Retrofitting Beam-to-Column Joints for Improved Seismic Performance

by

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ABSTRACT

Before the 1970's, most codes for the design of reinforced concrete structures did not include provisions for ductility during seismic events was not prevalent in most codes. The guidelines that did exist were minimal, and often left a fair amount of room for interpretation by the design engineer. Hence, many of the reinforced concrete structures designed during that time are suspect under today's more stringent design guidelines. Moreover, even the present designs are often deficient and vary from building to building and from jurisdiction to jurisdiction.

This report is a presentation of the findings of an experimental study to evaluate a method of retrofit which addresses a particular weakness that is often found in reinforced concrete structures, namely the lack of sufficient reinforcement in and around beam-to-column joints. Many of these structures lack the required confining reinforcement within the joints and in adjoining beams and columns. The result is a reinforced concrete frame that is weak in the joint area and lacks sufficient ductility during a seismic event.

The proposed retrofit method consists of encasing the reinforced concrete joint with a grouted steel jacket that provides confinement to the joint area, and imparts ductility to the frame. In this study, two styles of retrofit jacket were tested: a circular steel tube and a rectangular casing. It was found that circular steel jackets have the advantage of providing

direct concrete confinement and, as well, of furnishing a ductile force transfer mechanism through the jacket itself, but are also more difficult and expensive to fabricate than rectangular jackets. Although rectangular jackets do not provide the same degree of concrete core confinement as circular jackets, the amount available seems sufficient to prevent damage in the joint area. The load transfer mechanism of the rectangular jackets was found to be adequate in withstanding the loads and deflections typical for seismic events.

In this thesis, the two jacket styles are evaluated for strength, stiffness and ductility, and their relative merits are discussed.

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CHAPTER 1

BACKGROUND AND PROJECT PROPOSAL

1.1 INTRODUCTION

In the past, earthquakes have occurred in many different areas of the world, and Canada has not been immune to this natural disaster. In fact, parts of Canada face an extreme risk of a very large earthquake sometime in the future. In such hazard regions, considerable attention must be paid to the seismic design and earthquake construction of buildings and other structures, as well as to the seismic capacity of existing structures. Older reinforced concrete buildings are of special concern. Many of these do not meet the reinforcing and detailing requirements of modern building codes and therefore are extremely susceptible to the effects of cyclic loads, because of the brittle nature of the material.

Although possessing superior strength in compression, concrete has poor material characteristics in tension and relies primarily on reinforcing steel for strength and ductility. Large structures are routinely constructed of reinforced concrete, the design of which is governed by appropriate design codes, such as the ACI Standard 318-1983 in the United States, and the CAN3-A23.3-M84 standard in Canada. These codes are compiled and regularly updated by technical experts using current research information. Over the years, as more and more information regarding the behaviour of concrete structures under seismic action becomes available, engineers gain a better understanding about the design of structures for seismic effects, and design codes are continually modified to reflect this increase in knowledge.

In the 1950's and 1960's, design and construction practices were different than they are today, especially where seismic effects are concerned. In particular, the relatively small amount of transverse reinforcement prevalent in the pre-1970's designs has resulted in the existence of many moment-resisting frames that today are considered to have a high risk of developing shear failures in the columns and of exhibiting a low level of ductility and energy dissipation ability. If such structures do not include other structural elements that may contribute to the overall ductility and provide other sources for dissipating energy, they may prove to be seriously inadequate when compared to current design standards.

A major turning point for seismic design, at least in North America, was the 1971 San Fernando Valley earthquake in California. Many reinforced concrete structures that were built to the design and construction standards at that time behaved poorly during the earthquake, prompting many modifications to the then existing ACI and Canadian codes.

1.2 IMPORTANCE OF DUCTILITY

Most code modifications referred to above, and implemented after the San Fernando earthquake addressed the ductility requirements of reinforced concrete members and, in particular, much attention was focused on the joint region between beams and columns. The joint region is subject to large shear forces during lateral seismic loading, particularly when beam moments on opposite faces of a column have the same direction. The longitudinal reinforcement is stressed in the same direction in this situation, and the bond between the reinforcement and the concrete is heavily relied upon to provide the required transfer of forces through the joint. Under severe seismic loading, plastic hinges are expected to form at the ends of the beams adjacent to the joint, and transverse reinforcement in both the beam and the column are required to provide confinement to the concrete in the core region, thereby safeguarding the ductility of the joint. All of the aformentioned points were examined following the 1971 San Fernando Valley earthquake and the design codes were modified accordingly, in line with the recommendations of ACI-ASCE Committee 352 for the design of connections in reinforced concrete frames with ductile moment-resisting capacity (ACI, 1991).

While updated design codes address the construction of new structures, structures that were built according to earlier design codes may not meet today's seismic standards. Many are inadequate, and may pose a severe risk to society. What can be done about them? One available option is to retrofit such structures. Retrofit is the process of modifying an existing structure so that it meets the current code design provisions.

1.3 OBJECT AND SCOPE OF INVESTIGATION

The purpose of this study was to investigate several methods of retrofit that can be applied to strengthen the joint region of a beam-to-column connection in a reinforced concrete frame built in accordance with earlier design codes. One of the more common characteristics of structures designed in accordance with outdated codes is that they possess insufficient transverse reinforcement in the joint core, resulting in inadequate ductility in the connection area. The method of retrofit that was studied here is to encase the deficient (or damaged) reinforced concrete members and joint with a steel jacket, and to fill the void between the jacket and the joint members with concrete grout. This is an elegant and simple solution; the steel provides shear reinforcement and adds ductility to the joint through its action of confining the core concrete. This approach can be applied to both existing, undamaged but deficient structures and, when appropriate, to existing structures which have been damaged by an earthquake.

The effectiveness of introducing a steel jacket around the beam-to-column connection was examined by performing six cyclic loading tests on a total of four specimens built according to typical 1960's design specificatons (fig. 1-1). The testing program included two tests on unretrofitted specimens, causing some initial damage and intended as a likely scenario after an earthquake. The last four tests were performed on the two damaged and the two undamaged specimens after they were retrofitted with a confining steel jacket (fig. 1-2). Two jacket types were tested, and the results compared. Two of the specimens were retrofitted with a circular jacket, the other two with a rectangular jacket. Each jacket type was tested with one damaged and one undamaged specimen, so as to facilitate the comparison of the effect of the initial damage on the behaviour of the retrofit. However, due to changes in the design details of the retrofits which followed the outcome of some of the initial tests, a comparison between previously damaged and undamaged specimens was not possible.

It is well known that circular jackets provide superior confinement and hence better

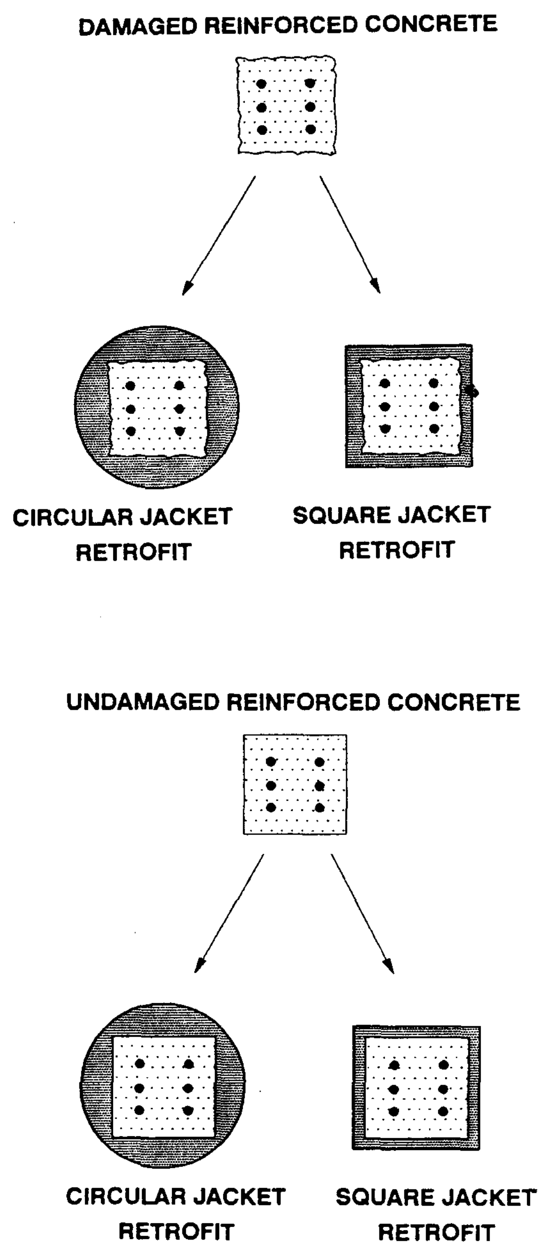


Figure 1-1 Testing Program

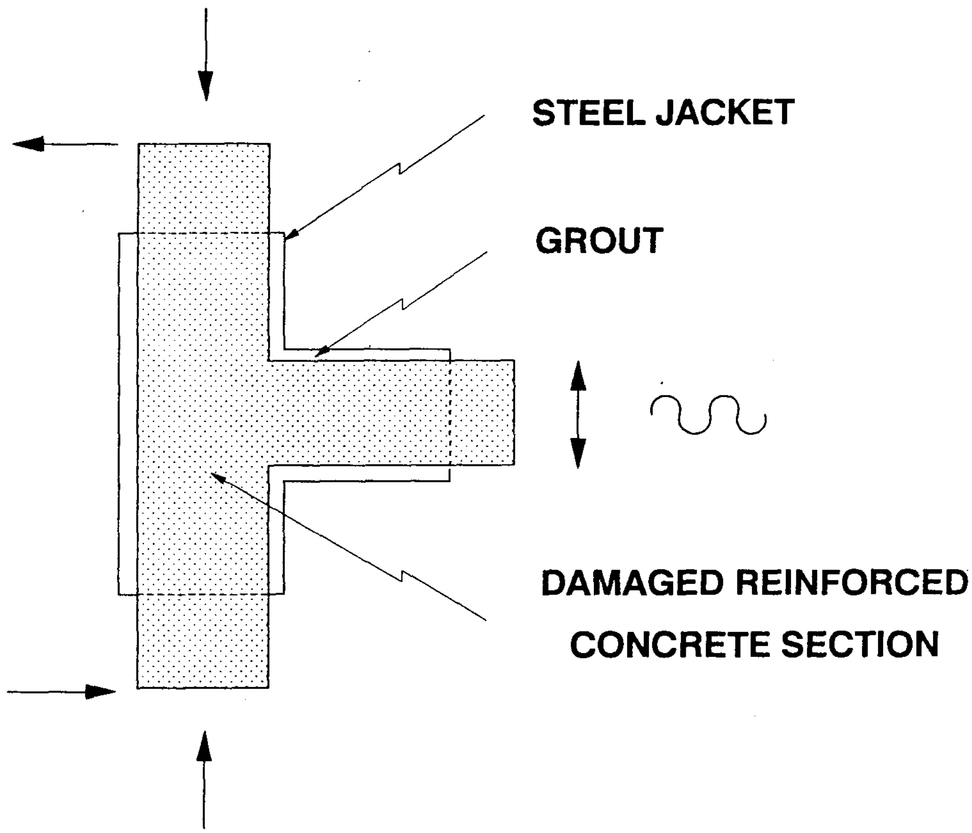


Figure 1-2 Beam-to-Column Joint Cyclic Loading Test

ductility to the core concrete than square or rectangular jackets (Mander et al, 1988). However, in the joint region, where rectangular members meet, it might be more practical to use a rectangular casing. The relative effectiveness of the jacket geometry on the behaviour of the joint region is compared with existing studies on their effectiveness on the behaviour of plastic hinge regions of columns alone.

The specifics of the steel jacketing used in each retrofitting case conformed to modern Canadian Code design (CAN3-A23.3-M84). Some of the parameters considered included: the overall size dimensions of the steel section, the diameter to thickness ratio (or width to thickness ratio) of the steel section, the use of grout or concrete, the appropriate length of reinforcement (hinge area vs. entire length of section), and the need for steel section continuity through a joint.

The tests that were performed as a part of this study reflect only the simplest of connections, that is to say a simple external beam-to-column connection. As far as standard design and construction procedures are concerned, this is a dramatic oversimplification. Under normal circumstances, a reinforced concrete structure will have joints that consist of more than just one beam framing into a column. As an example, a typical interior joint will consist of four beams as well as a slab framing into the column. Retrofitting a joint of this type with a steel jacket would be a much more complicated proposition. However, this research was intended as a preliminary study to determine the retrofit behaviour of incorporating steel jackets at a connection. For this purpose, a simple beam-column connection should be sufficient to determine whether or not the retrofit method proposed is worthy of further research in more realistic situations.

CHAPTER 2

PREVIOUS RESEARCH

2.1 INTRODUCTION - REINFORCED CONCRETE COLUMNS

It is well known that the design of reinforced concrete columns for seismic loads requires attention to detailing in the plastic hinge zone so that adequate strength and ductility can be achieved. Sufficient transverse reinforcement must be provided for this purpose. Closely spaced transverse reinforcement confines the concrete core of the column, and not only substantially increases the compressive strength of the column, but also enhances the ductility. Design codes require a certain level of ductility in columns to ensure that brittle failures do not occur, and to provide a certain level of post-yielding strength. It is important to note that high axial loads adversely affect the strength and ductility of columns. In all cases, the codes require that the shear strength of columns be sufficient to ensure that the flexural strength is reached in advance of a brittle shear failure, which is strictly forbidden.

2.2 INCREASING DUCTILITY OF REINFORCED CONCRETE COLUMNS

The usual method of providing confinement in reinforced concrete columns is to provide transverse reinforcing steel in the form of hoops or spirals, depending on the geometry of the column. To increase the effectiveness of the confinement, and hence increase the strength and ductility levels, a number of options can be considered when using hoops or spirals as transverse reinforcement. An increase in the transverse steel ratio, either by increasing the size of the transverse steel, or by decreasing the spacing or pitch of the reinforcement, was found to increase the compressive strength and ductility of columns (Mander et al, 1988).

It has also been shown that circular hoops are more effective for confinement than square or rectangular hoops, as the confinement stress is equally distributed along the entire perimeter of the spiral, and not heavily concentrated at corners, as is the case in non-circular lateral reinforcement (Mander et al, 1988). The problem with circular structural members, and hence circular reinforcement patterns, is that they are much more difficult to construct, particularly in joint regions. Economic considerations usually dictate the use of rectangular members in construction above circular members.

Although an increase of the steel ratio is generally used to enhance the strength and ductility of columns, other methods have been either tested or adopted to achieve similar results. Increasing the yield strength of the steel also increases the strength and ductility of the columns (Muguruma, 1984). The longitudinal reinforcement geometry can also be adjusted to assist in the confinement of the concrete core (Sheikh and Uzumeri, 1982). Spreading the steel around the perimeter of the core by using a larger number of smaller bars is more effective in providing confinement than having only a few large bars. Other attempts at either providing or improving strength and ductility have included the use of steel fiber reinforced concrete (Ganesan and Murthy, 1990), welded-wire fabric (Razvi and Saatcioglu, 1989) and prestressed bolts (Cheong and Perry, 1991). Limited success has been achieved in improving strength and ductility by increasing the concrete strength (Shin et al, 1989).

2.3 CONFINEMENT OF REINFORCED CONCRETE COLUMNS BY STEEL JACKET

An alternative method of providing confinement for concrete in columns is to encase the concrete core within a hollow steel member. In new construction this is typically achieved by filling hollow steel sections with concrete. When reinforced concrete columns need to be retrofitted, they can be enclosed in a steel jacket, and the space between the two elements filled with grout. In both cases, the external steel completely encloses the entire concrete core, and effectively confines all the concrete, inclusive of the cover concrete, thus assisting in reducing bond failures in columns. Confining the concrete core with a steel jacket also prevents the steel from buckling, and improves the behaviour of the column to the extent that the strength of the confined column is greater than the combined separate strengths of the steel and concrete (Tidy, 1988). It is important that the steel jacketing provide confinement only, and should not actually carry any of the axial load. Applying an axial load to the steel jacket as well as to the concrete core, and designing the retrofit with a continuous steel jacket, would alter the characteristics of the structure, resulting in lower ductility ratios and higher moment capacities at the critical sections (Priestley et al, 1990 ; Priestley and Park, 1985). It is absolutely essential to maintain a shear capacity that is higher than the corresponding moment capacity to ensure ductile failure modes.

2.4 PROBLEMS IN EXISTING STRUCTURES

Before the early 1970's, the seismic design provisions were significantly different than they are today. The requirements for seismic design were less stringent, and reflected the amount of knowledge available at that time regarding the behaviour of reinforced concrete under cyclic loading. In terms of present standards, earlier designs of reinforced concrete columns commonly exhibited poor detailing of transverse reinforcement, either in terms of the use of ties, anchorage of ties and hoops, or simply excessive spacing of transverse reinforcement (Mitchell, 1991). Other problems included inadequate lapping of longitudinal reinforcement in hinge areas (Priestley and Park, 1984) or, in the case of short columns, designs in which the flexural strength exceeded the shear strength, often resulting in brittle failure modes (Chai et al, 1991). As a result of experience gained from recent earthquakes, and in particular the San Fernando earthquake of 1971, an extensive retrofit program was undertaken in California to upgrade existing structures to modern earthquake code requirements. Although all new structures are, of course, built to modern standards, the largest percentage of existing structures in the different seismic regions of the U.S. and elsewhere (including British Columbia), do not meet modern specifications, and may need to be upgraded.

2.5 ADVANTAGES OF THE STEEL JACKETING METHOD

A favoured method of retrofitting reinforced concrete columns is to either partially or fully encase the member in a steel jacket. In the case of short columns, the entire column length is typically encased, as shorter reinforced concrete columns are particularly susceptible to brittle shear failure (Chai et al, 1991). In columns with higher slenderness ratios, which are the ones usually found in structures, only the plastic hinge region need be confined in a steel jacket. The steel jacket is expected to provide confinement only, and enhances the ductility of the column, so as to reliably provide a load carrying mechanism after general yielding of the reinforcement. Columns which have to support large axial loads need jacketing along the entire length to enhance the compressive strength as well. In this case, the steel jacketing is not expected to actually carry any of the axial load, a function that is left to the added concrete alone.

Numerous studies dealing with the effectiveness of different methods of confinement of reinforced concrete columns have either been completed or are presently ongoing. The investigation presented here focuses on the retrofitting of reinforced concrete members at frame connections. In particular, attention was paid to the behaviour of connections between beams and columns that have been retrofitted after suffering some initial damage. The object of this study was to assess the strength and ductility improvement in the behaviour of a retrofitted beam-to-column joint.

As is the case with hoops and spirals, in reinforced concrete members and columns retrofitted with steel jackets it is expected that a beam-to-column joint that has been retrofitted with a circular jacket will exhibit more desirable characteristics than a joint retrofitted with rectangular or octagonal jackets. Columns retrofitted with a circular jacket were found to exhibit greater ductility due to confinement, a larger overall strength, and greater bond strength between the concrete and both the reinforcement steel and the steel jacket (Morishita et al, 1988).

2.6 BEHAVIOUR OF BEAM-TO-COLUMN JOINT

During extreme cyclic loading, which is common during a significant seismic event, the forces within a beam-to-column connection are relatively large and often approach or exceed the load-carrying capacity of the connection. The connection is considered as part of the column, and ideally should behave elastically, without the development of yield hinges within the joint, in order to avoid failure of the shear panel, and subsequent anchorage failure of the reinforcement (Otani, 1991). Under such conditions, the energy absorption capacity of adjacent plastic hinges is thus maintained without the shear or anchorage failure of the joint core (Kaku and Asakusa, 1991). To avoid collapse of the structure, plastic hinges are designed to occur in the beams following the rules of standard practice: strong column/weak beam, strong joint/weak element, strong shear/weak moment (Bolong and Yuzhou, 1991). Modern design codes address these concepts by specifying a ratio of strengths between beam and column. For instance, the ACI-ASCE Committee 352 on the design of reinforced concrete connections specifies that the ratio of flexural strengths between column and beam should be at least 1.4, so as to avoid the development of a flexural hinge in the column.

Over the last 20 years, a fair amount of research has been done on the behaviour of beam-to-column connections, which is reflected in the recommendations by the ACI-ASCE Committee 352 for the design of beam-to-column connections. Before these recommendations, the joint was considered to behave much like a deep beam, and designed accordingly (Ehsani and Wight, 1990). Today we know that this is not the case, that the joint behaves in a manner that is in some ways different and in other ways similar to a deep beam. During seismic loading, the force transfer between the beam and the column within a joint occurs in the form of a truss action, consisting of a concrete strut and a tension tie formed by the transverse reinforcement steel within the joint, which is activated by bond stress and anchorage. Recent experimental results suggest that the concrete strut transfers most of the force, and hence only enough transverse steel to provide confinement is required within the joint (Leon, 1990).

Joint performance appears to be a function of the joint shear stress and confinement level (Alameddine and Ehsani, 1991). These authors showed that high joint shear stresses reduce the energy absorption capacity and cause a rapid loss of load-carrying capacity of the joint. The primary role of transverse joint reinforcement is one of confinement (Ehsani and Wight, 1990); joint deterioration due to high joint shear stress can be prevented by providing sufficient anchorage of the longitudinal steel (Leon, 1990). Other recommendations for the improvement of joint behaviour are the avoidance of large plastic deformations within the joint, the limitation of concentration of damage to prescribed sections, and the avoidance of brittle failures.

