigid floor diaphragm, strength degradation is expected to start almost simultaneously.

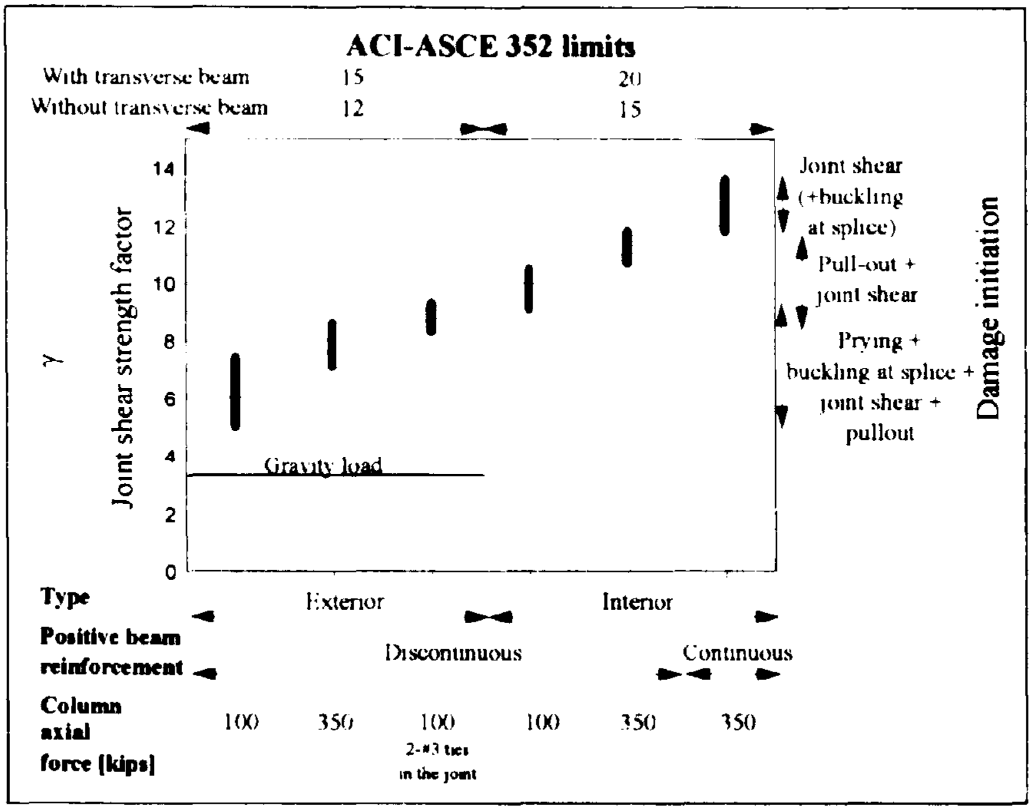


FIGURE 5-4 Summary Chart of the γ Factors for All Beam-Column Joint Specimens

As was pointed out in the behavior descriptions, damage to the specimens was almost always confined to the joint-panel and the adjacent regions. Loss of strength capacity was often related to diagonal cracking within the joint. Although in some specimens the initiation of damage and the subsequent strength degradation was attributed to reasons other than high joint shear (e.g., pullout of the positive beam reinforcement, buckling of the longitudinal bars at the lightly confined splice region, prying of the bent-down negative rebars), the strength capacity expressed in terms of the joint shear strength coefficients may serve as a good basis for comparing the effectiveness of different joint configurations.

The joint shear strength factors (γ) summarized in table 5-I are set by the ACI-ASCE 352 Committee [1991] for the design of reinforced concrete joints. The recommendations are mostly based on the state-of-art reviews of Meinheit, and Jirsa [1982] and Kurose, Guimaraes, Liu, Kreger, and Jirsa [1988] and have been partly incorporated to the 1989 ACI-318 Building Code. According to ACI-ASCE 352R, the nominal joint shear peak strength is calculated as:

$$V_n = \gamma \sqrt{f'_c}\, b_j h$$

where $b_j h$ is the effective joint cross section area

TABLE 5-I γ Joint Shear Strength Factors (ACI-ASCE 352)

Joint Type	Interior	Exterior	Corner
Type 1	24	20	15
Type 2	***20***	***15***	***12***

The interior-exterior-corner classification of the ACI-ASCE 352 reports, used in table 5-I and explained by figure 5-5, is different from the terminology used in this report. Joints are also classified as type 1 and type 2 joints. The fundamental difference between these two types is in the characteristics of the assumed loading conditions and the anticipated deformations. The lower values designated for type 2 joints are applied to frames designed to resist significant lateral loads, where the reversing cyclic action causes inelastic behavior. Type 1 joints connect members in which no significant inelastic deformation is expected and these joints are not required to dissipate energy during load reversals. Therefore, to evaluate joints in structures subjected to seismic loading, although they were not designed originally for this type of effects, the lower γ factors belonging to type 2 class should be considered.