2.7 DESCRIPTION OF RETROFIT STUDY

The object of the beam-to-column connection retrofit/repair program under study was to improve the behaviour of the connection region during cyclic lateral loading. The problems associated with joint shear failure (yield hinges within the joint, and insufficient confinement and anchorage) were to be solved by the use of steel jackets. The intention was to develop plastic hinges within the beam adjacent to the connection, and to increase the ductility of the system at the same time. This would result in an improvement of the seismic behaviour of the overall frame. The one condition that had to be avoided when retrofitting by means of a steel jacket was the potential development of a plastic hinge in the beam outside of the retrofitted region. A plastic hinge outside of the retrofitted region would mean that the original problem of insufficient ductility had been moved elsewhere, and no real advantage would have been gained. A plastic hinge that has been moved along the beam away from the joint would also mean higher shear stresses in the unretrofitted section of beam, resulting in a higher risk of brittle shear failure.

As noted earlier, it is a well-established fact that circular and elliptical steel jackets provide superior performance in retrofitted columns compared to jackets of square, rectangular or other geometrical configurations. However, the expected advantages of incorporating such jackets in the retrofit of beam-to-column connections have not been adequately studied. The forces that act within a joint are somewhat more complicated than those developed in the isolated column retrofit case. It was the intention to investigate the relative merits of both the circular and rectangular jackets surrounding a beam-to-column connection. Whereas the circular jacket was expected to behave in a superior fashion, the rectangular jacket was simpler to apply and might provide sufficient improvement in the behaviour of the connection to favour its use in practice.

CHAPTER 3

TEST SPECIMEN DESIGN

3.1 DESIGN PHILOSOPHY AND CODES

The aim of this research project was to observe the expected joint damage in an older code-designed reinforced concrete frame building under seismic load conditions and to investigate means of retrofitting the joint to improve its seismic behaviour. In order that the study be viable, practically, a realistic building was designed for the investigation and a joint sub-assembly of this structure was considered for the detailed study.

A two-storey office structure situated in Vancouver, BC was first designed in accordance with all of the requirements of the 1970 National Building Code of Canada, and primarily satisfied the CAN3-A23.3-M66 (1966) code for reinforced concrete structures. The reinforced concrete beam-to-column joint specimens were then taken from this office structure, and their designs were modified slightly.

The design of the test specimens was controlled by the available materials and the testing equipment limitations. The existence of a set of reusable forms from a previous project dictated the size of the specimens (Katzensteiner, 1992). The specimens were accordingly designed as part of a frame whose overall dimensions reflected the forms; this resulted in an approximately half-scale beam-to-column joint model.

3.2 SPECIMEN DESIGN CONSIDERATIONS

In deciding on the lengths of the beam and columns in the specimens it should be noted that the intention of the experiment was to observe the behaviour of the joint region under seismic loading by applying a cyclic load at the end of the beam. The columns of the test specimens were to be cut off at the presumed inflection points, and in the testing apparatus would be replaced with hinges, resulting in zero moment at the ends of the columns. However, because of the existence of the testing apparatus from a previous experiment (Kuan, 1992), it was decided to match the length of the columns to the available space in the test apparatus. Consequently, both the upper and the lower columns were made to be of equal length, thus simplifying the testing process somewhat.

A decision was made to alter the design of the joint from that specified in the 1966 reinforced concrete code. All transverse reinforcement was omitted from the joint region. The main reason for taking this step was that it conformed to common practice due to ambiguities in the interpretation of that edition of the code. Whereas the joint region should be considered as part of the column, with specific transverse reinforcement requirements, this was never explicitly stated in that issue of the code, as opposed to the actual modern code. Designers at that time typically did not extend the transverse column reinforcement into the joint for a variety of reasons. Foremost among them, is ease of construction, since transverse reinforcement within a beam-to-column joint causes considerable difficulty in reinforcement placement; it also increases the possibility of the formation of honeycomb voids due to the congestion of steel within the joint area. Joints with three or more framing beams were considered "confined", due to the action of the beams, and joint reinforcement was deemed unnecessary. The resulting design of the test specimens is outlined in Appendix A.

3.3 UNRETROFITTED TEST SPECIMENS

One of the specific aims of this research project was addressing the deficiencies in beam-to-column joints caused by neglect, construction mistakes or misinterpretation of the codes. The four identical test specimens (fig. 3-1) were fabricated to meet the program goals. The positive moment capacity of the beam was designed for 16.27 kNm; to resist this moment, four 10mm bars were needed at the ends of the beam. Overall, the steel requirements were: two 10mm bars for both positive and negative flexure elsewhere in the beams, and three 10mm bars for flexural reinforcement on either side of the columns; transverse reinforcement consisted of 10mm ties at 70mm spacing in both the beams and the columns.

3.4 RETROFIT DESIGN

In the first phase of the experimental program, two specimens were loaded cyclically such that the expected damage to the joint region occurred. One damaged specimen and one undamaged specimen were subsequently retrofitted with a circular steel jacket, while the other pair of specimens were retrofitted with square steel jackets. The dimensions of the jackets were similar in both cases.

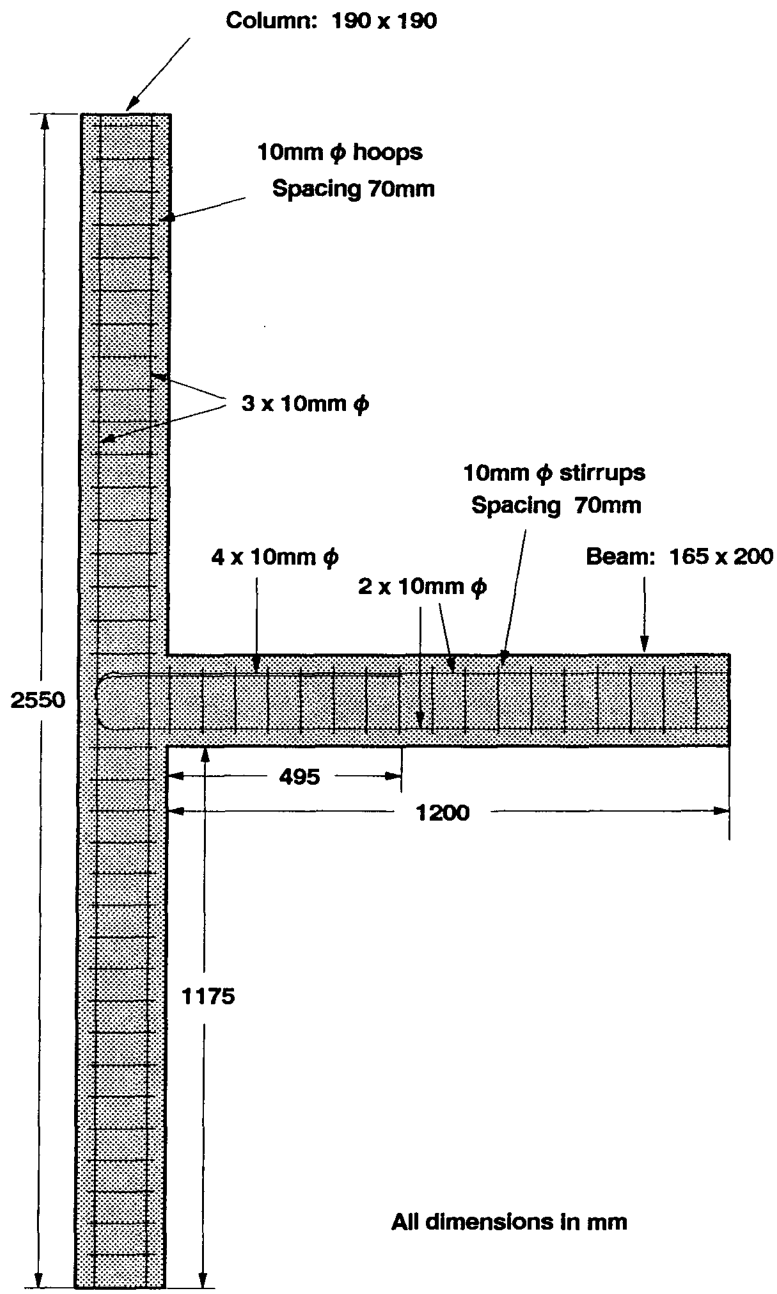


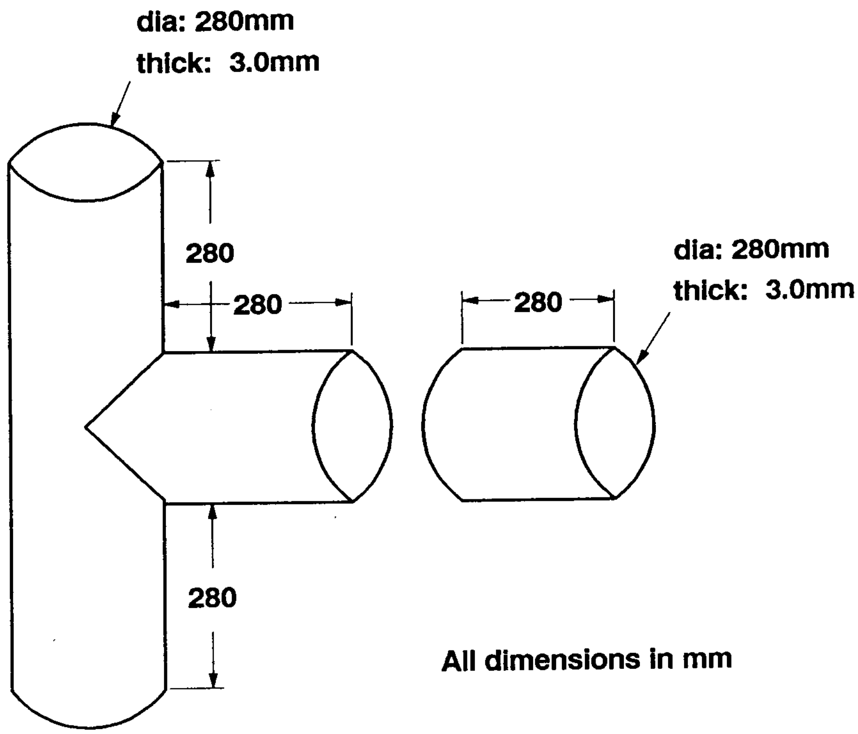
Figure 3-1 Specimen Design

The steel jackets were intended to provide confinement and additional ductility to the joints, without increasing the moment capacity of the specimens. Modern design codes for new construction require an increased amount of transverse reinforcement near the joint regions to increase ductility. The steel jackets were to provide the desired ductility by substituting the reinforcement steel that was missing in the original design. Under modern design codes, the spacing of the transverse reinforcement should have been 35mm in both the beams and the columns, provided the same 10mm bars were used in these elements. The same transverse reinforcement spacing would have been required in the joints, since they were not framed on all four sides by beams. To make up for the missing steel, the thickness of steel required for the joint governed, as there was no existing transverse steel within the joint. The result was a steel jacket with a thickness of 2.86 mm (0.110"). The lengths of the retrofit were defined as being equal to the member depth (d) along the column and twice the member depth (2*d) along the beam, measured from the beam-to-column interface. A gap was left halfway along the length of the beam jacket, to create a flexural hinge at that point.

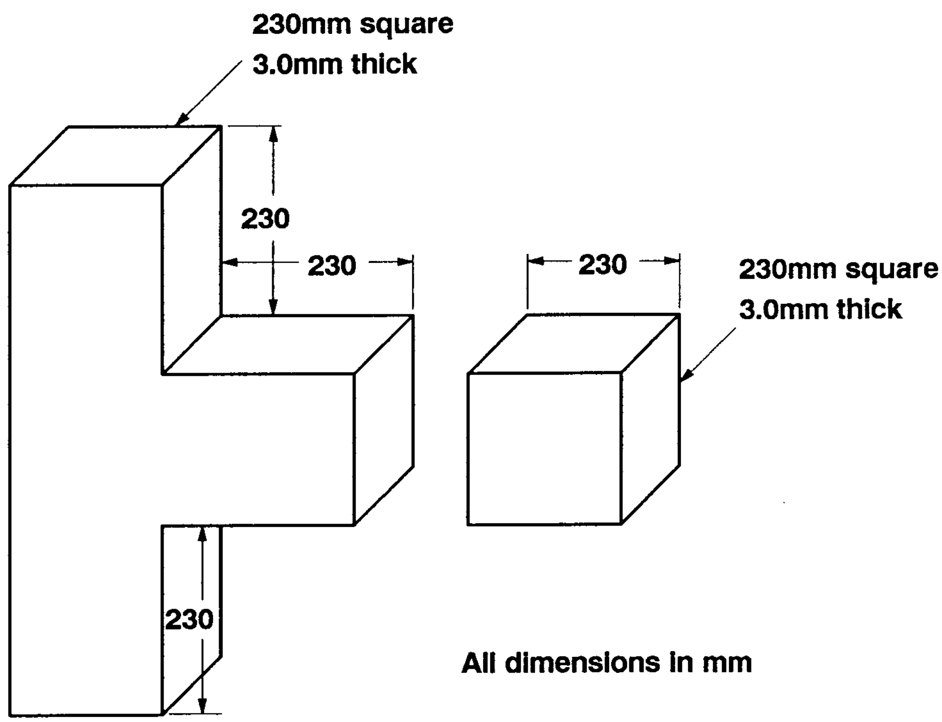
Preliminary tests of retrofitted beams (Ross, et al, 1992) show that the flexural strength of retrofitted beams can be increased approximately eight-fold. A retrofit that consists of an unbroken tube length of 2*d along the beam would force the flexural hinge to the end of the retrofit, and no gain in ductility would result. This inadequate retrofit would merely increase the overall strength of the beam, without increasing its ductility; furthermore, it would also increase the risk of shear failure. The gap in the retrofit beam jacket was therefore introduced to assure that a flexural hinge would occur within the retrofitted area. The joint area would still be confined by the steel tube, and ductility would be added to the joint region. There would still be an increase in the overall moment resistance; in the present case this was expected to be about one-third.

The size of the steel jackets was kept to a minimum to reduce the amount of disruption to the structure during the retrofit process, and also to limit the increase in the capacity of the frame members beyond their original design values. An increase in the moment capacity of a section in one portion of the structure could force a failure elsewhere in the structure by increasing the loading at that point beyond design values. The dimensions for each of the retrofit strategies used in this study are summarized schematically in fig. 3-2a,b.

An original aim of the retrofit portion of this study was to compare the retrofit behaviour of both undamaged and previously damaged specimens. However, as the testing progressed, modifications in the program were deemed necessary, which resulted in differences in the retrofit details between the two sets of retrofitted specimens. For example, because of the state of the reinforced concrete of the original specimen, it was impractical to compare the results of the two specimens retrofitted with a circular jacket. With the square jacket retrofit scheme, premature failure of the beam outside of the retrofit range forced an additional length of jacket to be added to the second beam specimen. A similar premature failure in the column of the circular retrofit scheme led to the addition of extra gaps in the beam jacket, in order to assure a reduction in the moment capacity of the retrofitted joint. These changes are detailed in the chapters dealing with the retrofitted specimens.



a) Circular Steel Jacket



b) Square Steel Jacket

Figure 3-2 Retrofit Concepts

CHAPTER 4

EXPERIMENTAL APPARATUS AND PROCEDURE

4.1 INTRODUCTION

The cyclic loading of both the retrofitted and unretrofitted specimens was carried out in a similar manner. The unretrofitted specimens, however, were expected to behave poorly and, as a result, to have a much lower ductility and to fail after only a few cycles. A large testing frame in the Civil Engineering Structures Laboratory at UBC was used to apply the cyclic loading. The frame incorporated two separate actuators: one to supply the column axial load, and the other to provide the load at the end of the beam for cyclic loading. Data acquisition from load cells, linear variable differential transformers (LVDT's) for displacement measurements, and numerous strain gauges was handled by an Optilog model 200 data collection system.

4.2 TESTING APPARATUS

The testing frame loading apparatus is shown in fig. 4-1. The cyclic load at the end of the beam was applied by a double-acting 100 kip (446 kN), 24" (610mm) stroke Temposonics Actuator, and the constant axial load for the column was applied by a 100 kip

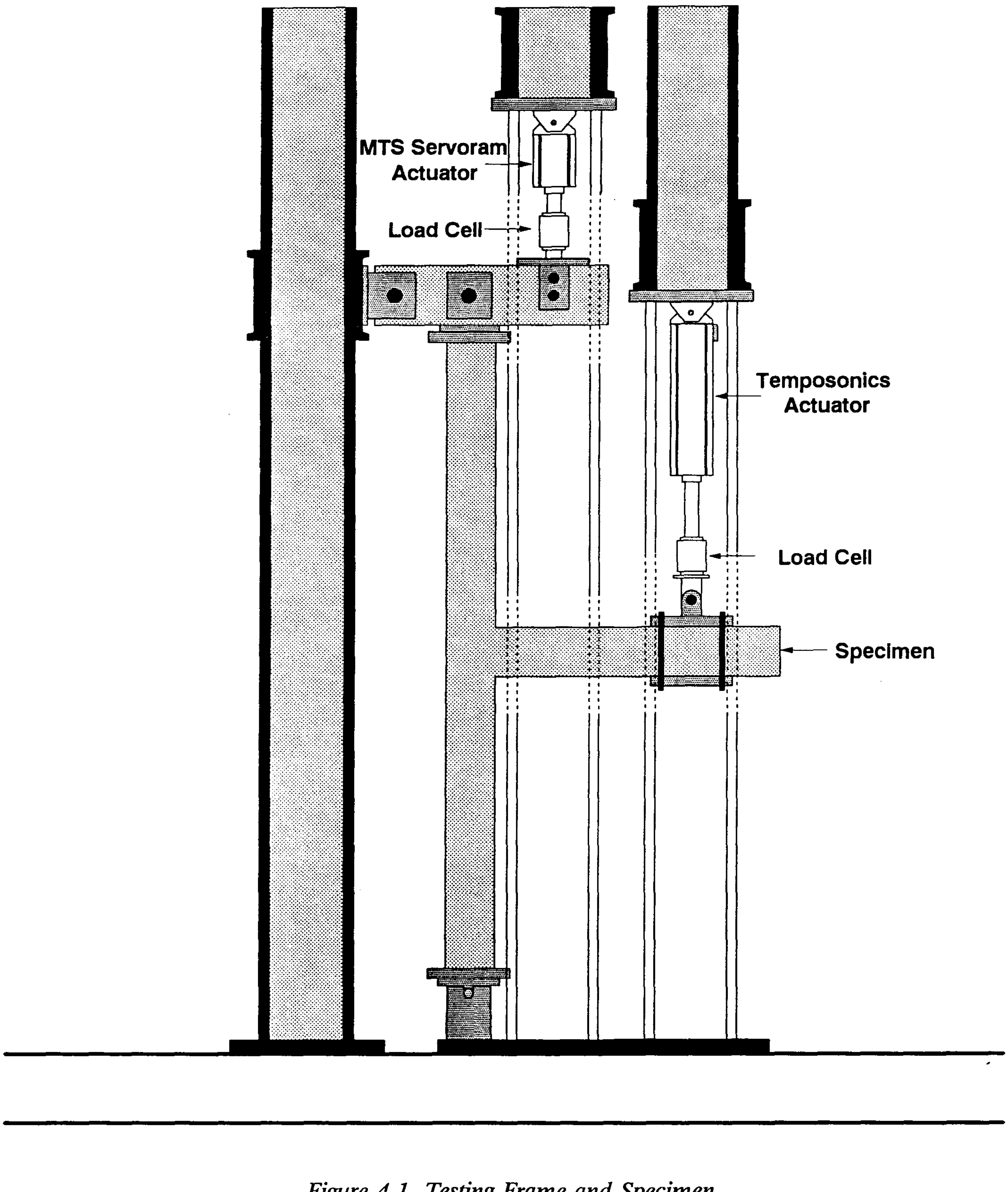


Figure 4-1 Testing Frame and Specimen

(446 kN), 6" (150mm) stroke MTS Actuator. Both actuators were controlled by an MTS servo-control system. The column actuator was set to load control, so that it provided a constant load during the testing sequence. The Temposonics actuator was operated under displacement control. It was much easier to control the cyclic load and displacement in this manner.

The applied loads were measured by 100 kip (446 kN) load cells that were located in line with the actuators. Deflections were detected with a set of LVDTs, and by the displacement transducer that is built into the Temposonics actuator. Strains on the jackets of the retrofitted specimens were measured with the use of 350-ohm CEA strain gauges. All of the loads, displacements and strains were collected and stored into a computer by the Optilog data acquisition system.