It was also anticipated that the actual γ values calculated from the experimental results will be lower that those specified in table 5-I because details of the tested specimens violate several requirements set for those values. The discrepancies are:

- Unsatisfactory member bending moment capacity ratio. The ratio of the sum of column moment capacity and sum of beam moment capacity ought to be larger than 1.4. This value is not reached even if pullout is considered
- Lack of confining ties within the joint where there are less than four beams framing in
- Apparent rebar buckling and potential yielding in specimens in which #7 column bars were used
- Excessive spacing of the longitudinal column reinforcement (the maximum center-to-center spacing allowed between adjacent longitudinal bars is the larger of either 8 inches or one-third of the column cross section dimension in the direction the spacing is being considered)
- Inadequate anchorage of the discontinuous positive beam reinforcement
- Inadequate embedment of the #8 negative beam reinforcement within the joint (minimum h/d_b 20 is required, the test specimens have h/d_b 16 for the #8 rebars)

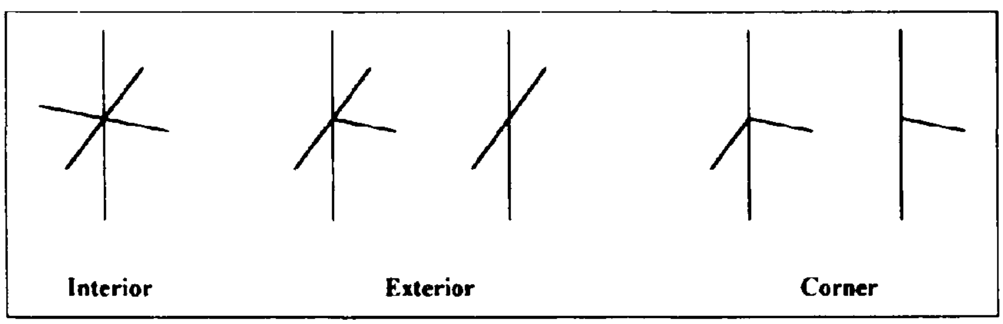


FIGURE 5-5 Joint Classification According to ACI-ASCE 352

Even though requirements set by ACI-ASCE 352R appear to be strict, they imply that some joint shear cracking and energy dissipation will occur when the beam-column joints are loaded close to their ultimate capacity. The influence of various parameters on the peak capacity[1] of the specimens expressed in terms of γ joint shear strength factor is shown in figure 5-4. As it was expected, the actual γ values were 30-40% below the limiting values specified by ACI-ASCE 352R for "properly" detailed connection regions. A strength increase of 15-25% was detected when higher column axial load was used. There is no provision in ACI-ASCE 352R for the magnitude of the axial force. While ACI-ASCE 352R suggests a 25-33% difference depending on the presence of the transverse beams, for the details examined here the experimental results did not support this.

The several detected damage initiation modes showed a closely spaced failure hierarchy. The only case where the joint shear strength clearly governed the capacity was in interior joints with continuous beam reinforcement and #10 longitudinal column bars. Since no transverse reinforcement was provided in those joints, the concrete capacity should have had a decisive role. An earlier version of the ACI-ASCE 352R [1976] was conceptually different from the 1991 report, including separate strength contribution values assigned to the concrete and the transverse reinforcement. The shear capacity provided by the concrete was formulated as:

$$V_c = 3.5\gamma_{76}\beta\sqrt{1 + 0.002\frac{N_u}{A_g}}\sqrt{f'_c}\,b_j h$$

where

V_c = Shear force capacity of the joint provided by the concrete alone.

β = 1.0 (for type 2 joints)

γ_{76} = 1.0 for joints without transverse beams, 1.4 for joints with transverse beams

---

[1] Although the specimens experienced significant peak deformations, the second-order effects were estimated not to influence more than 3-4% the joint shear strength factors.

(the subscript 76 refers to ACI-ASCE 352 report version 1976 to distinguish from γ used in later versions)

N_u = Column axial force (lb)

A_g = Column cross section area (in)

To compare the specified concrete contribution with the test results a γ_c factor is defined here reflecting the concrete contribution to the joint shear strength Results of calculations on the tested specimens are shown in table 5-II

$$\gamma_c = 3.5\gamma_{76}\beta\sqrt{1 + 0.002\frac{N_u}{A_g}}$$

As mentioned above, the only case when the specimens might have experienced purely joint shear failure was in the interior specimens with continuous reinforcement. In those specimens, γ was about 12-13, significantly higher than the values listed in table 5-II Note that the γ_c formula takes into account the role of the axial force. The column normal stress term in the above formula resulted in 33% and 93% increases of γ_c value for axial forces of 100 kips and 350 kips, respectively

TABLE 5-II Joint Shear Strength Concrete Contribution Factors

γ_c	Joints without beam stub	Joints with beam stub
N_u= 100 kips	4.7	6.5
N_u= 350 kips	6.8	9.4

A 40% increase for joints with transverse beams was also built into this formula with the γ_{76} factor The experimental results described here do not support this difference in capacity It is possible that the prevalence of other failure initiations might have diminished the influence of the transverse confinement.