4.3 MEASUREMENTS FOR ROTATION

The LVDTs were arranged as illustrated in fig. 4-2a,b. The actual locations at which the LVDTs were placed in the unretrofitted and retrofitted tests varied, due to the differing sizes of the members (fig. 4-2). The individual measured rotations were used to separate and identify the various types of movement that make up the total rotation of the beam. The LVDTs on the beam (#4 in fig. 4-2a, #4 and #5 in fig. 4-2b) measured the total rotation of the beam. This rotation consists of the rotation caused by elastic deformation of the column, the shear deformation of the joint, and the rotation of the beam assuming a fixed joint (cantilever action). Each type of rotation was identified and analyzed separately, using the data supplied by the remaining LVDTs. The various types of rotation

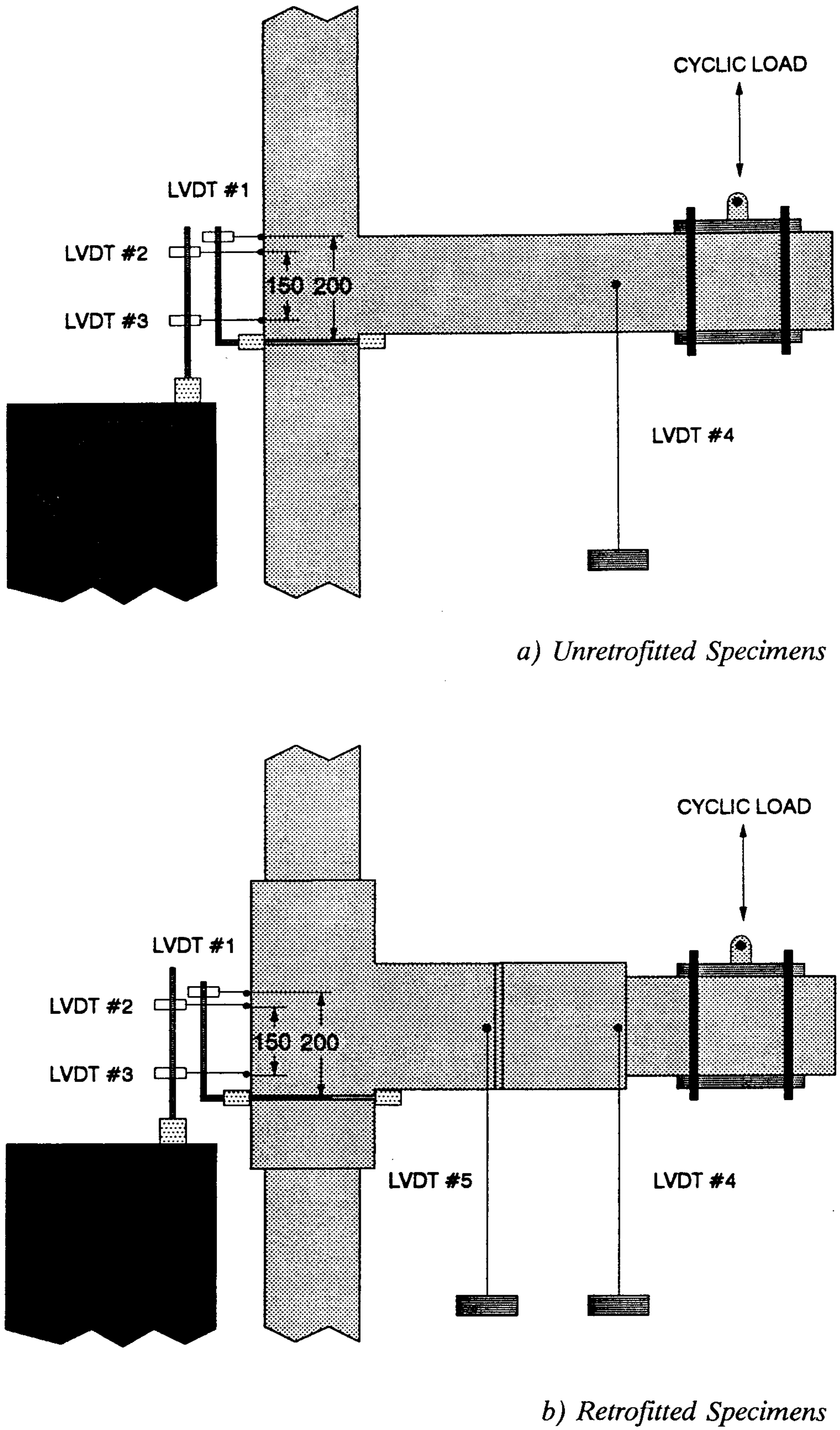


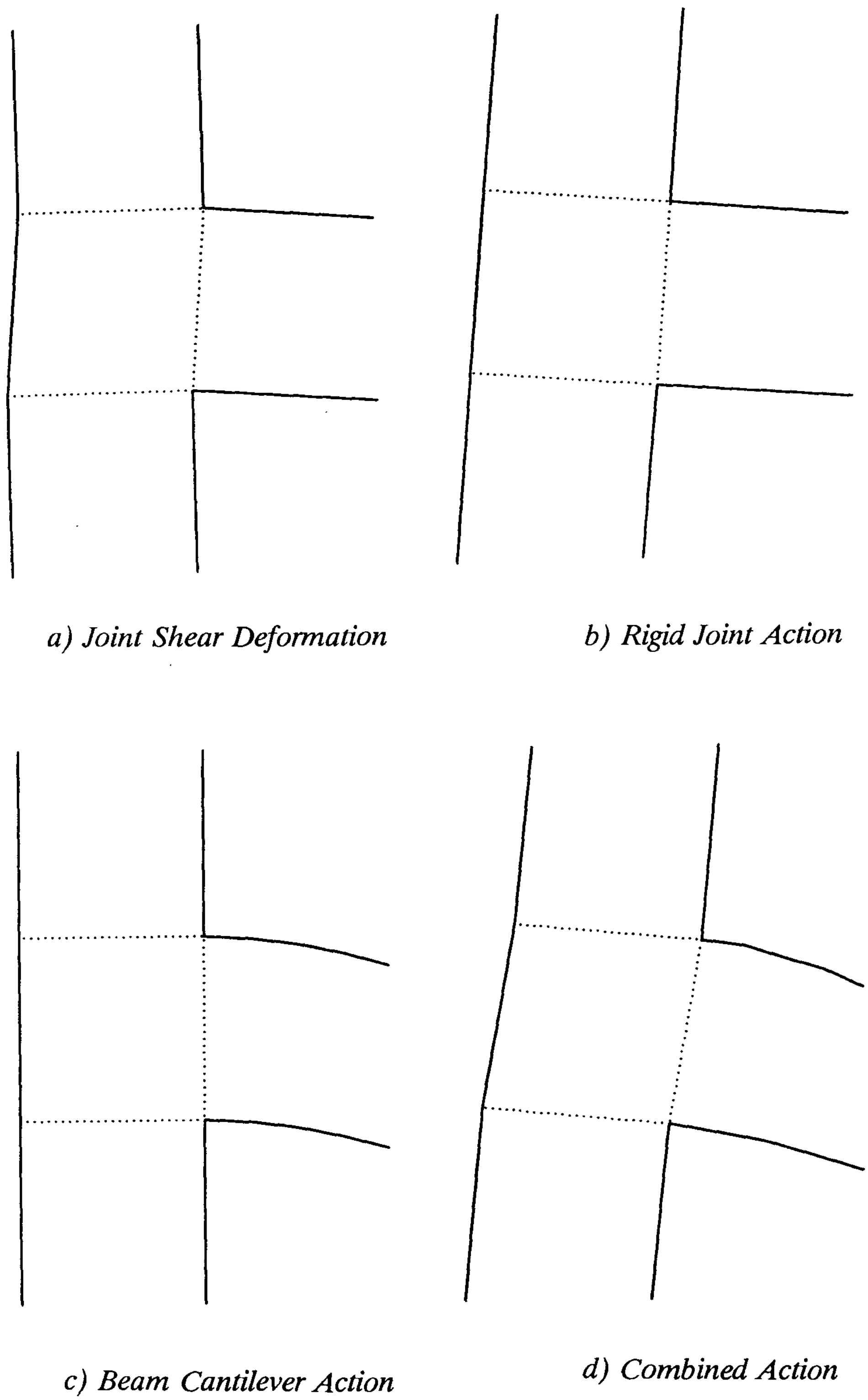
Figure 4-2 Placement of LVDTs

recorded are illustrated in Figure 4-3. The rotation caused by the shear deformation of the joint was measured by LVDT #1, and it can be seen that the rigid joint rotation, which is a result of the elasic deformation of the column, is determined by subtracting the shear deformation rotation from the rotation as measured by LVDTs #2 and #3. The cantilever rotation of the beam is then evaluated by subtracting the shear deformation and rigid rotation effects from the total deflection measured by LVDT #4.

4.4 STRAIN GAUGES

Strain gauges were used in the retrofitted tests to determine the behaviour of the steel jackets during cyclic loading, and their layout is shown in fig. 4-4. This figure illustrates the maximum number of gauges used corresponding to the tests of the retrofitted, undamaged specimens RETRO-SU and RETRO-CU. After reviewing the data from these two tests, it was decided to omit some of the gauges when testing the retrofitted, previously damaged specimens RETRO-SD and RETRO-CD; these gauges are identified in fig. 4-4. The strain gauges were concentrated within the joint area and were intended to provide an insight into the complicated stress pattern of the steel jacket in that area. Of particular interest was the amount of longitudinal, circumferential and transverse stresses developed at the same time within the steel jacket.

The intention of the steel jacket was solely to provide confinement for the concrete core. Ideally the steel jacket should only undergo circumferential or transverse stress. However, due to the complicated nature of the stresses within the joint due to confinement and concrete to steel bonding, it was anticipated that longitudinal stresses would be



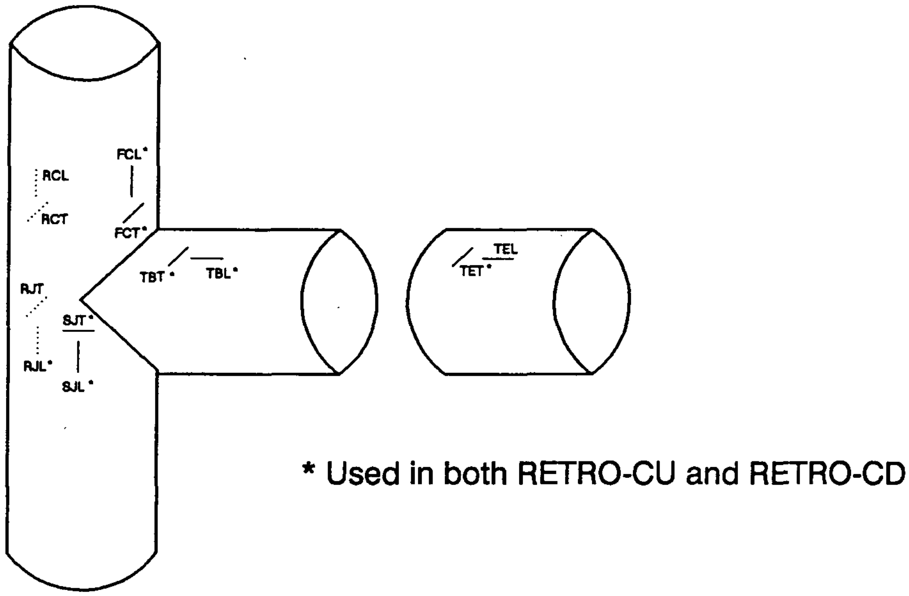
a) Joint Shear Deformation

b) Rigid Joint Action

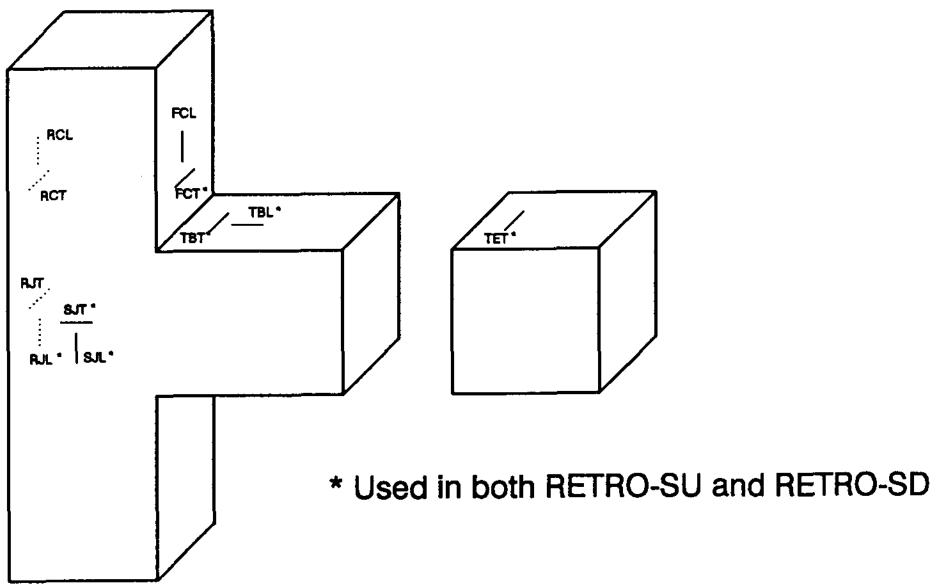
c) Beam Cantilever Action

d) Combined Action

Figure 4-3 Rotation Types Within the Joint



a) Specimens with Circular Retrofit



b) Specimens with Square Retrofit

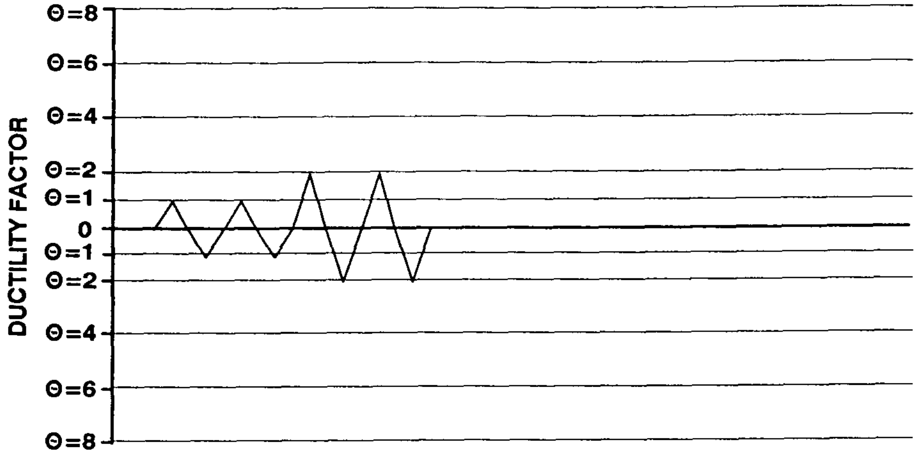
Figure 4-4 Placement of Strain Gauges

developed, resulting in an increase in the moment capacity of the specimen. The placement of a jacket gap within the retrofit section of the beam was intended to mitigate some of these undesired effects.

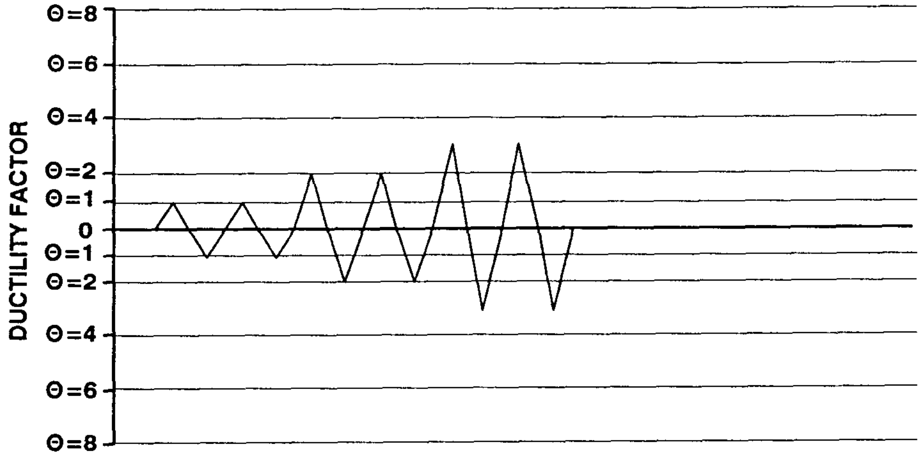
4.5 TESTING PROCEDURE

The test procedure for the unretrofitted tests was straightforward. The axial load was first applied to the column, and this load was maintained throughout the duration of the test. A moment was then applied to the joint by loading the end of the beam, in an upward (positive) direction. The load was steadily increased until the observed yield moment was reached. Due to the non-linear response of the specimen, the definition of the yield moment depended somewhat on judgment. An analog plot of the Temposonic actuator load versus displacement was used to decide when the yield moment was reached. A ductility factor of $\Theta=1$ was associated with this moment. This same moment was then applied in the opposite direction (negative), and then the full cycle was repeated a second time. Two more complete cycles were then applied, but at double the displacement. The cyclic loading pattern of the unretrofitted joints is illustrated in fig. 4-5a. For the unretrofitted tests, four cycles of loading caused enough distress to the joint that it could be considered damaged, but repairable. The unretrofitted tests were not intended to completely destroy the joints, since one of the original aims of this study was to compare the behaviour of damaged and undamaged joints when retrofitted with steel jackets.

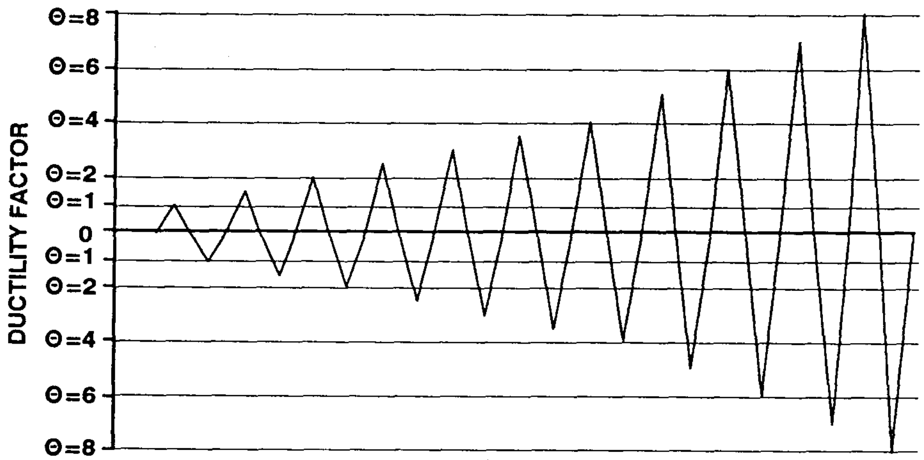
The loading pattern of the retrofitted joints was similar to that followed with the unretrofitted joints, as shown in fig. 4-5b,c. The retrofitted specimen RETRO-SU followed



a) Unretrofitted Specimens RCBC1, RCBC2



b) Retrofitted Specimen RETRO-SU



c) Retrofitted Specimens RETRO-CU, RETRO-SD, RETRO-CD

Figure 4-5 Testing Sequence

the pattern established for unretrofitted specimens RCBC1 and RCBC2, save that each cycle began with the load applied in the negative direction. For the retrofitted specimens RETRO-CU, RETRO-SD and RETRO-CD, after $\Theta=1$, each subsequent cycle was incremented by half of one full ductility factor. After $\Theta=4$, each cycle was incremented by one full ductility factor. The joints were loaded until failure, or to the stroke limit of the actuator.

The unretrofitted specimens RCBC1 and RCBC2 were tested first, and once the retrofit had been completed, the retrofitted specimens RETRO-SU, RETRO-CU, RETRO-SD and RETRO-CD were tested.

CHAPTER 5

UNRETROFITTED TEST RESULTS

5.1 INTRODUCTION

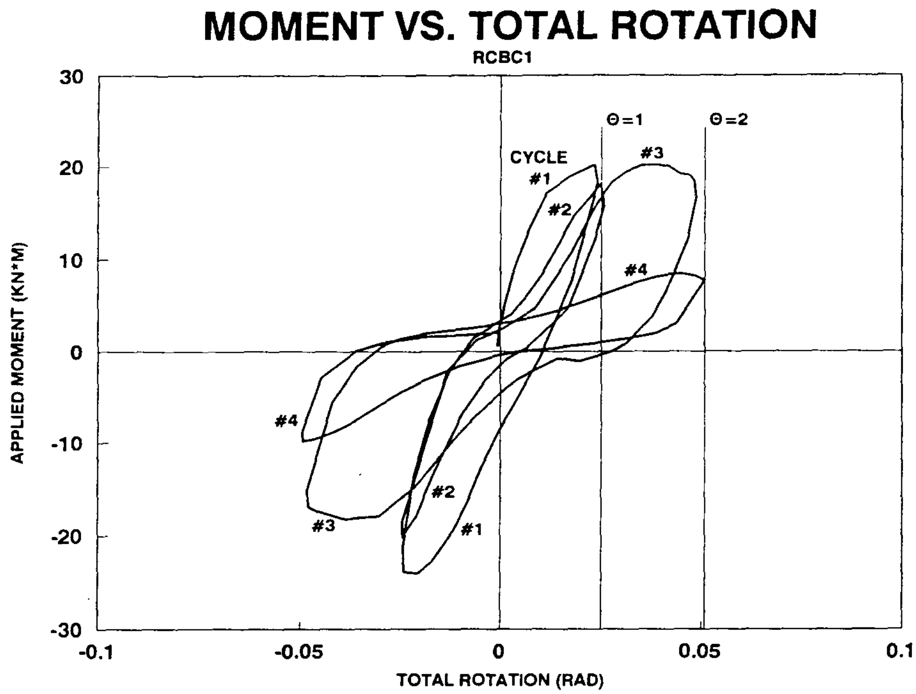
The tests on the unretrofitted reinforced concrete specimens yielded results consistent with previous studies, i.e. lack of shear strength and ductility in the joint area. The specimens performed inadequately by today's standards, as they did not attain an acceptable ductility ratio, nor did they even reach their theoretical elastic limits under certain conditions, as will be shown. Both RCBC1 and RCBC2 were tested under cyclic loading. Each loading cycle began with the beam being pulled upwards in positive loading, such that the bottom layer of flexural steel was stressed in tension. Each cycle ended with negative loading producing a deflection which was equal and opposite to the deflection imposed during the positive loading, so that the top layer of steel was in tension. The deflections in the first and second cycles corresponded approximately to the yield deformation of the lower layer of steel. The deflection of each successive pair of cycles was increased to correspond to the next whole number multiple of this initial yield deformation, until failure took place. All recorded data for the unretrofitted specimens are presented in Appendix C.

5.2 OBSERVED BEHAVIOUR OF SPECIMENS DURING TEST

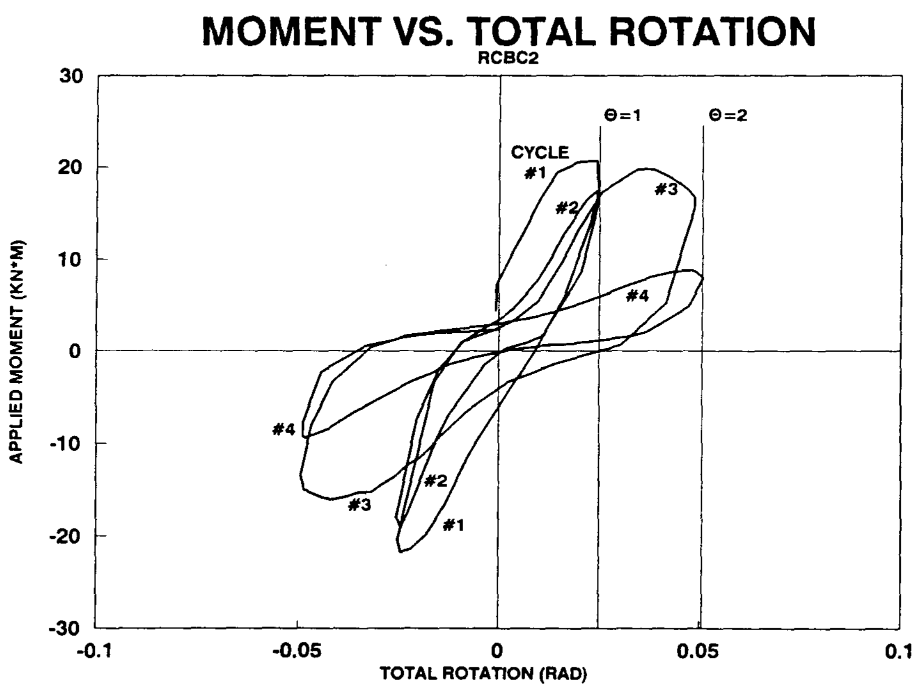
The hysteresis loops obtained for both RCBC1 and RCBC2 (fig. 5-1a,b) show that a rapid degradation in stiffness and strength occurred during the first four cycles. The displacement during the fourth cycle corresponded to twice the yield displacement. After the fourth cycle the damage within the joint was so extensive that only a fraction of the maximum strength and stiffness still remained, and failure of the specimen had obviously taken place. The strength loss during the final cycle was of the order of 60%. The hysteresis loop exhibited the characteristic pinching effect that is prevalent in reinforced concrete members tested under cyclic loading.

There was absolutely no indication of a failure of any type having occurred within either the beam or the column. Flexure cracks formed within the column and the beam at regular intervals; these cracks appeared as expected according to the design. There was plenty of evidence, however, to suggest that all the major damage had occurred within the joint region. As shown in the photographs (fig. 5-2a,b), both specimens developed a large amount of shear cracking within the joint region. These cracks opened and closed in synchronization with the cyclic load that was applied to the end of the beam. After the displacement passed the yield point during negative loading, the shear cracks within the joint no longer closed completely. A large amount of the concrete cover spalled off the rear face of the column within the joint area. Some concrete crushing also took place within the hinge area at the interface of the column and the beam.

At best, both specimens were only able to develop a rotational ductility of $\Theta=2$. A noticeable reduction in both strength and stiffness took place on the negative loading portion



a) RCBC1



b) RCBC2

Figure 5-1 Hysteresis Loops: Unretrofitted Specimens

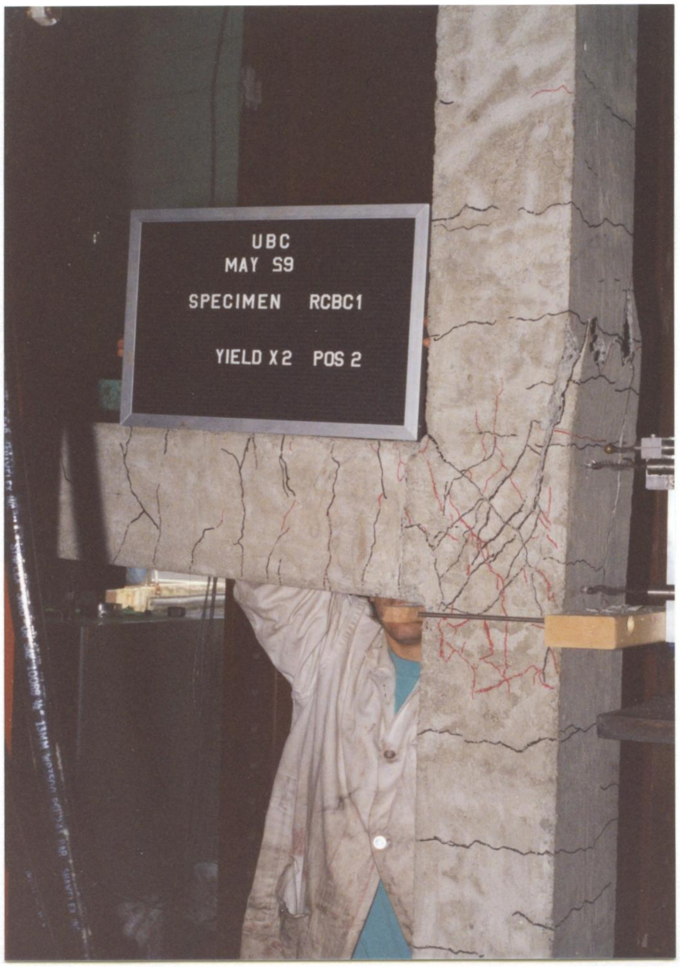
a) RCBC1

b) RCBC2

Figure 5-2 Joint Damage in Unretrofitted Specimens of the 3rd cycle (the first at $\Theta=2$), and this trend continued into the 4th cycle, where a significant strength and stiffness loss took place. Failure of both specimens was declared in the fourth cycle at a rotational ductility of $\Theta=2$. This indicated that at this point the joint capacity had fallen to less than 80% of its original strength.

5.3 FAILURE MECHANISMS WITHIN THE JOINT AREA

The failure mechanism within the joint area was complex. At the point when failure of the specimen was considered to have taken place, the size of the cracks in the joint were visibly large (fig. 5-2), particularly during the negative load cycle. The top layer of longitudinal reinforcement consisted of 4 bars in the part of the beam adjoining the column, as opposed to only 2 bars in the bottom layer. The larger amount of steel in the top layer prevented hinging during the positive cycle, which in turn pushed the damage into the joint region. However, the reinforcement in the bottom layer of steel actually attained the yield stress, and a plastic hinge formed in the beam near the interface of the beam and the column (fig. 5-3a). During the negative portion of the cycle, a different type of failure occurred, as the stress in the top layer of steel never reached yield stress. The attainment of maximum moment was limited by the failure of the bond between the longitudinal reinforcement bars and the surrounding concrete in the joint area, as well as by the large amount of straining in the cracked concrete within the joint area (fig. 5-3b). Evidence of bond failure was indicated by the spalling of the cover concrete, particularly on the rear face of the joint along the column (fig. 5-3c). This suggests that the hooked ends of the longitudinal reinforcement moved outwards to split the cover and lose its bond with the



a) Hinge in Joint

b) Shear Cracking in Joint Area

Figure 5-3 Failure of Unretrofitted Specimens

c) Spalling on Rear Face of Column

Figure 5-3 Failure of Unretrofitted Specimens interior concrete.

On removal of the concrete rubble after the tests, a large amount of loose concrete was found in zones adjacent to the longitudinal bars, particularly those of the top layer of steel. After the bond between the top layer of steel and the core concrete was lost, the top layer of longitudinal steel in the beam was free to move back and forth over the concrete core, resulting in further damage. The beam effectively pivoted about a point near the lower layer of steel, which had not lost its bond. It is interesting to note that the concrete core of the interior of the joint was virtually free of visible cracks.

5.4 MATERIAL PROPERTIES

The steel used in the testing program was obtained from the same batch of reinforcing bars. A number of tensile coupon tests were carried out and indicated that the tensile yield strength of the steel was approximately 566 MPa (appendix B); the ultimate strength of the steel was approximately 800 MPa. The stress-strain curves did not feature a marked yield plateau and the standard 0.2% offset strain was therefore used to define the tensile yield stress of the reinforcement steel. It should be noted, however, that this approach only provides a nominal yield value for use in design; it does not define a yield point in the true sense.

Due to the lack of a defineable yield plateau, it was difficult to pinpoint exactly the onset of yielding during the cyclic loading tests. As a result, a certain amount of judgment was used during the tests to decide when this "yield point" was reached. Clearly, this led to opportunities for discrepencies in determining the commencement of yield for RCBC1 and

RCBC2, although every attempt was made to be consistent in assigning their respective yield points.

The compressive strength of the concrete was determined from uniaxial tests of standard 30 cm. cylinders and was found to be 26.3 MPa (Appendix B). All the test material strengths lie within the commonly used range of values.

5.5 COMPARISON WITH MODERN SEISMIC DESIGN

When comparing the hysteresis loops of the two unretrofitted specimens, RCBC1 and RCBC2, with loops obtained from specimens that are designed according to current standards, the shortcomings of the two tested specimens become evident. Figs. 5-1a,b show the hysteresis curves for RCBC1 and RCBC2, whereas fig. 5-4 shows a loop obtained from a specimen of similar configuration whose reinforcement design conformed to current seismic standards (Filiatrault, 1992). The specimen used to produce the hysteresis loop shown in fig. 5-4 was full size, and designed according to the most recent Canadian Standard, which included proper detailing for ductility within the beam-to-column joint area. Neither RCBC1 nor RCBC2 attained a rotational ductility greater than $\Theta=2$, whereas the specimen tested by Filiatrault attained a ductility ratio of about $\Theta=3$ without loss of strength.

5.6 MEASURED DATA: RCBC1 AND RCBC2

Instrumentation was positioned to capture the major expected deformations of the specimens. As noted in section 4.3, LVDT's were placed to determine the contributions of each of the deformation types to the total rotation of the joint within the specimen.

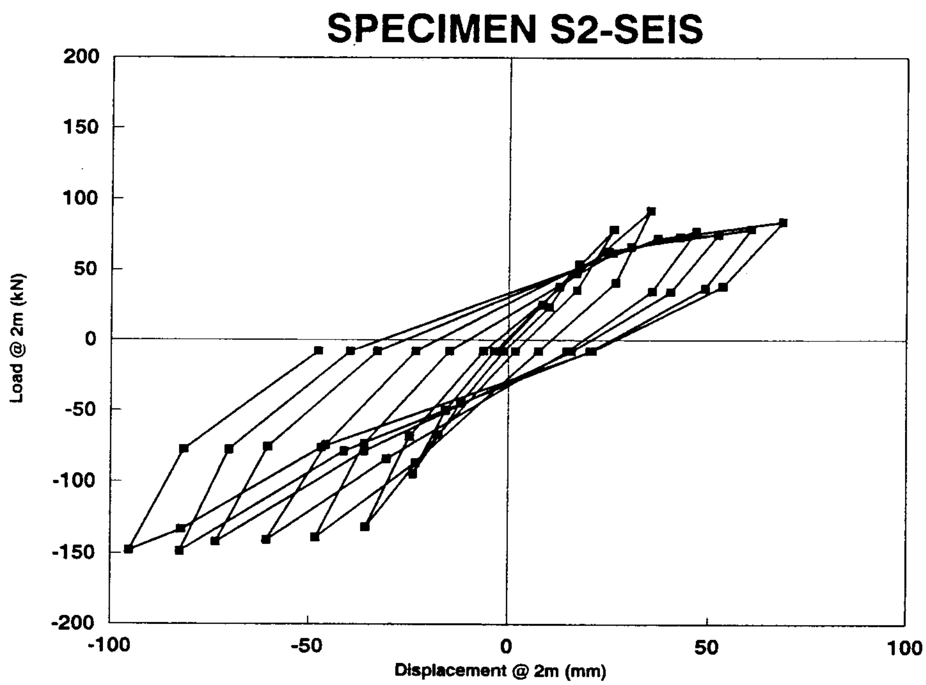
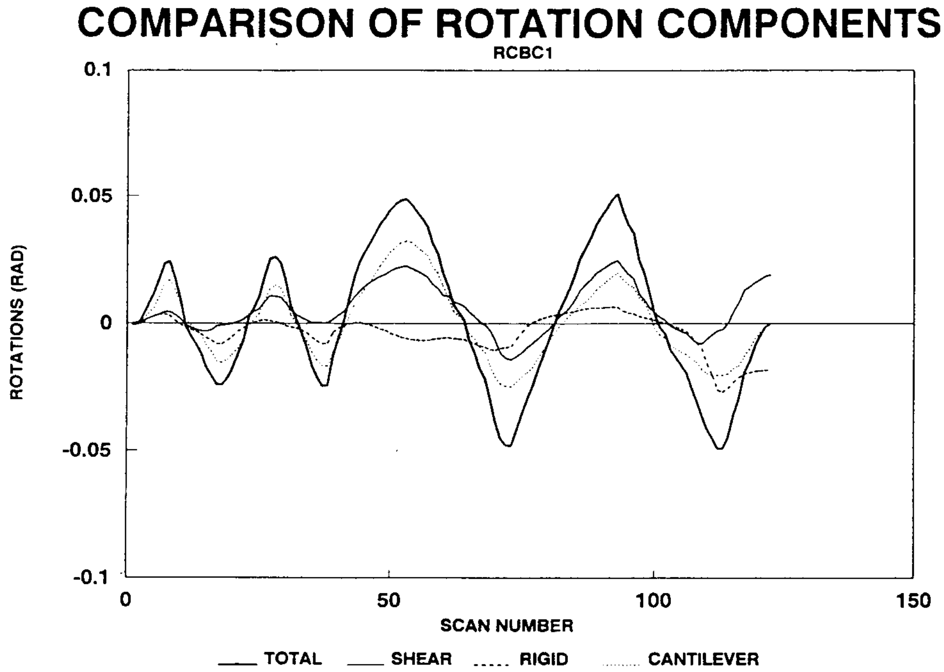


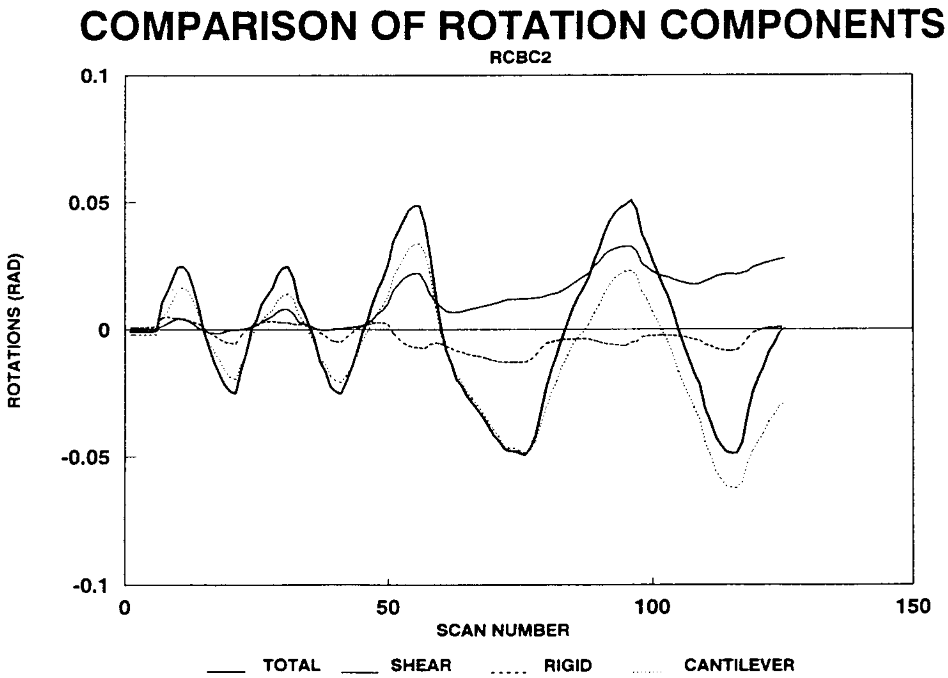
Figure 5-4 Hysteresis Loop of a Specimen of Modern Design

As a general rule, the deformation of the column, which produced the "rigid" rotation of the joint area, made the smallest contribution to the total rotation (figs. 5-5a,b), and was primarily dependent on the bending stiffness of the columns. The columns were designed to withstand much higher axial loads and moments than those applied. The axial loads applied in the tests represented the dead load of the proposed structure and a certain proportion of the live load, and were relatively low. The design of the columns was also governed by the prescribed minimum steel ratio, and these members were thus able to withstand a moment much higher than the beam yield moment (figs. 5-5a,b). The shear deformation of the joint area initially contributed only a small percentage of the total deformation, but once cracking was initiated within the joint the contribution of the shear deformation increased significantly. During negative loading, when the top layer of the beam longitudinal steel was in tension, the shear deformation component of the total rotation was the greatest. During positive loading, the shear contribution was small, as only a few tension cracks developed within the joint. The greatest deformation component during positive loading was due to the elastic cantilever deflection of the beam, and eventually the rotation of the plastic hinge that developed at the interface of the beam and column.

During negative loading, no flexural plastic hinge formed either within the beam or the joint. The force that was applied to the free end of the beam, and thus the moment at the beam-to-column interface, never attained the theoretical capacity of the reinforcement in the top layer of the beam. However, there was still a large amount of deflection that was not caused by shear deformation of the concrete, elastic deformation of the column, or by hinge rotation at the interface of the beam with the column. This seems to indicate that a



a) RCBC1



b) RCBC2

Figure 5-5 Rotation Components: Unretrofitted Specimens bond failure had taken place between the longitudinal steel of the beam that was anchored within the joint and the concrete core. Since the top layer of steel was stressed in tension, once bond failure occurred these bars were prevented from being completely pulled out from the concrete by the resistance provided by the bearing of the reinforcing bar anchors on the core concrete. The post-testing evidence suggests, however, that the bars were crushing the concrete at the bearing areas, as well as sliding back and forth, thus damaging these bearing areas further, and allowing more movement to take place. Due to the arrangements of the transducers, this deformation shows up in the data partly as shear deformation.

The respective strengths and stiffnesses of RCBC1 and RCBC2 were comparable, and both specimens behaved in a similar manner, although it appears that RCBC2 was marginally weaker than RCBC1, most noticeably during negative loading.

5.7 LIMITATIONS OF THE MEASURING DEVICES

The placement of the LVDT's, and the methods used to support these measuring devices, proved not to be entirely reliable for securing exact measurements of the various deformation modes. The LVDT that measured the shear deformation was attached to the lower column by a bracing system located immediately below the joint area. Two problems with this system tended to skew the measured shear deflection of the joint area according to the direction of loading. Firstly, the area in the column immediately below the joint, where this brace was situated, was a highly disturbed area, in that this region was greatly influenced by what occurred in the joint during cyclic testing. During negative loading, when a large number of cracks appeared in the joint area, some of the stress was transferred to the bordering areas in the column and the beam. The resulting deformation caused the timber brace holding the LVDT to crush and rotate somewhat, resulting in inflated shear deflection readings. The second problem involved the spalling of the cover concrete at the rear face of the column. As the cover concrete spalled off the column, the large crack that occurred between the core concrete region and the cover concrete caused the shear deflection LVDT to record a deflection that was larger than the actual value during positive loading and smaller than the actual value during negative loading. This same spalling problem affected the pair of LVDT's that measured the combination of shear deformation and column rotation; however, the error in the individual instruments of the pair was not quite the same, as the magnitude of the error in each was dependent upon the pattern of spalling and cracking of the cover concrete.

These difficulties in measurement also affected the determination of each of the other types of rotation. The total measured rotation was a relatively reliable measurement, however, as it was measured from a point that was well removed from the disturbed regions.