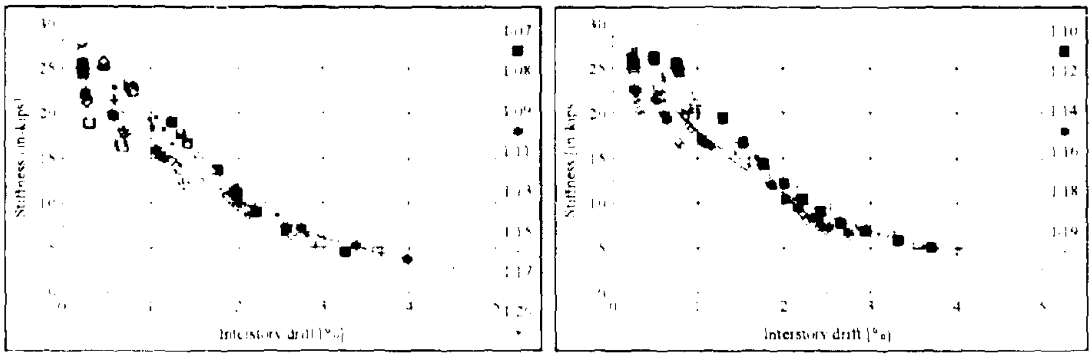
5.2 Stiffness

This section discusses the calculated results of the stiffness variation with increasing deformations of the entire beam-column subassembly Figures 5-6 and 5-7 show the stiffness deterioration patterns for all interior and exterior specimens with discontinuous reinforcement The figures are grouped depending on the presence of the transverse beam stubs The subassembly stiffness values were extracted from the column shear force versus interstory drift plots In the interior joints, stiffness was approximated as tangent of the peak-to-peak lines Since the hysteresis loops of the exterior connections exhibited pinching and largely unsymmetrical hysteretic behavior, stiffness values were generated from the secant going through the first x-axis intercept of each loop and the peak in the first quadrant as shown in figure 5-8

All specimens examined exhibited rapid stiffness deterioration due to various factors, such as nonlinear elastic deformations, flexural and shear cracking, distortion of the joint panel, slippage of reinforcement, loss of cover, and concentrated flexibility at the construction joints The higher axial force on the column resulted in the highest initial stiffness values This was mostly attributed to the closure of shrinkage cracks, the delay of the tensile cracking and the opening of the construction joints There was no marked difference in the subassembly stiffness based on other factors, such as presence of the transverse beams or the amount of column reinforcement Also, the initial scatter is partly attributed to the imperfections of the seating of the specimen and the instrumentation.

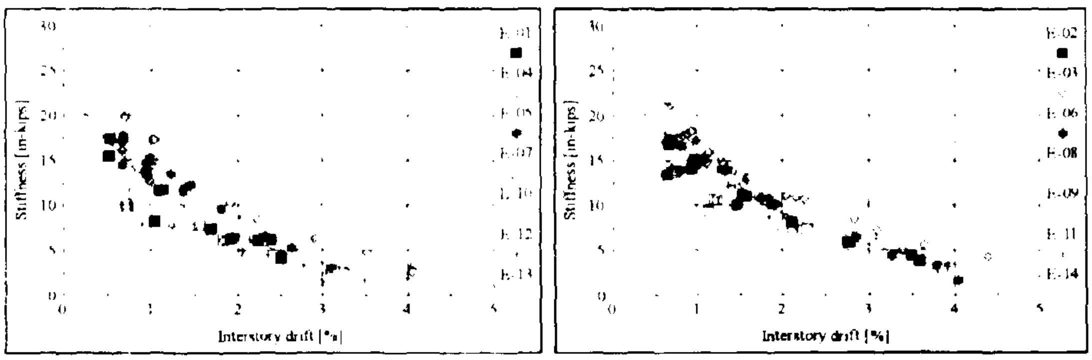
In the interior joints, about 30% maximum scatter of stiffness values can be seen during the initial cycles while for exterior joints, the maximum scatter is about 50% This variation gradually decreases, in both types of joints, with the application of more cycles. The presence of 2-#3 ties within the joint panel (with 100 kips column axial force) provided as high an initial stiffness as that provided by the presence of the higher (350 kips) axial force but with no ties.