It is important to note that the individual contributions by shear deformation, column deflection and beam deflection are only reasonably reliable up to the approximate yield point of the joint. After yielding, these deformations only provide an indication of the joint behaviour to the failure mode, and are thus useful in determining the mechanism of failure.

5.8 THEORETICAL VALUES

The strength values experimentally obtained for the unretrofitted specimens RCBC1 and RCBC2 conformed very well with those that were predicted based on standard section analysis of a reinforced concrete specimen (Table 5.1). Joint strength during negative loading was reduced because of the disturbed state of the joint region after attaining yield during positive loading. The bending moment (M_b) required to cause yielding at the beam-to-column interface was based on section analysis. The equivalent theoretical yielding moment at the center of the beam-to-column joint (M_y) and the actual experimental moment applied (M_{app}) are all moments measured at the center of the joint. The contributions to the total rotation by the elastic deformation of the column and the hinge rotation at the beam-to-column interface are reasonable. The only components of the total rotation that are unaccounted for by the theory are the rotations caused by shear deformation within the joint and also by the slippage of the longitudinal bars, which was greatly influenced by that same shear deformation.

Table 5-1: RCBC1 and RCBC2 Section Analyses

Specimen	**M_b (kNm)**	**M_y (kNm)**	**M_{app} (kNm)**
RCBC1			
Positive Loading	16.8	18.8	20.0
Negative Loading	32.4	36.7	24.8
RCBC2			
Positive Loading	16.8	18.8	20.2
Negative Loading	32.4	36.7	22.0

It is not necessarily important to know how much of the rotation was actually caused by the shear and by damage due to loss of bond within the joint; it is sufficient to know that such effects did occur. The important concern is to find an effective method of eliminating, or at the very least, reducing, the outcome of these occurrences.

5.9 EVALUATION OF SPECIMEN PERFORMANCE

Considering the real shortcomings in the design of the older buildings under study, and hence in their representative test specimens, the outcome of the tests was as expected. The lack of detailing for ductility in and around the beam-to-column joint resulted in poor behaviour during cyclic loading, which was characterized by the development of a low rotational ductility of $\Theta=2$. In addition, the lack of confining ties within the joint area resulted in poor shear behaviour. The deficiency in shear strength within the joint also resulted in the inability of the specimen to develop the full elastic tensile capacity of the top layer of steel. This could have serious consequences during a seismic event, particularly in light of the poor ductility of the structure. Ideally, the best possible behaviour for this specimen would have occurred if the beam would have had the ability to form a plastic hinge during both negative and positive loading; this undoubtedly would have increased the integrity of the joint.

CHAPTER 6

RETROFITTED TEST RESULTS

6.1 INTRODUCTION

A total of four retrofitted specimens were tested. Two of the specimens were retrofitted with a circular steel jacket, and two were retrofitted with a square steel jacket. In each pair, one of the specimens was previously damaged, and the other was undamaged. The specimens were labelled as follows (in order of testing):

RETRO-SU: Square, undamaged specimen

RETRO-CU: Circular, undamaged specimen

RETRO-SD: Square, previously damaged specimen (RCBC2)

RETRO-CD: Circular, previously damaged specimen (RCBC1)

The original objective was to have identical designs for each of the two retrofit shapes, in order to compare the different behaviour of the undamaged and previously damaged specimens. As testing proceeded, however, it became apparent that some undesirable effects changed the outcome of the tests, and some modifications were subsequently made.

The modifications are discussed in the appropriate sub-section. With these modifications to the jackets, it was no longer possible to make direct comparisons based on the previous condition of the original specimen; however, other comparisons can be readily made based on geometry, spacing of gaps within the retrofit, and modes of failure.

The steel jackets had a yield strength of 267 MPa (Appendix B), despite the fact that 400 MPa strength steel had been specified. The characteristic compressive strength of the original concrete was found to be f_c'= 30.3 MPa, and the strength of the grout used for the retrofit was f_c'= 31.3 Mpa. All recorded data for each of the retrofitted specimens are presented in Appendix D.

6.2 RETRO-SU: UNDAMAGED SPECIMEN WITH SQUARE JACKET

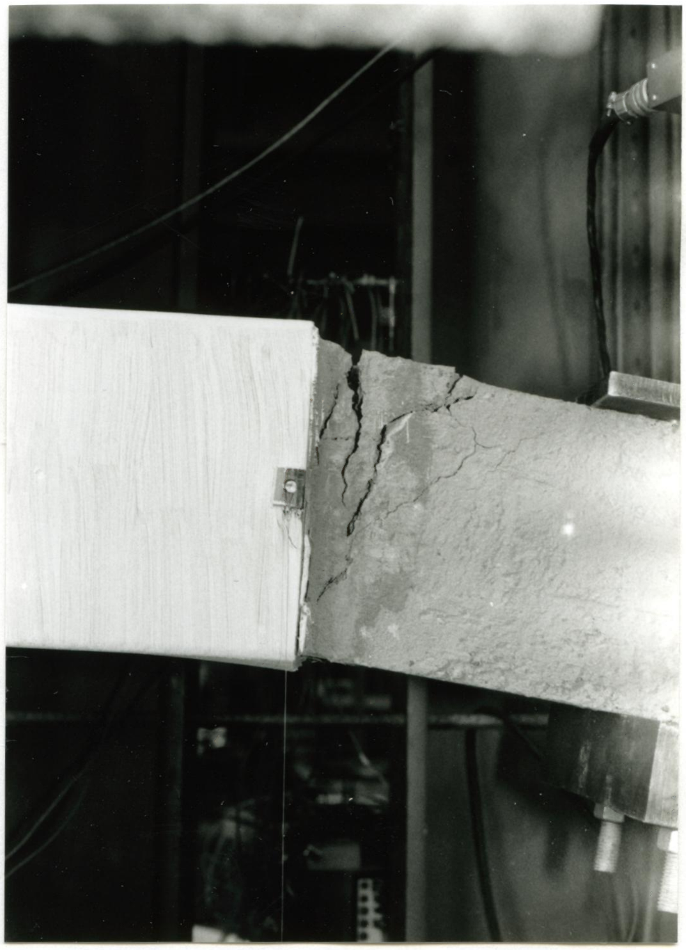
RETRO-SU (fig. 6-1a) developed two modes of failure, depending on the direction of the loading. With positive (upward) loading, a flexural hinge formed at the gap between the two sections of steel jacket in the beam (fig. 6-1b). With negative (downward) loading, a flexural hinge formed at the end of the retrofit area of the beam (fig. 6-1c). With repeated cycling this flexural hinge progressed into a shear failure of the specimen at this point (fig. 6-1d). The flexural hinge that had been developing on the bottom face of the gap area stopped expanding once the shear cracks on the upper face became very large. The failure of the specimen outside of the retrofit area was considered to be undesirable in the extreme. A subsequent section analysis of the specimen showed that the moment capacity during negative loading at the gap was large enough to deflect failure to the end of the retrofit.

a) Before Test

b) Hinge Formation During Positive Loading

Figure 6-1 Retrofitted Specimen RETRO-SU

c) Hinge Formation During Negative Loading



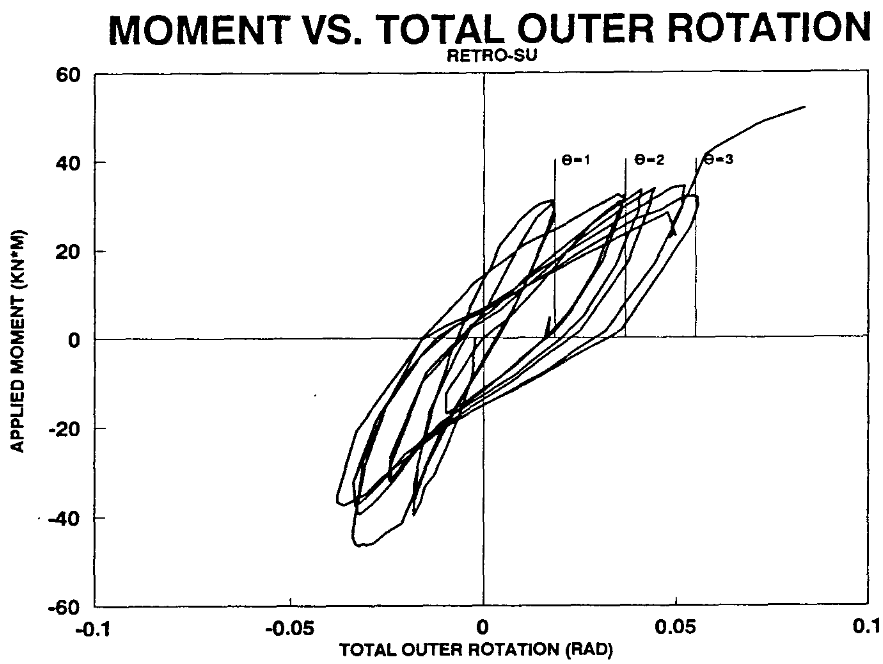
d) Failure

Figure 6-1 Retrofitted Specimen RETRO-SU

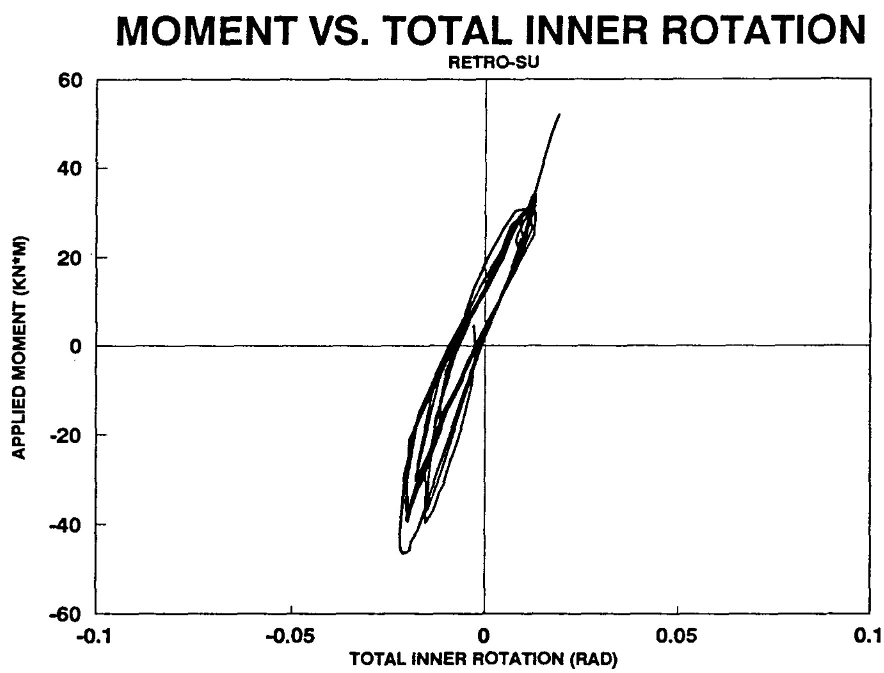
Not much strength loss was recorded during the progression of the experiment (fig. 6-2a,b), although there was some reduction in stiffness. Some pinching occurred during both positive and negative loading, but it was much less than that observed in the unretrofitted specimens.

There was not much shear deformation within the joint area (fig. 6-2d). About 5% of the total rotation of the joint resulted from shear deformation within the joint region, primarily during upward loading when the top layer of steel was in compression. The rigid rotation of the joint that was caused by column deformation tended to be the highest component of the total joint rotation, due to the increased stiffness of the joint area. The cantilever deformation of the beam area tended to be significant, but was limited once hinging began taking place either in the gap or outside the beam retrofit region. Both the rigid rotation and the cantilever rotation within the joint were elastic. The hysteresis loops of the gap (fig. 6-2c) show the progressive development of the plastic hinge at the top of the gap during positive loading. Since hinging during negative loading was outside the gap, the top layer of steel in the gap never reached yield stress, and hence little or no plastic action was observed for this portion of the loading cycle.

During the test of this specimen, a number of "pops" were heard as the bond between portions of the concrete and the steel jacket was lost and the jacket buckled outwards. This was not unexpected, since square tubing has a tendency to buckle under compression. The question of load transfer through bond became quite prevalent in this study. If no bond existed between the concrete and the steel jacket, flexural failure of the specimen should have occured in the joint area, as in the unretrofitted specimens, since this was the location

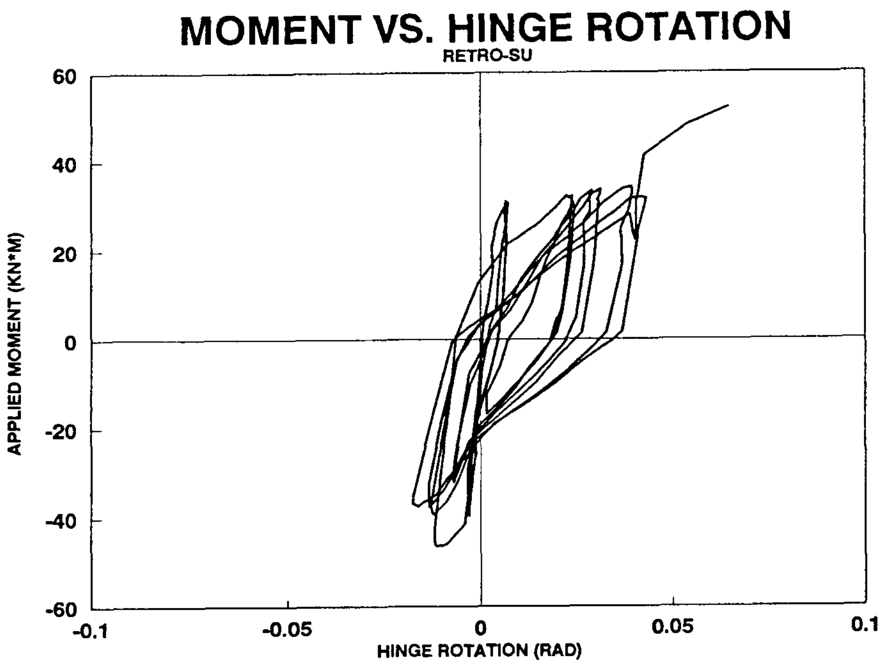


a) Outer Beam Location

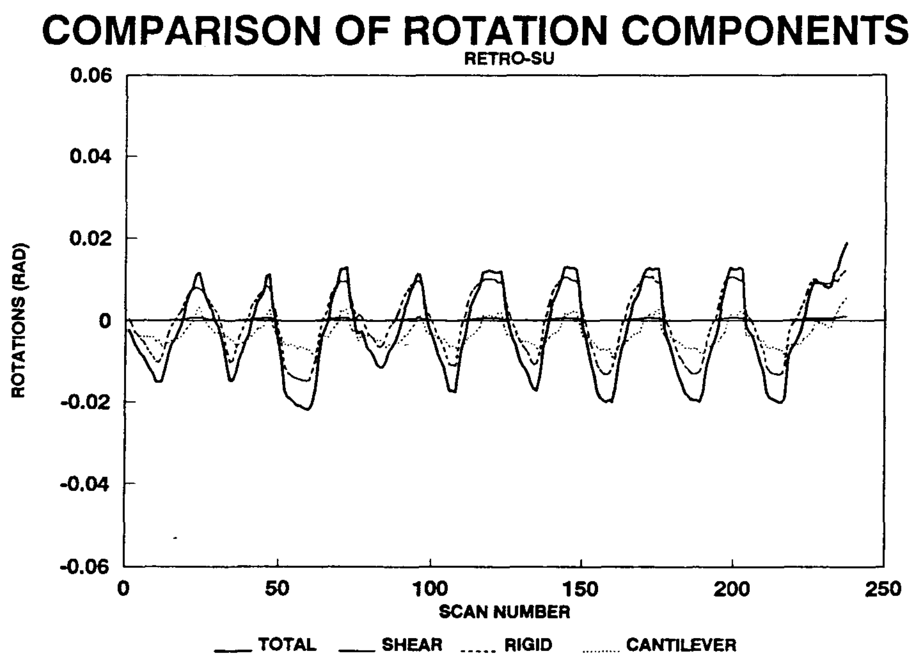


b) Inner Beam Location

Figure 6-2 Rotation: RETRO-SU



c) Hinge at Retrofit Gap



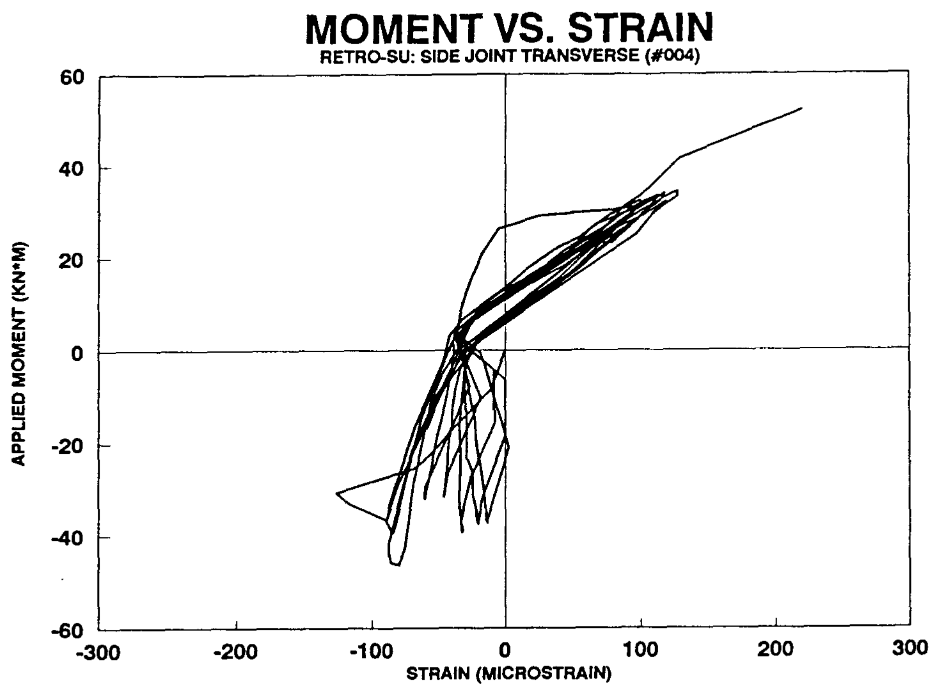
d) Rotation Components

Figure 6-2 Rotation: RETRO-SU

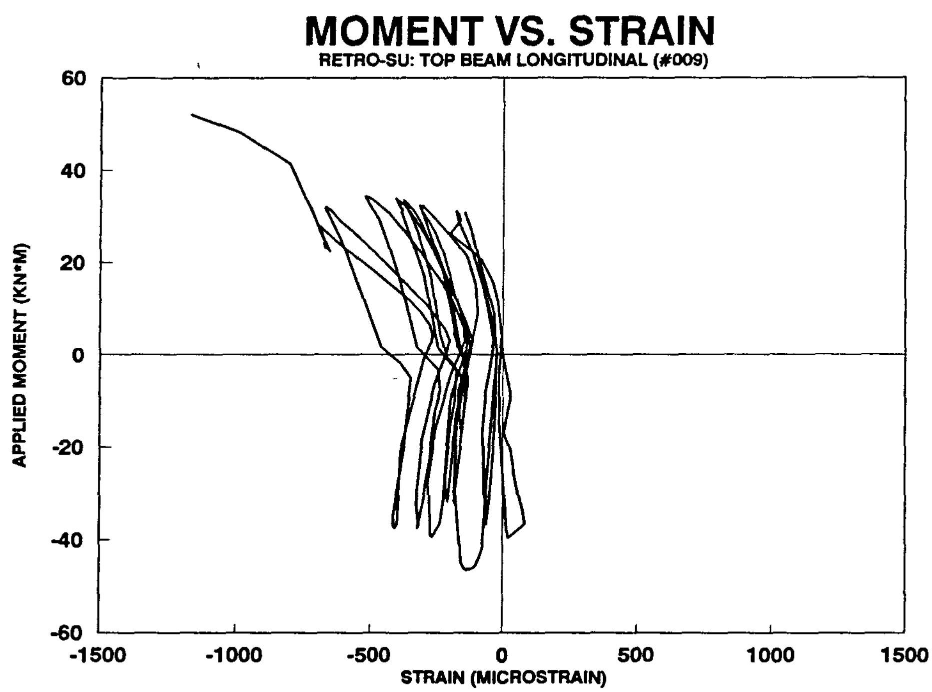
with highest applied moment. However, the flexural failures did not occur near the joint; in both cases they were located at points where no retrofit steel was present, indicating that the steel jacket significantly affects the moment capacity of the beam, and likely the column as well. Other load transfer mechanisms may also have played an important role. It is noted that the confinement properties of square (or rectangular) steel tubing are not nearly as effective as those of circular steel tubing. This is due to the fact that the faces of the square steel tube under compression tend to lose their bond with the concrete interior and hence confinement by physical enclosure is lost.

At no point during the test were plastic deformations of the jacket evident (fig. 6-3) since all recorded strains in the jacket remained within the elastic limit (less than e_y 1335 microstrain). The only visible signs of plastic deformation only occurred long after failure had already taken place in the specimen, when some bending of the jacket surrounding the retrofit gap was observed. The strains that were recorded by the gauges did not produce any major surprises; the strain distribution reasonably reflected expected patterns. The relatively low strains observed in some areas was due to a loss of bond between the concrete and steel. Loss of bond was generally indicated by an abrupt change in the strain readings (fig. 6-3a). This was accompanied by an audible "pop" when the bond was lost.