There is no marked difference in the subassembly stiffness based on other factors, such as presence of the transverse beams or the amount of reinforcement except for the specimen with #7 longitudinal column bars which had lower stiffness



a. Without Transverse Beam Stub **b. With Transverse Beam Stub**

FIGURE 5-6 Stiffness Degradation of Interior Joint Specimens



a. Without Transverse Beam Stub **b. With Transverse Beam Stub**

FIGURE 5-7 Stiffness Degradation of Exterior Joint Specimens

The average stiffness values obtained using an exponential regression function (parameters are listed in Appendix B) for exterior joints were about 35-50% less than those for the interior joints at a given story drift level as shown in figure 5-9. Out of the 35-50% stiffness decrease

5-15% can be attributed to the difference in calculating the stiffness and about 25-30% to the number of the adjoining members

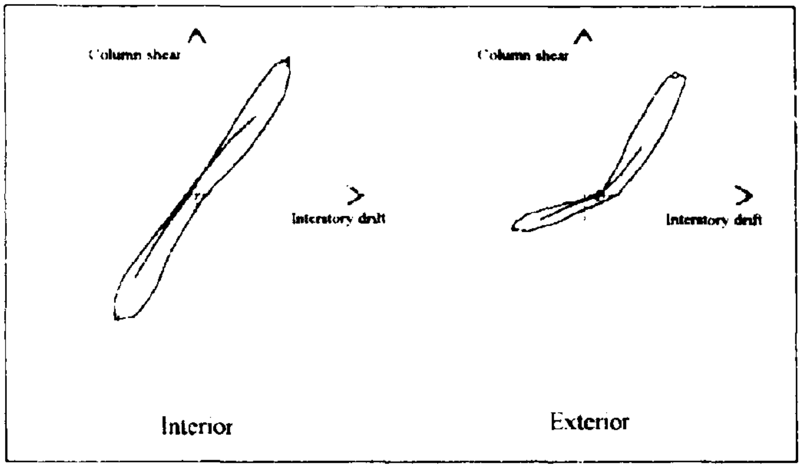


FIGURE 5-8 Approximation of Stiffness for Interior and Exterior Joints

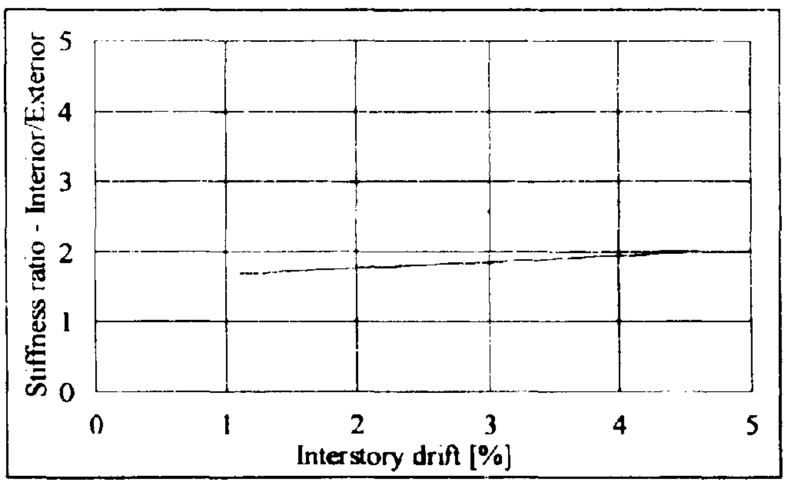
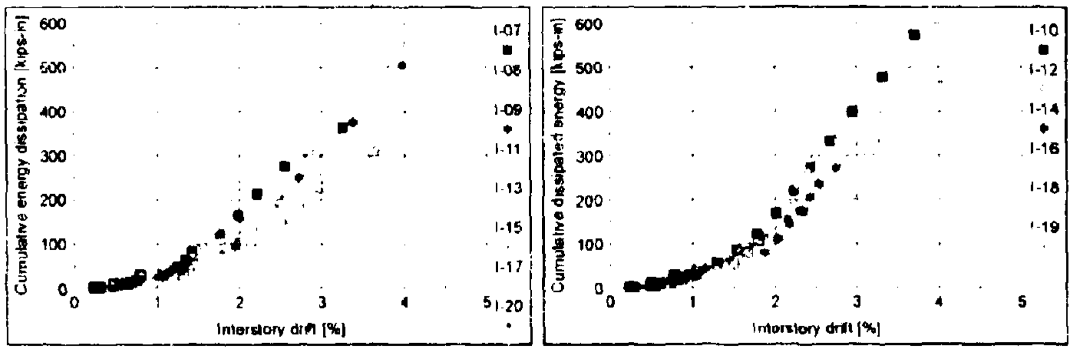