Most of the strain in the steel casing took place in the beam retrofit region bordering the retrofit gap, or at the end of the beam retrofit. The column and joint area strain gauges generally behaved elastically, suggesting no apparent damage of the internal concrete in these locations. There were also many indications of bond failure in these areas. The strain gauges on the beam jackets showed the existence of a net circumferential tensile strain in

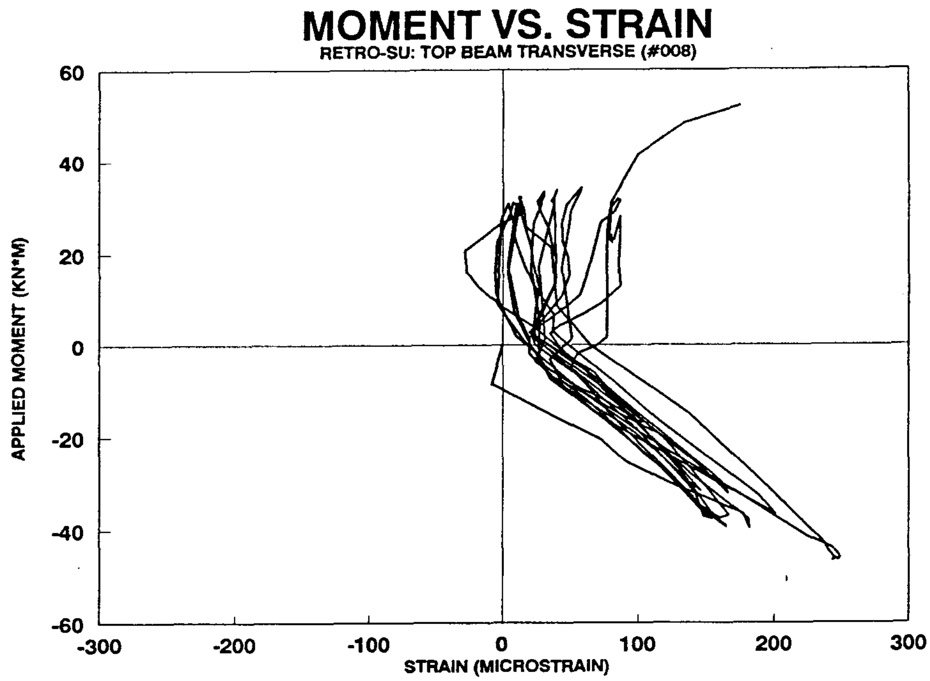


a) Side Joint Transverse (SJT)

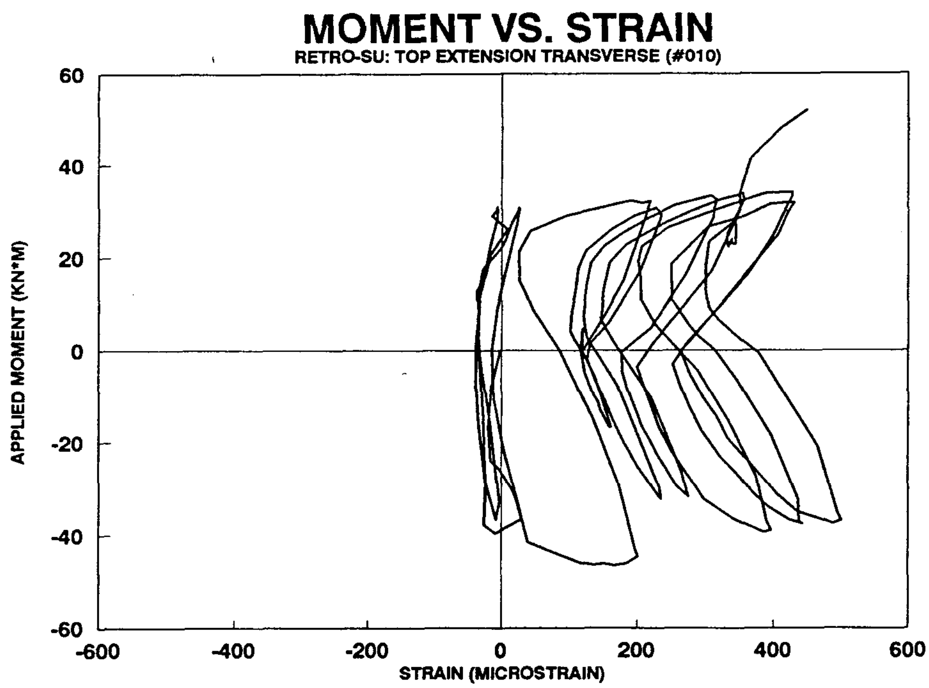


b) Top Beam Longitudinal (TBL)

Figure 6-3 Strains: RETRO-SU



c) Top Beam Transverse (TBT)



d) Top Extension Transverse (TET)

Figure 6-3 Strains: RETRO-SU

the steel on both sides of the gap (fig. 6-3b,d), and a net longitudinal compressive strain in the steel jacket between the gap and the joint (fig. 6-3c). The tensile strain in the radial direction was due to the lateral expansion of the concrete under compression; the compressive longitudinal strain was due both to this same lateral expansion and to the development of a friction bond between the concrete and steel. There was also some evidence of a permanent tensile strain in the jacket at the side of the joint, parallel to the column and perpendicular to the beam.

At one point during the test the entire specimen was lifted off the lower hinge. This was due to the fact that the downward load provided by the column actuator under load control was insufficient to hold the specimen in place. This load was subsequently increased to avoid a repetition of this condition.

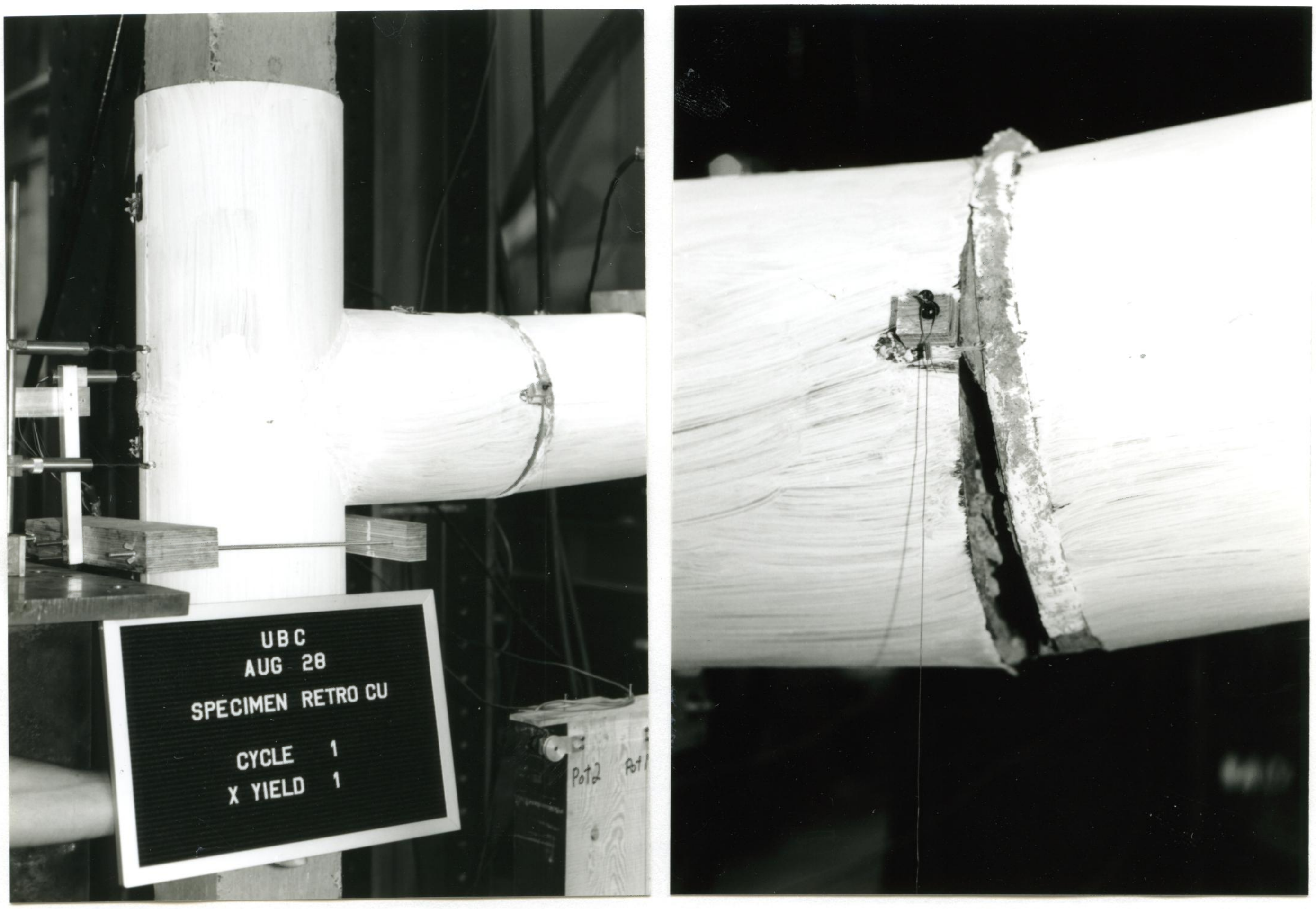
Under positive loading, yielding occurred in the retrofit gap at an applied moment (M_{app}) of 27.4 kNm. Under negative loading, the yielding occurred outside the retrofit area at the end of the beam, at an applied moment of 42.7 kNm. As indicated in Table 6.1, the location of failure and the value of the yield moment (M_y) can be reasonably accurately predicted by analysis. The bending moment (M_b) required to cause yielding at a section was based on a section analysis. The equivalent theoretical yielding moment at the center of the beam-to-column joint (M_y) and the actual experimental moment applied (M_{app}) are all moments measured at the center of the joint. The M_{app} values are the yield moments taken from the hysteresis loop.

Table 6-1: RETRO-SU Section Analyses

Failure Location	M_b (kNm)	M_y (kNm)	M_{app} (kNm)
Positive Loading			
Beam - End of Retrofit	16.8	46.5	----
Beam - Gap Region	18.7	29.9	27.4
Column	22.9	59.0	----
Negative Loading			
Beam - End of Retrofit	16.8	46.5	42.7
Beam - Gap Region	35.8	57.3	----
Column	22.9	59.0	----

6.3 RETRO-CU: UNDAMAGED SPECIMEN WITH CIRCULAR JACKET

Specimen RETRO-CU (fig. 6-4a) also developed two modes of failure, depending on the direction of loading. With positive loading, a flexural hinge developed at the gap between the two sections of the steel jacket surrounding the beam (fig. 6-4b). With negative loading, a flexural hinge once again formed outside the retrofitted region, but in this case its location was at the end of the retrofit on the upper portion of the column (fig. 6-4c). In order to avoid a catastrophic buckling failure of the column, negative loading past a ductility factor of $\Theta=1.5$ was avoided, since it was deemed more desirable to observe the effects of the development of the plastic hinge within the retrofitted zone. Had the experiment continued according to the originally planned cycle of loading in both the positive and



a) Before Test *b) Hinge Formation During Positive Loading*

Figure 6-4 Retrofitted Specimen RETRO-CU

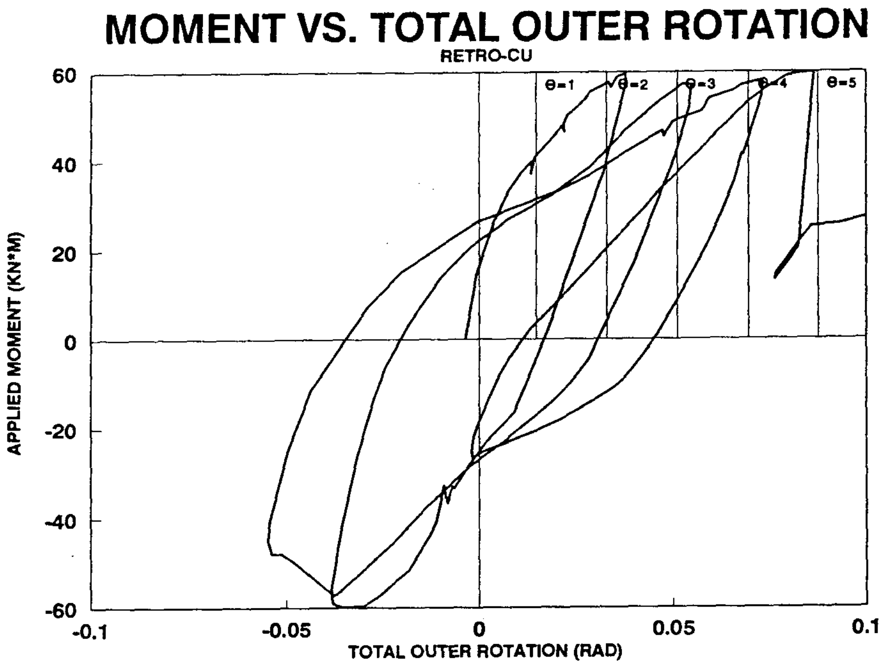
c) Column Hinge Formation During Negative Loading

d) Failure

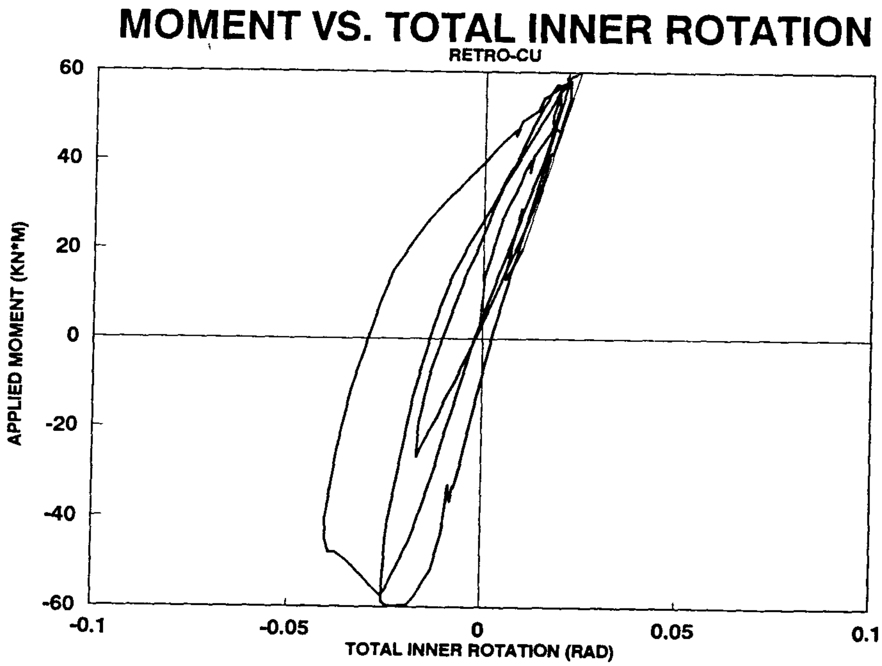
Figure 6-4 Retrofitted Specimen RETRO-CU negative directions, it is quite possible that final failure of the specimen would have resulted from the formation of a hinge within the column. As it happened, the failure of the specimen was caused by the tensile rupture of the lower layer of beam reinforcing steel under positive loading (fig. 6-4d). It is apparent that the yield moment was reached in the first cycle of testing, resulting in a ductility factor of about $\Theta=2$ (fig. 6-5). The failure of this specimen occurred during the 4th cycle, corresponding to a ductility factor of about $\Theta=5$. The flexural hinge was concentrated over a very short distance, due to the confinement of the core concrete. The column load was increased on two occasions during the testing, to avoid the specimen lifting off of the lower support.

The hinge that developed in the column was cause for concern. While a failure outside the retrofit area is undesirable, it would be potentially catastrophic to have that failure develop in the column rather than the beam. The somewhat longer sleeve of the circular jacket, compared to that of the square jacket, prevented a flexural hinge from forming at the end of the beam retrofit. The added moment capacity of the section, both due to the placement of the steel jacket and the added concrete area, caused the failure to occur in the column rather than the beam.

During positive loading, when the flexural hinge developed within the retrofit gap, there was no loss of strength of the specimen (fig. 6-5a,b). However, a reduction in the strength of the specimen occurred during negative loading due to P-δ effects. Fig. 6-5 indicates that some reduction in the stiffness of the section and some hysteresis loop pinching took place during the test, most noticeably during the negative loading, when failure occurred within the unretrofitted section of the column.

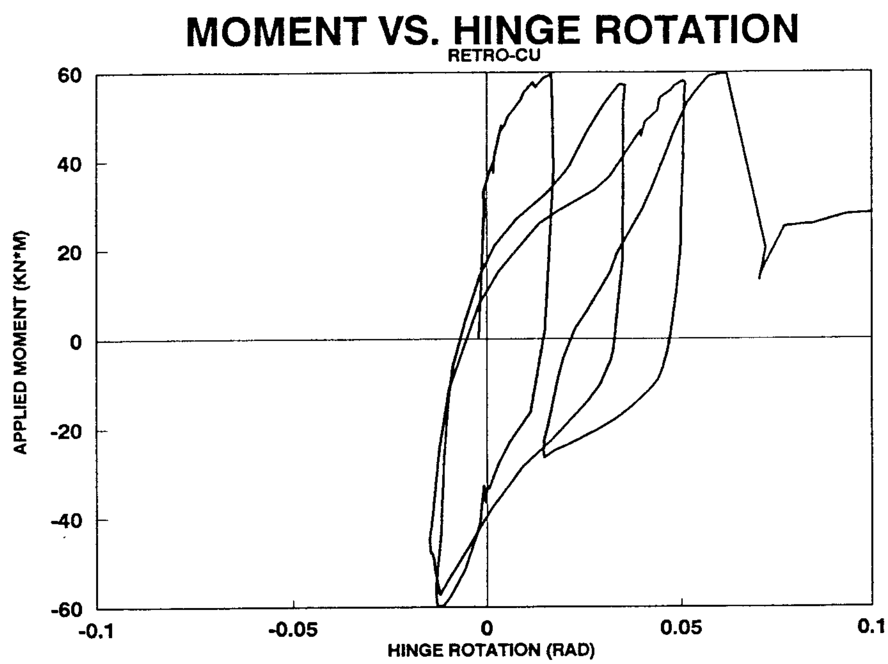


a) Outer Beam Location

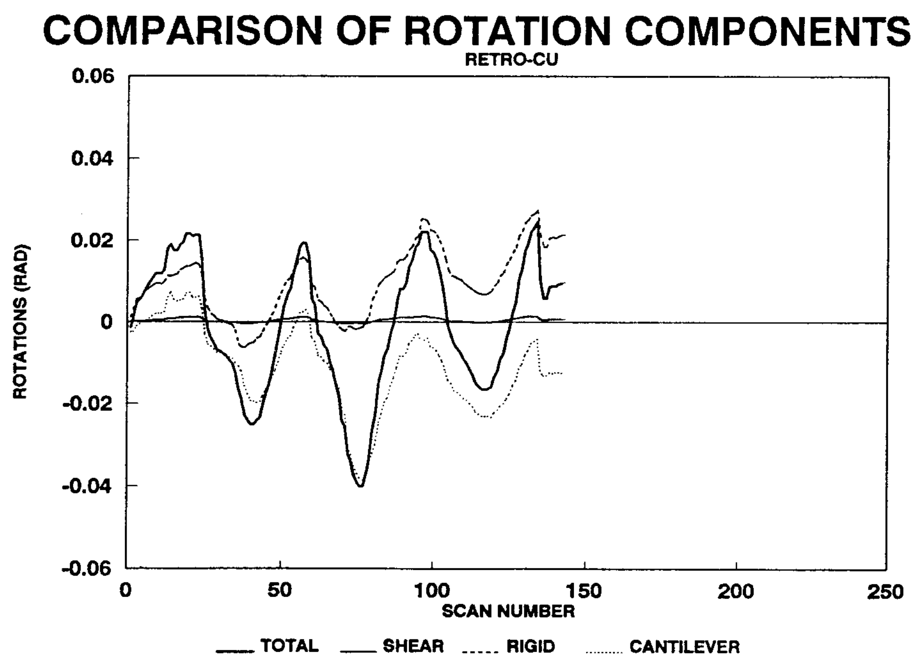


b) Inner Beam Location

Figure 6-5 Rotation: RETRO-CU



c) Hinge at Retrofit Gap



d) Rotation Components

Figure 6-5 Rotation: RETRO-CU

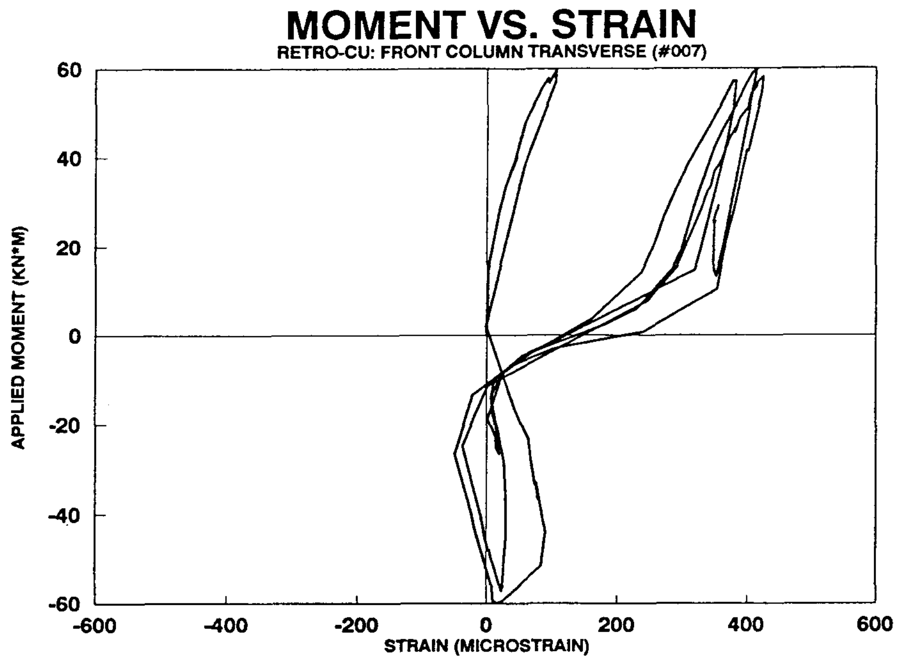
There was not much deformation within the joint area. Shear deformation again only accounted for a small portion (5%) of the total rotation of the joint (fig. 6-5d). Inconsistencies were detected between the measured and observed behaviour of the rigid and cantilever rotations of the joint area. Although visual observations indicated quite clearly that a substantial rotation of the entire joint occurred during both negative and positive loading, this is not evident from the LVDT data. The discrepency stems from the amount of deflection of the entire column, and hence the lateral movement of the joint area, during testing. The LVDTs, which are limited to an effective range to about 1" in either direction, were not able to reliably record the data once the columns deflected more than 1" laterally. Beyond this limit the LVDT measuring the lateral deflection of the top of the joint lost contact with the specimen. Thus, these readings are highly skewed for rigid rotation in the positive direction and cantilever rotation in the negative direction, and are not useful past the point of initial yielding.