FIGURE 5-9 Stiffness Ratio - Interior versus Exterior Joints

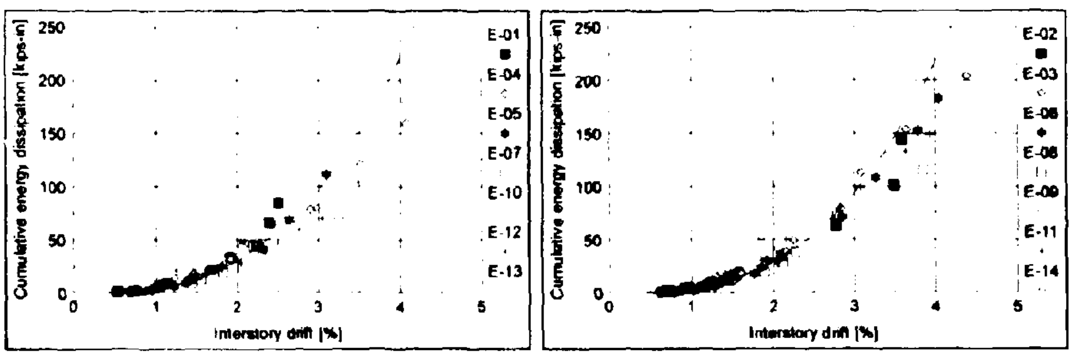
5.3 Energy Dissipation

Energy dissipation results presented here in figures 5-10 and 5-11 were approximated by computing the areas enclosed within the loops of the column shear versus the interstory drift plots.



a. Without Transverse Beam Stub **b. With Transverse Beam Stub**

FIGURE 5-10 Cumulative Energy Dissipation of Interior Joint Specimens



a. Without Transverse Beam Stub **b. With Transverse Beam Stub**

FIGURE 5-11 Cumulative Energy Dissipation of Exterior Joint Specimens

In interior joint specimens, the cumulative dissipated energy values showed little variation up to about 1.5% interstory drift. At higher drift levels, the scatter increased. The higher axial load produced higher (up to about 170%-200%) energy dissipation. The transverse beams had negligible influence on the cumulative energy curves. However, the presence of the transverse beams made a difference in the damage distribution among the adjoining beams and columns (the ratios of energy dissipation among the members are shown in Appendix C):

1. If there were no transverse beam stubs present, the beams had dominant participation, the upper column less and the lower column negligible.

2. If there were beam stubs, the participation of the upper column regions increased to comparable level to those of the beams. Also, the energy ratio corresponding to the lower column increased.

3. The most dramatic difference is exhibited between I-19 and I-20 (both with #7 column bars and 350 kips column axial force) because of the apparent buckling of the column bars in specimen I-20 within the joint when there was no confinement provided by the transverse beam.

The average energy dissipation values obtained using a power regression function showed that the typical exterior joint dissipated on the average about half as much energy at a given drift level as a typical interior joint (figure 5-12). This fact is partly the result of the fewer cycles applied to cause identical interstory drift values and the lower strength capacity. No trend was noticeable on the total cumulative dissipated energy curves related to any of the changed parameters.

The beam region next to the joint had higher participation (about 40%). Also, energy dissipated at the upper column was significant. The lower column and regions not adjacent to the joint played a negligible role. When #7 column bars and transverse beam stubs (specimens E-11, E-14) were present, the beam and column members adjacent to the joint dissipated almost all the energy.

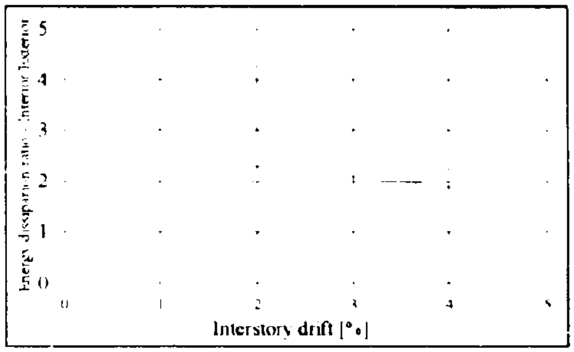


FIGURE 5-12 Cumulative Energy Dissipation Ratio - Interior versus Exterior

SECTION 6

SUMMARY

This study is a part of a report series on current experimental research at Cornell University on existing reinforced concrete frame structures, designed primarily for gravity induced loads. These reinforced concrete frames were detailed with little or no attention given to lateral load effects, and are characterized by non-ductile details. Having the details that are in contrast to modern seismic design, these structures are suspect even in low-to-moderate seismicity zones.

Beam-to-column connections are regarded as the critical parts of frame structures under seismic loading. This report describes the full-scale test series conducted on the behavior of interior and exterior beam-to-column joints. The experimental program covered a wide range of parameters including different geometries and reinforcing configurations using thirty-four specimens.

Section 2 provides a background on the subject discussing the characteristics and critical details of the frames under examination. It also reviews the influence of past ACI Code requirements on detailing and includes references to the relevant technical literature. Section 3 describes the testing program including the specifics of the testing methodology and the examined specimens.

Section 4 summarizes the damage development characteristics for the specimens grouped to three categories: interior joints with continuous and discontinuous positive moment reinforcement and exterior joint with discontinuous bars. Finally, Section 5 contains a study examining the effect of various parameters on strength, stiffness, and energy dissipation.

This reports findings are confined to the examined geometries. A companion NCEER report (Part II) will discuss the subject in a more generalized framework including modeling and analysis of frames with non-ductile details and implications on building behavior.

SECTION 7

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APPENDIX A

TEST PARAMETERS OF THE SPECIMENS

Table A-1 Test Parameters of Exterior Joint Specimens

Specimen Identifier	Column Axial Load [kips]	Presence of Transverse Beam Stub	Column Longitudinal Reinforcement	Positive Beam Reinforcement	Other Significant Details
E-01	100	w/o	4-#10	2-#8	
E-02	100	w	4-#10	2-#8	
E-03	100	w	4-#10	2-#8	2-#3 ties in the joint
E-04	100	w/o	4-#10	2-#8	2-#3 ties in the joint
E-05	350	w/o	4-#10	2-#8	
E-06	350	w	4-#10	2-#8	
E-07	100	w/o	4-#10	2-#6	
E-08	100	w	4-#10	2-#6	
E-09	350	w	4-#10	2-#6	
E-10	350	w/o	4-#10	2-#6	
E-11	100	w	4-#7	2-#8	
E-12	100	w/o	4-#7	2-#8	
E-13	100	w/o	4-#7	2-#8	Separated splices
E-14	350	w	4-#7	2-#8	Separated splices

Table A-II Test Parameters of Interior Joint Specimens

Specimen Identifier	Column Axial Load [kips]	Presence of Transverse Beam Stub	Column Longitudinal Reinforcement	Embedded Positive Beam Reinforcement	Other Significant Details
I-01	350	w/o	4-#10	2-#9 continuous	unspliced column bars
I-02	350	w/o	4-#10	2-#9 continuous	
I-03	350	w/o	6-#8	2-#9 continuous	
I-04	350	w/o	8-#7	2-#9 continuous	
I-05	350	w/o	8-#7	2-#9 continuous	2-#3 ties in the joint
I-06	350	w/o	8-#7	2-#9 continuous	6-#3 ties in the joint
I-07	350	w/o	4-#10	2-#8 discontinuous	
I-08	350	w/o	4-#10	2-#6 discontinuous	
I-09	100	w/o	4-#10	2-#8 discontinuous	
I-10	350	w	4-#10	2-#8 discontinuous	
I-11	100	w/o	4-#10	2-#6 discontinuous	
I-12	350	w	4-#10	2-#6 discontinuous	
I-13	100	w/o	4-#10	2-#6 discontinuous	
I-14	100	w	4-#10	2-#6 discontinuous	
I-15	350	w/o	4-#10	2-#8 discontinuous	
I-16	350	w	4-#10	2-#8 discontinuous	
I-17	100	w/o	4-#7	2-#8 discontinuous	
I-18	100	w	4-#7	2-#8 discontinuous	
I-19	350	w	4-#7	2-#8 discontinuous	
I-20	350	w/o	4-#7	2-#8 discontinuous	

APPENDIX B

REGRESSION DATA OF THE STIFFNESS DEGRADATION AND ENERGY DISSIPATION PLOTS

Table B-I Regression Parameters for the Stiffness Degradation Plots

Regression parameters	Interior joints		Exterior joints	
	Without	With	Without	With
	Transverse beam stub		Transverse beam stub	
a	28.1	28.73	23.6	24.27
b	-0.51	-0.48	-0.63	-0.49
R^2	0.94	0.94	0.76	0.88

($y=ax^b$ power function was used for the regression)

Table B-II Regression Parameters for the Energy Dissipation Plots

Regression parameters	Interior joints		Exterior joints	
	Without	With	Without	With
	Transverse beam stub		Transverse beam stub	
a	28.74	29.72	4.28	3.93
b	2.13	2.18	2.85	2.89
R^2	0.97	0.98	0.92	0.95

($y=ae^{bx}$ exponential function was used for the regression)

APPENDIX C

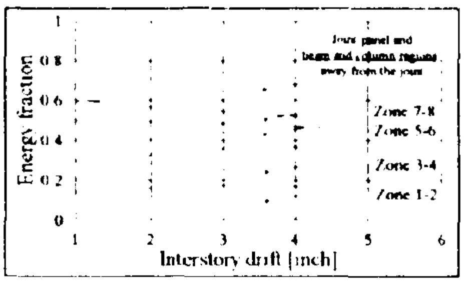
PARTICIPATION OF THE INDIVIDUAL BEAM AND COLUMN ZONES IN THE TOTAL CUMULATED ENERGY DISSIPATION