The hinge hysteresis loop (fig. 6-5c) shows the progression of the plastic hinge within the bottom layer of steel during positive loading. The top layer of steel apparently never reached the yield stress under negative loading, since the flexural hinge under negative loading formed within the column.

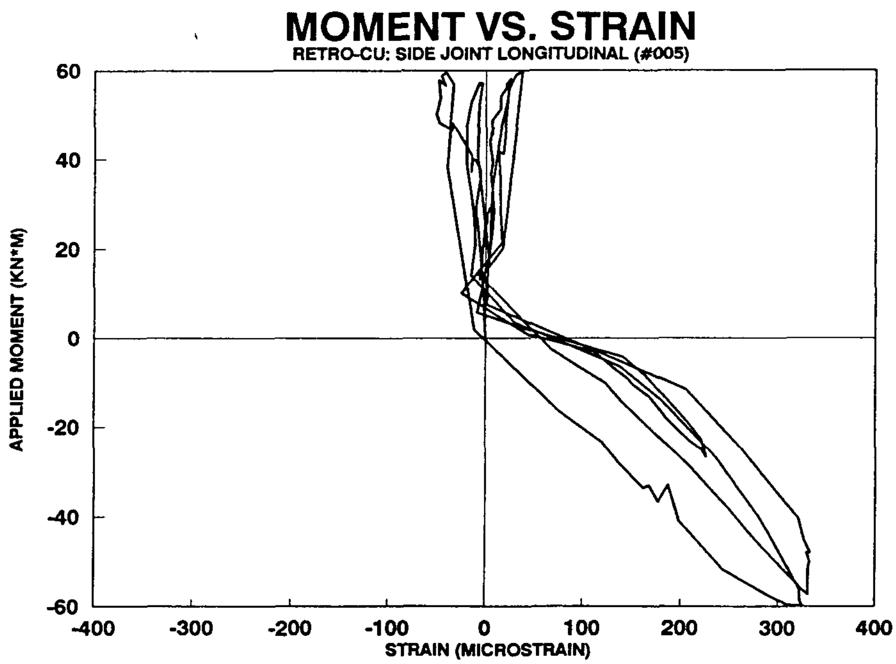
There was no audible indication and very little visual evidence of any bond failure between the concrete and the steel jacket. Only a small portion of concrete around the gap lost its bond with the steel jacket. This was due to the propagation of the flexural crack in the region where failure eventually took place. There was no reason to believe that a general bond failure took place over a substantial portion of the retrofit area. Some spalling of the surface concrete was observed in the gap area.

No portion of the jacket was strained beyond yield (1335 microstrain) during the test (fig. 6-6). Most of the measured strains within the joint area were in the 400 microstrain range. Again, the strains (and hence the stress patterns) recorded by the strain gauges did not indicate anything unusual. Observed tensile transverse strains in the retrofit area were as expected and tended to be of a higher magnitude than those observed during testing of RETRO-SU. Tensile transverse strains were monitored in RCT, RJT, SJL (fig. 6-6b) and TET (fig. 6-6d) during negative loading, and RJT, SJT, FCT (fig. 6-6a), TBT (fig. 6-6c) and TET (fig. 6-6d) during positive loading. Permanent strains occurred primarily in the beam extension retrofit (the retrofit beyond the gap), and to a lesser extent in the retrofit area between the joint and the gap. This was likely due to the damage of the concrete core, because of its proximity to the flexural hinge in the gap area.

Under positive loading, yielding was initiated in the gap region in the beam at an applied moment of 39.6 kNm. This value is somewhat higher than might be expected for this section, because both the top and bottom layers of reinforcing steel were in the tension zone. It was found that the flexural capacity of this specimen under negative loading was governed by the flexural capacity of the column (56.4 kNm) at the end of the retrofitted area. As indicated in table 6.2, the location of failure and the value of the yield moment can be predicted reasonably accurately by analysis.

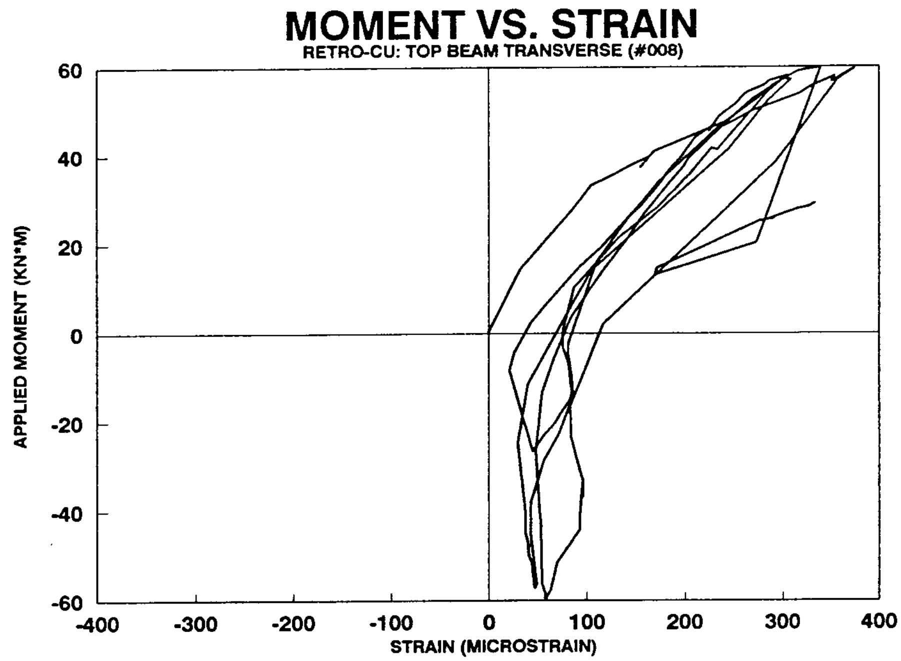


a) Front Column Transverse (FCT)

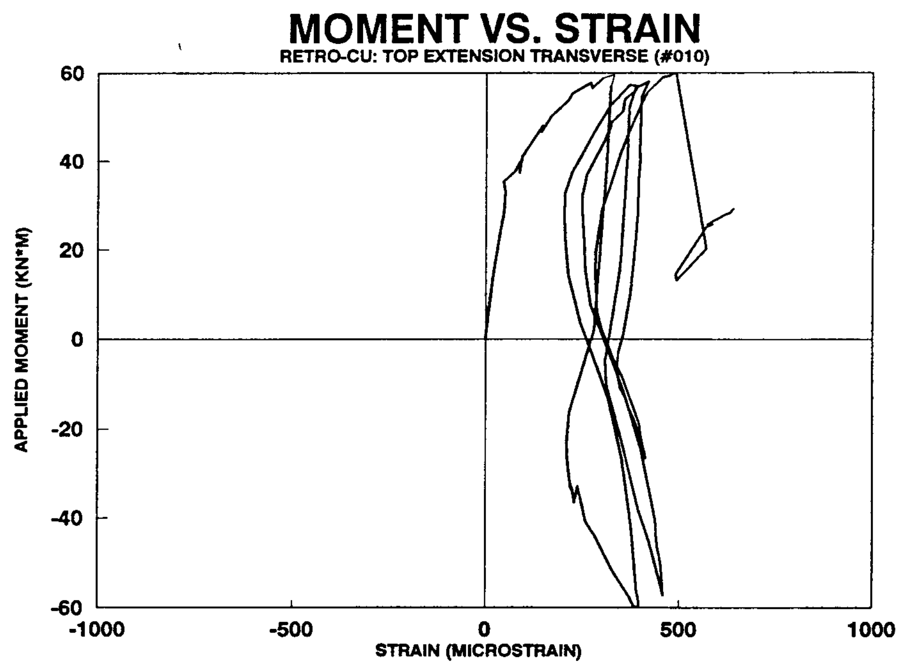


b) Side Joint Longitudinal (SJL)

Figure 6-6 Strains: RETRO-CU



c) Top Beam Transverse (TBT)



d) Top Extension Transverse (TET)

Figure 6-6 Strains: RETRO-CU

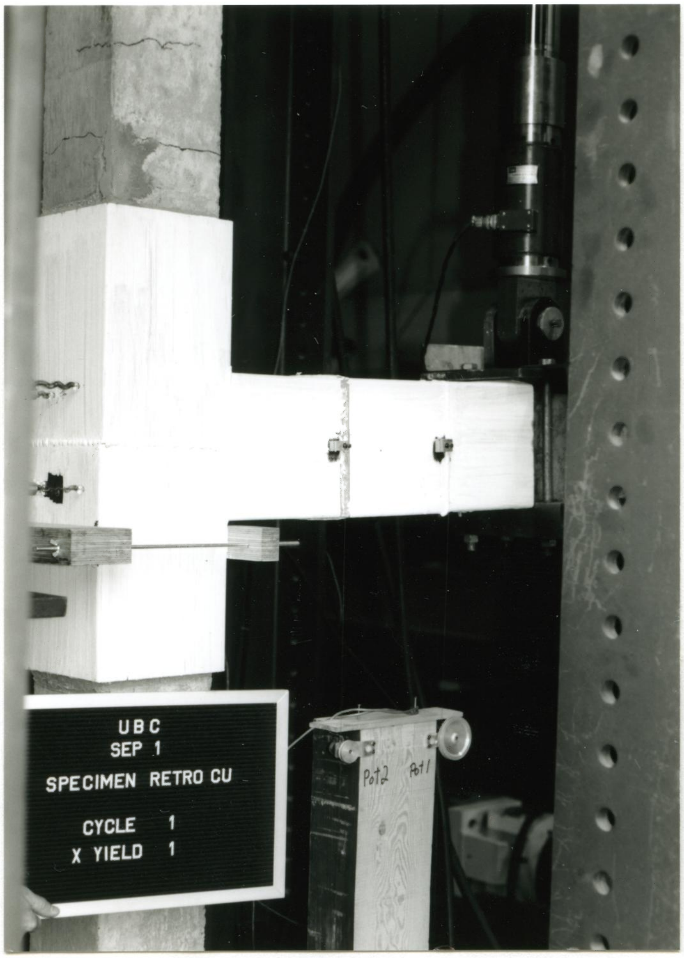
Table 6-2: RETRO-CU Section Analyses

Failure Location	M_b (kNm)	M_y (kNm)	M_{app} (kNm)
Positive Loading			
Beam - End of Retrofit	16.8	75.6	----
Beam - Gap Region	21.4	37.6	39.6
Column	22.9	63.2	----
Negative Loading			
Beam - End of Retrofit	16.8	75.6	----
Beam - Gap Region	38.9	68.3	----
Column	22.9	63.2	56.4

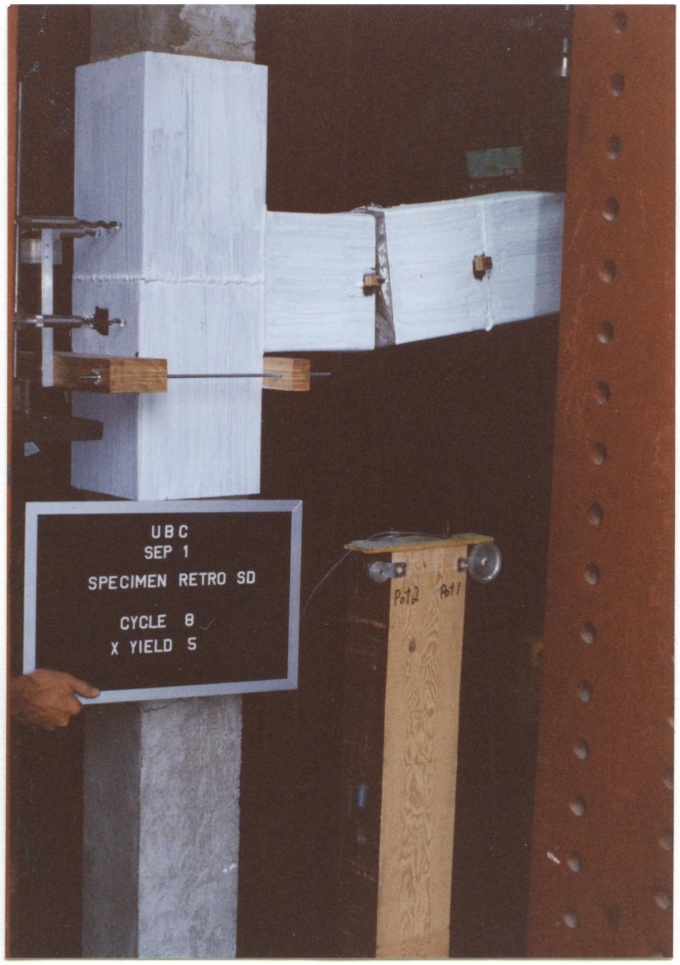
6.4 RETRO-SD: PREVIOUSLY DAMAGED SPECIMEN WITH SQUARE JACKET

Based on the results from RETRO-SU, it was decided to add some length to the steel jacket along the beam. To achieve this, the extension section of the sleeve was removed from specimen RETRO-SU, and welded onto the extension section of RETRO-SD. The entire length of the beam of specimen RETRO-SD was thus effectively wrapped in the steel jacket (fig. 6-7a). The critical section for both loading cases was now expected to be within the gap region. This specimen actually developed the desired mode of failure under loading in both directions. That is, the flexural hinge formed within the gap region under both positive and negative loading (fig. 6-7b,c).

Failure of the specimen was caused by flexural yielding and tensile rupture of the



a) Before Test



b) Hinge Formation During Positive Loading

Figure 6-7 Retrofitted Specimen RETRO-SD

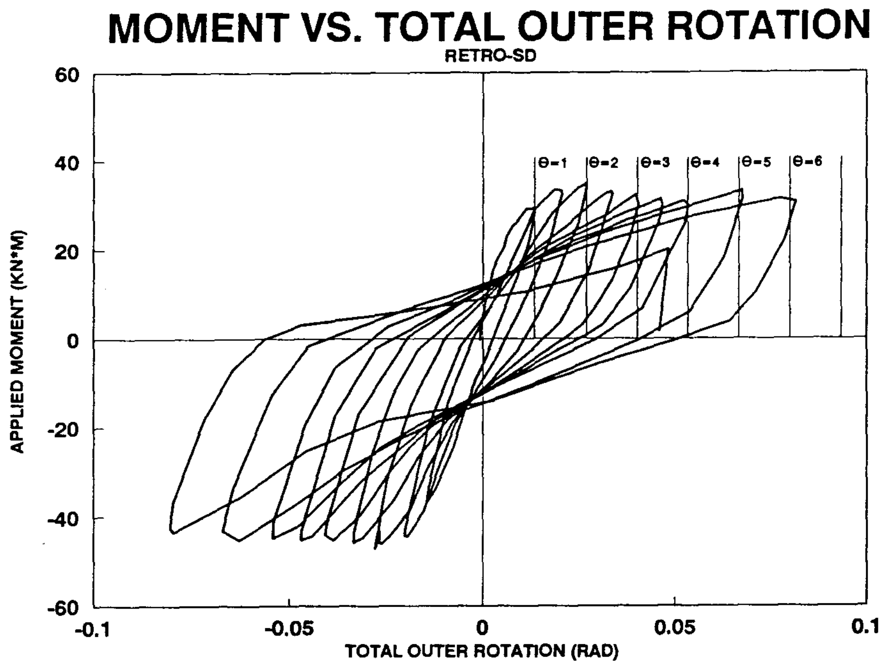
c) Hinge Formation During Negative Loading

d) Failure

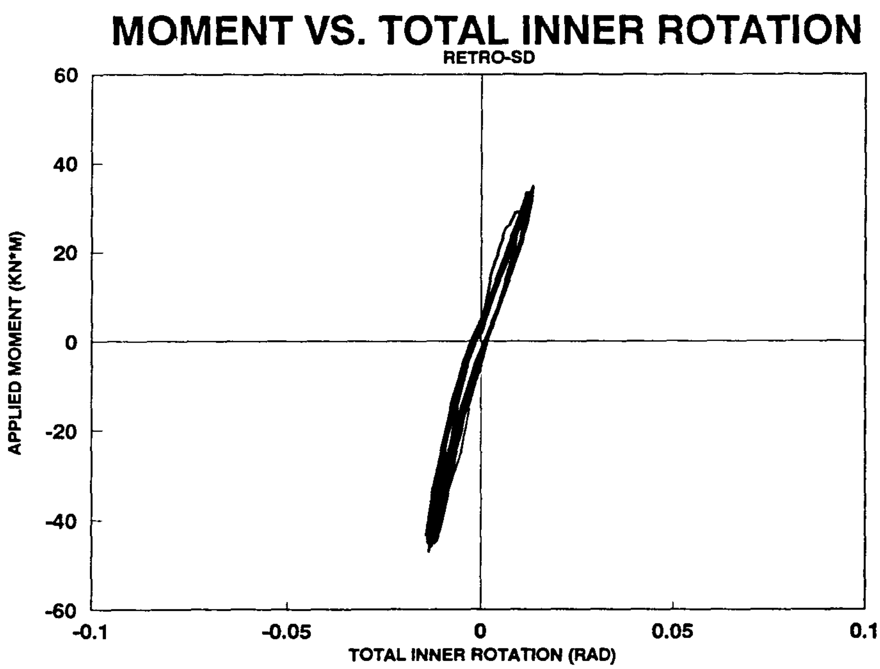
Figure 6-7 Retrofitted Specimen RETRO-SD bottom layer of steel (fig. 6-7d), but only after a very large number of cycles. A maximum ductility factor of $\Theta=6$ was reached during the ninth cycle. Failure took place on the positive loading portion of the 10th cycle; the steel failed before a ratio of $\Theta=6$ could be reattained in this last cycle (fig. 6-8a,b). Only a marginal loss of strength was recorded during cycles four to seven, but that strength was regained during cycles eight and nine, probably due to strain hardening in the reinforcement steel. In this specimen, hysteretic loop pinching only occurred during negative loading, and it was clearly smaller than the pinching noted in both RETRO-SU and RETRO-CU. This can be explained by the fact that the bending stiffness under negative loading was mainly due to the reinforcing steel (tension and compression), whereas the concrete in compression contributed significantly to the stiffness under positive loading. As the flexural cracks closed, the stiffness during positive loading did not increase until the cracked concrete surfaces came into contact again.

The shear deformation within the joint was approximately constant and of the order of 7% of the total joint rotation (fig. 6-8d). The rigid rotation caused by the column deformation, and the cantilever beam rotation also remained relatively constant over the duration of the test. There were some fluctuations in the rotation of the joint, but these are attributable to the loss of stiffness in the section within the gap area, where yielding was taking place. Because of the stiffening effect of the jacket, the rigid rotation of the joint area represented the largest component of total rotation. The cyclic motions within the joint area took place about the origin and remained elastic during the testing sequence.