Interior Joints

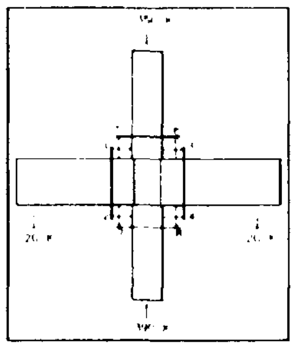
Without **With**

Transverse beam stub

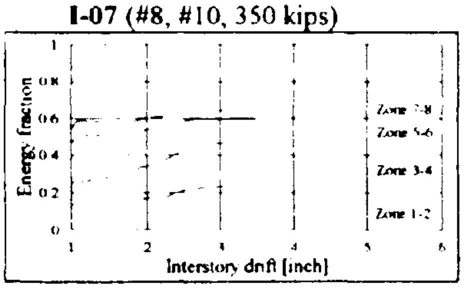
(beam rebar, column rebar, column axial force)



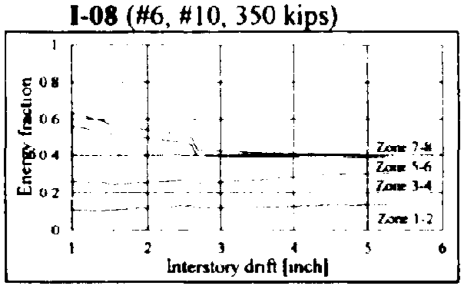
I-07 (#8, #10, 350 kips)



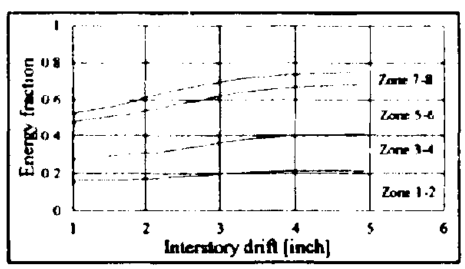
Location of transducers



I-08 (#6, #10, 350 kips)

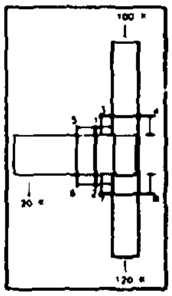


I-09 (#8, #10, 100 kips)



I-10 (#8, #10, 350 kips)

Exterior Joints

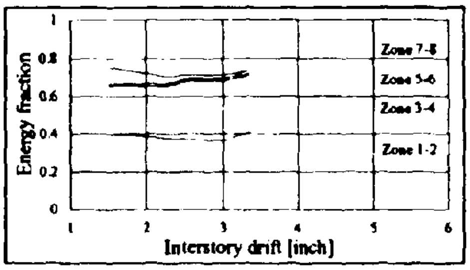


Location of transducers

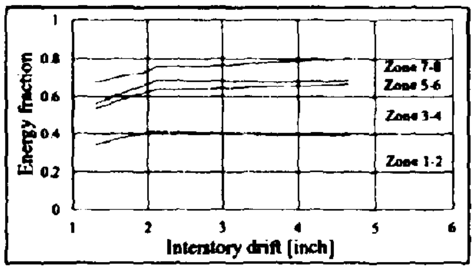
Without **With**

Transverse beam stub

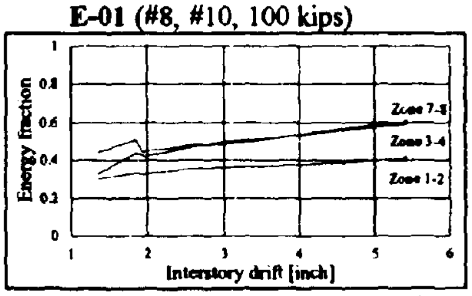
(beam rebar, column rebar, column axial force)



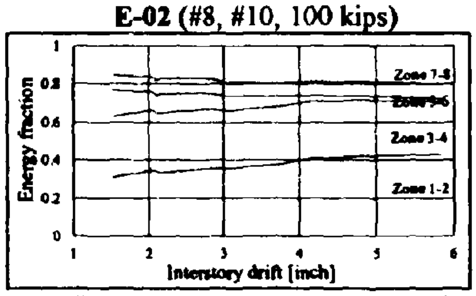
E-01 (#8, #10, 100 kips)



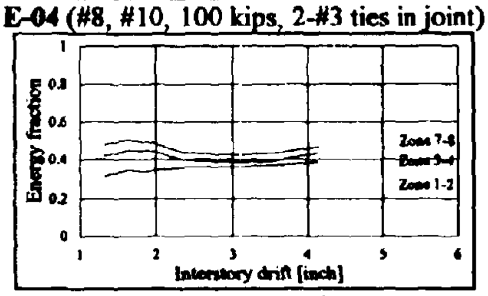
E-02 (#8, #10, 100 kips)



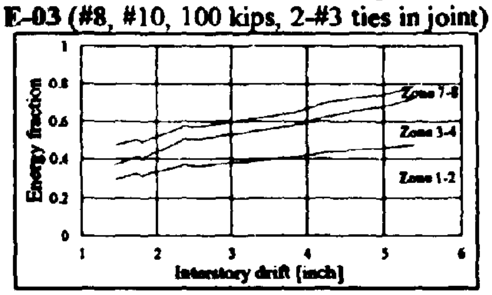
E-04 (#8, #10, 100 kips, 2-#3 ties in joint)



E-03 (#8, #10, 100 kips, 2-#3 ties in joint)



E-05 (#8, #10, 350 kips)

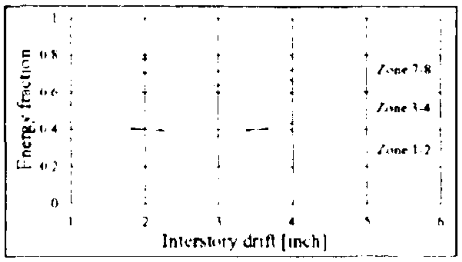


E-06 (#8, #10, 350 kips)

Without **With**

Transverse beam stub

(beam rebar, column rebar, column axial force)



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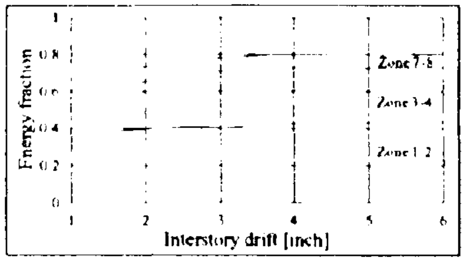

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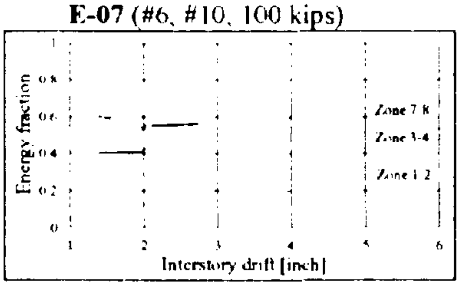

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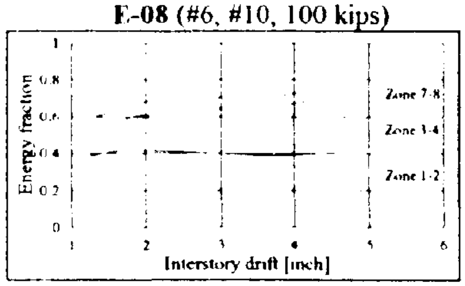

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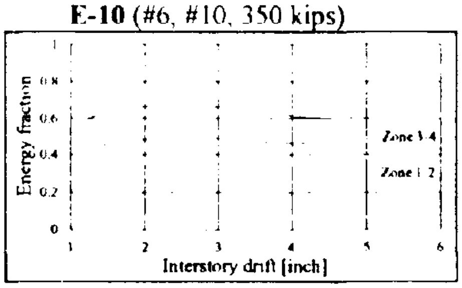

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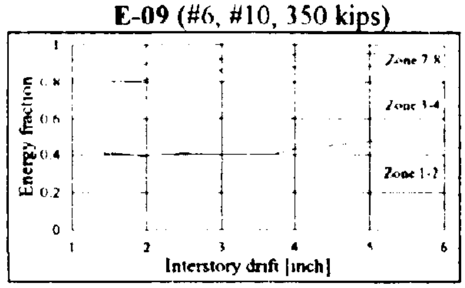

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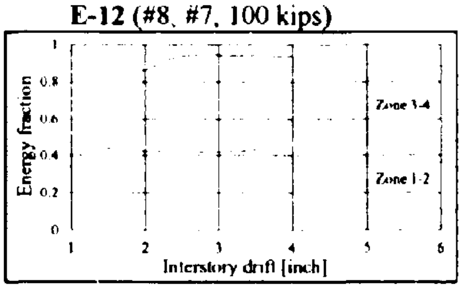

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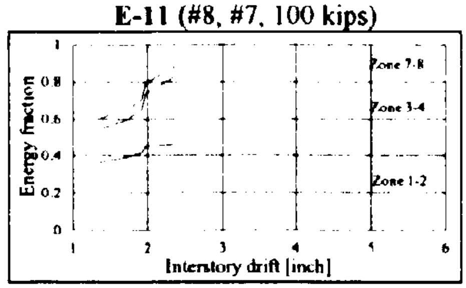

E-14 (#8, #7, 350 kips)

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