The hysteresis loops (fig. 6-8a,c) that illustrate the development of the plastic hinge in the gap region indicate a stable increase in the ductility ratio. All the plastic motion

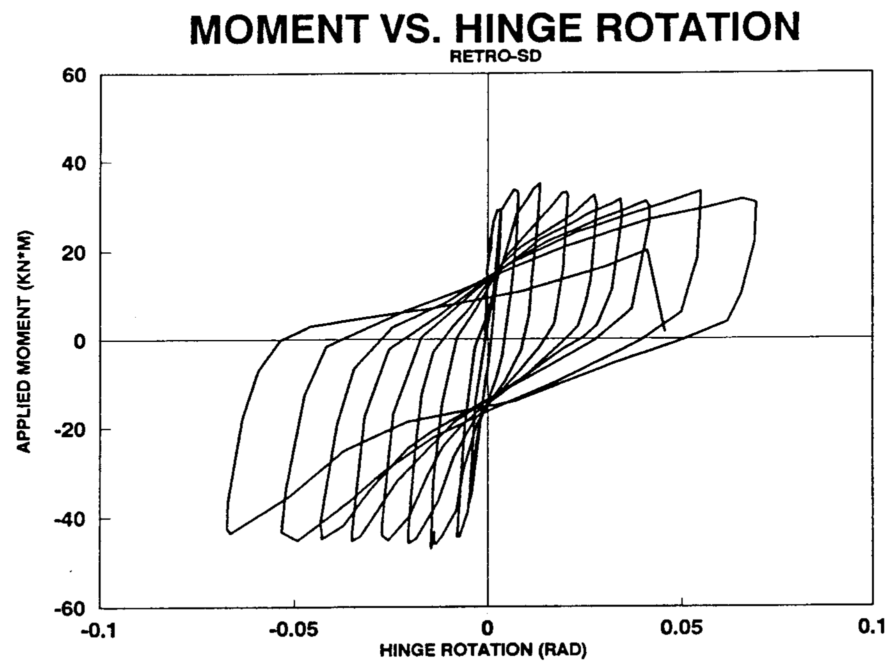


a) Outer Beam Location

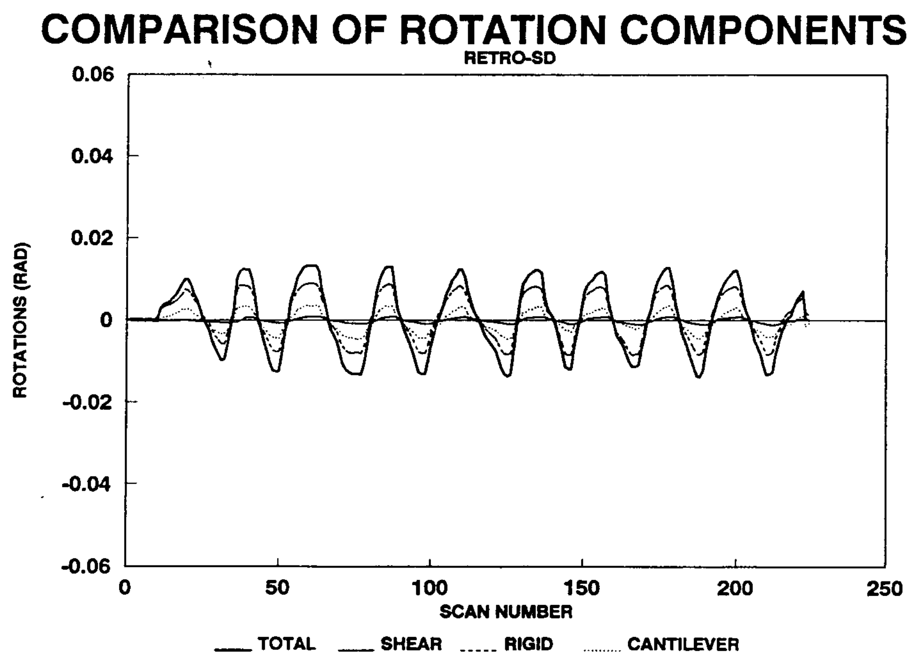


b) Inner Beam Location

Figure 6-8 Rotation: RETRO-SD



c) Hinge at Retrofit Gap



d) Rotation Components

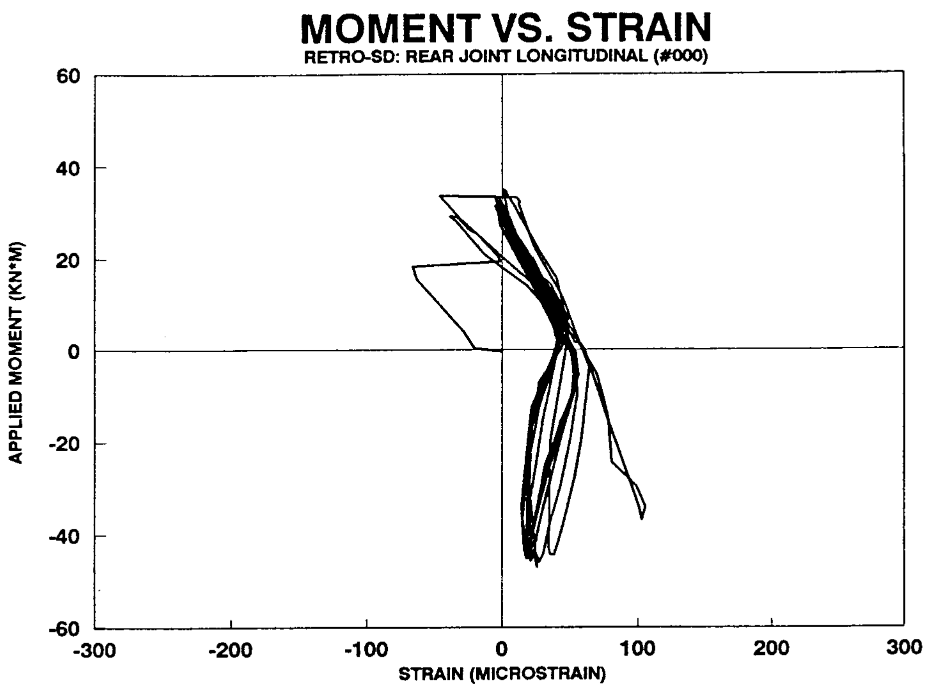
Figure 6-8 Rotation: RETRO-SD

occurred within the gap region of the retrofit.

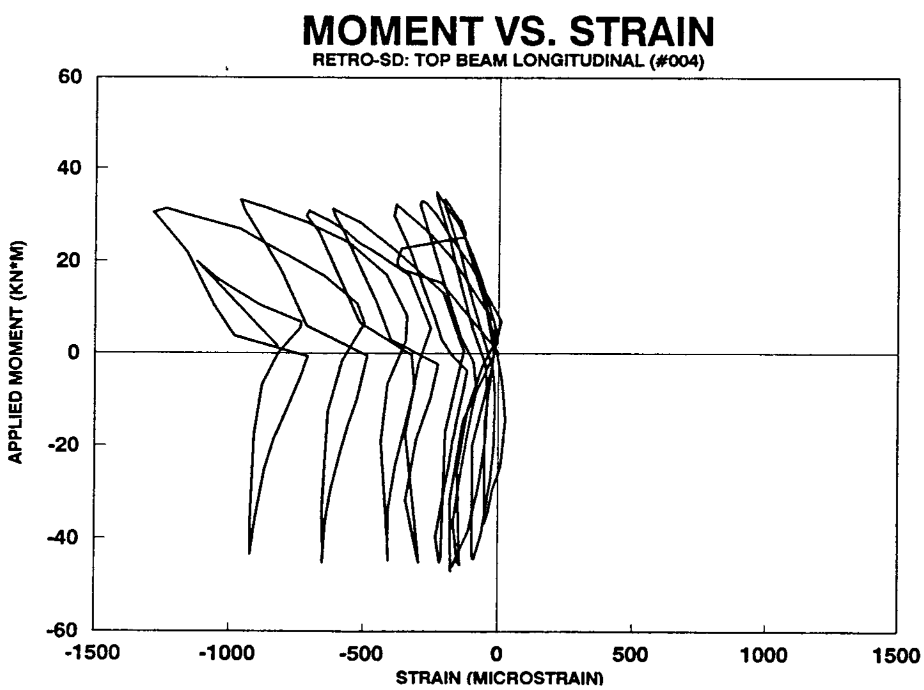
As was the case with specimen RETRO-SU, a number of "pops" could be heard during the testing cycle. Most of these noises occurred during the first two cycles, indicating a loss of bond between the concrete and steel jacket over certain regions of the retrofit. There was some visual evidence of plastic deformation of the steel jacket in the area bordering the gap of the retrofit steel on the beam. Strain gauges were not located close enough to the ends of the retrofit to record these plastic deformations. The deformations were caused by the compressive crushing and subsequent lateral expansion of the concrete in this area, and were most noticeable along the lower surface of the jacket during compression under negative loading. The deformations were characterized by an outward bulging of the middle of the jacket; the jacket corners tended to remain square.

Stress and strain patterns tended to follow the usual pattern, commensurate with the direction of loading. Good examples of indicators pointing to the loss of the concrete-steel bond can be seen in the moment versus strain graphs of FCT, SJL and RJL (fig. 6-9a). These indicators are the sudden change in strain with a small change in applied moment. Permanent deformations are visible in the beam and beam extension plots (fig. 6-9b,c,d): tensile in the transverse direction, and compressive in the longitudinal direction. These permanent deformations were located in sections bordering the area where extensive cracking occurred. Some strain gauges did not indicate any significant strains, which is not too surprising considering the remoteness of some of these areas from the plastic hinges.

Under positive loading, the flexural hinge began to form at the gap in the retrofit at an applied moment of 29.0 kNm. This yield moment was predicted using section analysis.

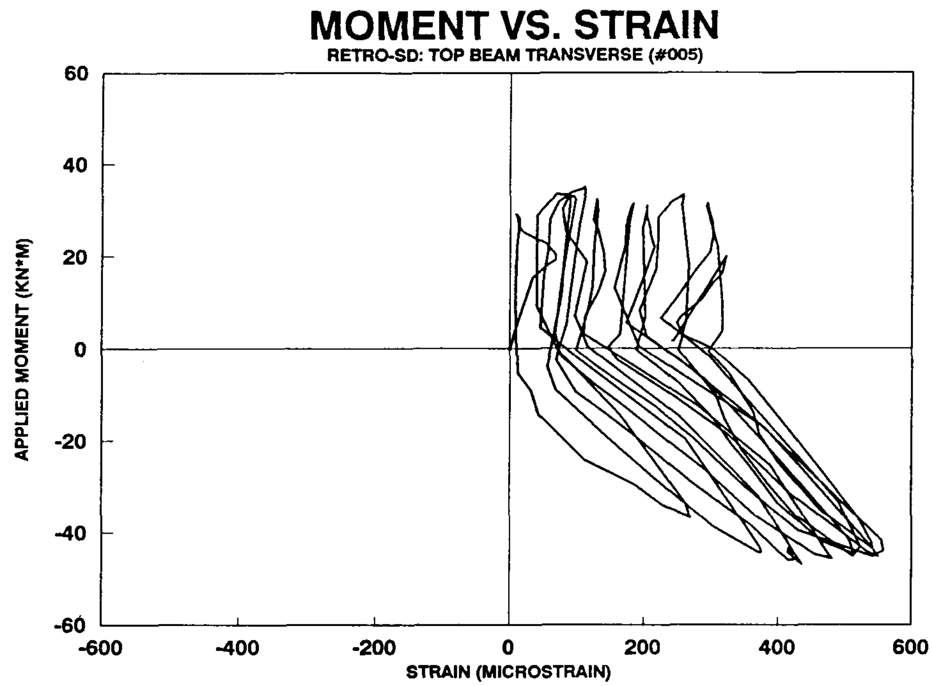


a) Rear Joint Longitudinal (RJL)

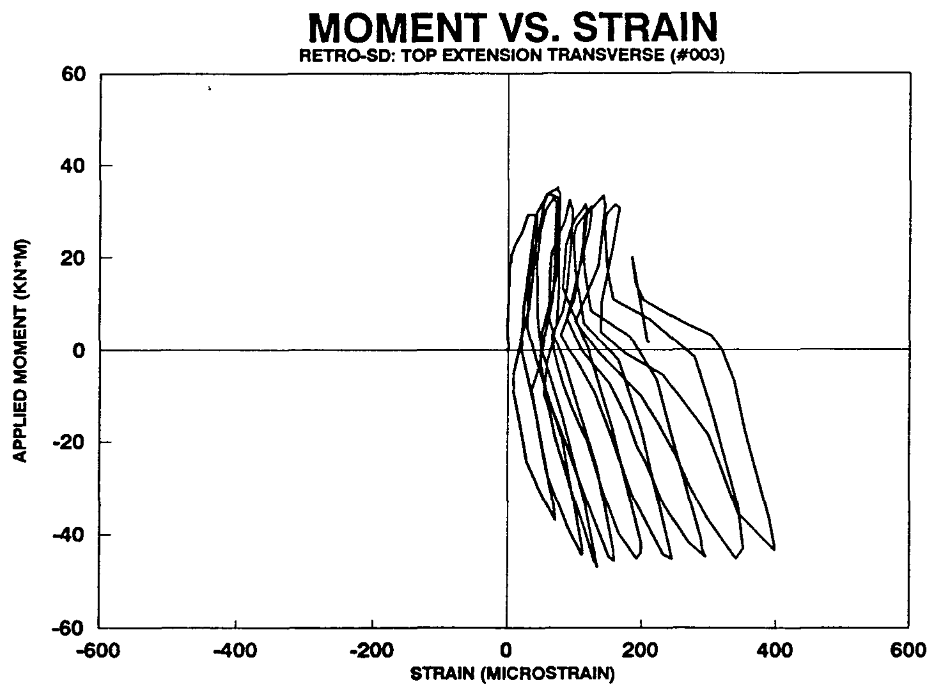


b) Top Beam Longitudinal (TBL)

Figure 6-9 Strains: RETRO-SD



c) Top Beam Transverse (TBT)



d) Top Extension Transverse (TET)

Figure 6-9 Strains: RETRO-SD

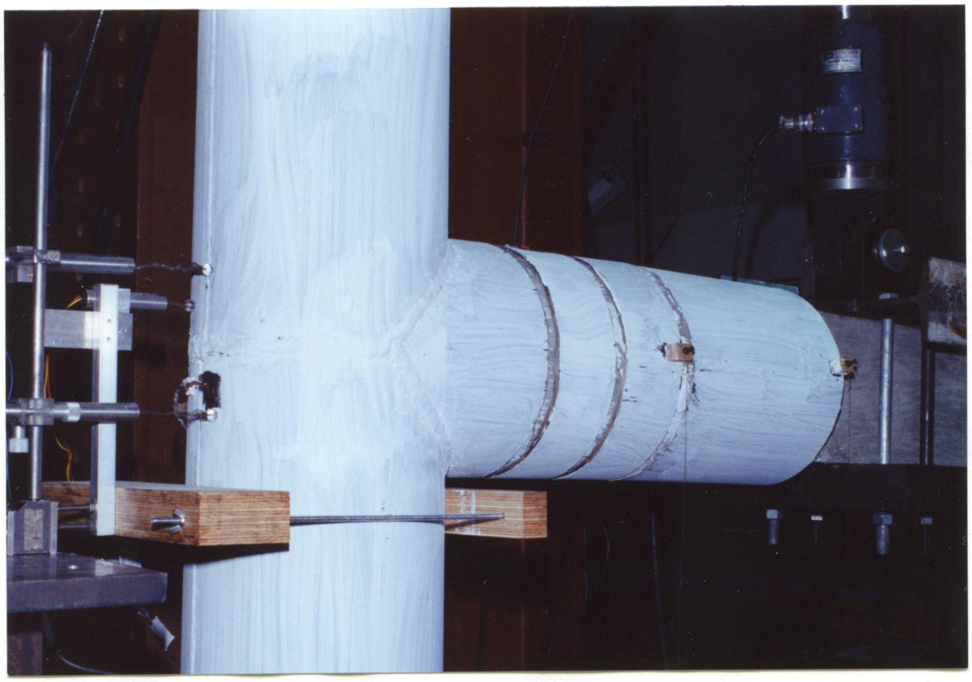
Under negative loading, the flexural hinge began to form in the gap at an applied moment of 44.3 kNm, which is substantially below the value of 57.3 kNm predicted by section analysis. This lack of correlation can be attributed to the inability of the short, longitudinal reinforcement bars in the top layer to develop their yield strength due to an insufficient development length. Table 6.3 shows that the locations of failure were as expected, even though the exact values of the yield moments were not accurately predicted in both cases.

Table 6-3: RETRO-SD Section Analyses

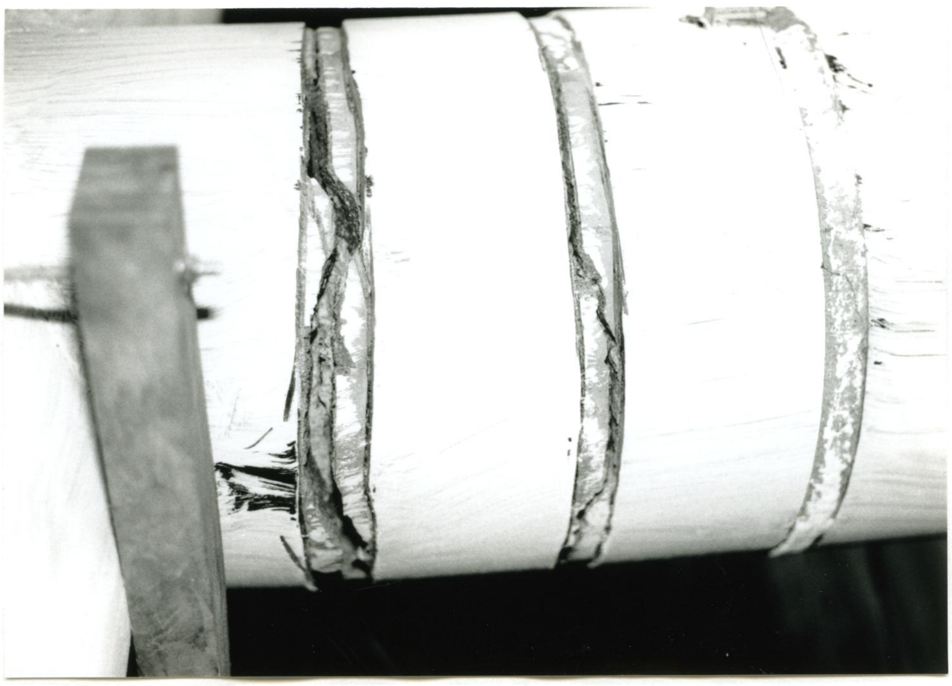
Failure Location	**M_b (kNm)**	**M_y (kNm)**	**M_{app} (kNm)**
Positive Loading			
Beam - End of Retrofit	16.8	151.2	----
Beam - Gap Region	18.7	29.9	29.0
Column	20.7	59.0	----
Negative Loading			
Beam - End of Retrofit	16.8	151.2	----
Beam - Gap Region	35.8	57.3	44.3
Column	20.7	59.0	----

6.5 RETRO-CD: PREVIOUSLY DAMAGED SPECIMEN WITH CIRCULAR JACKET

The testing of specimen RETRO-CU showed that a modification was necessary to avoid development of a flexural hinge in the column. To rectify the situation, an extra pair

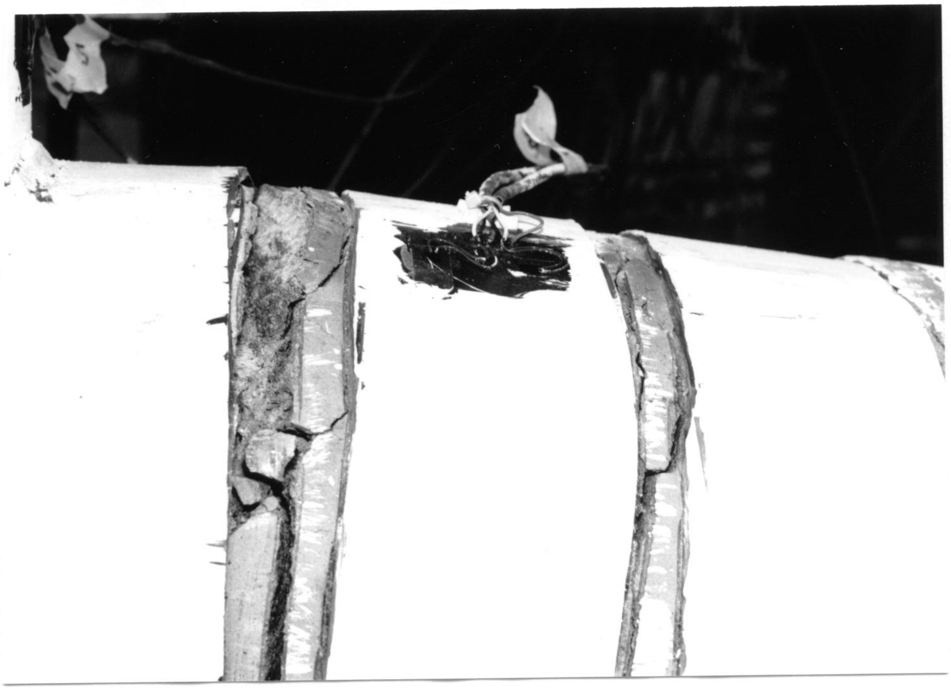


a) Before Test

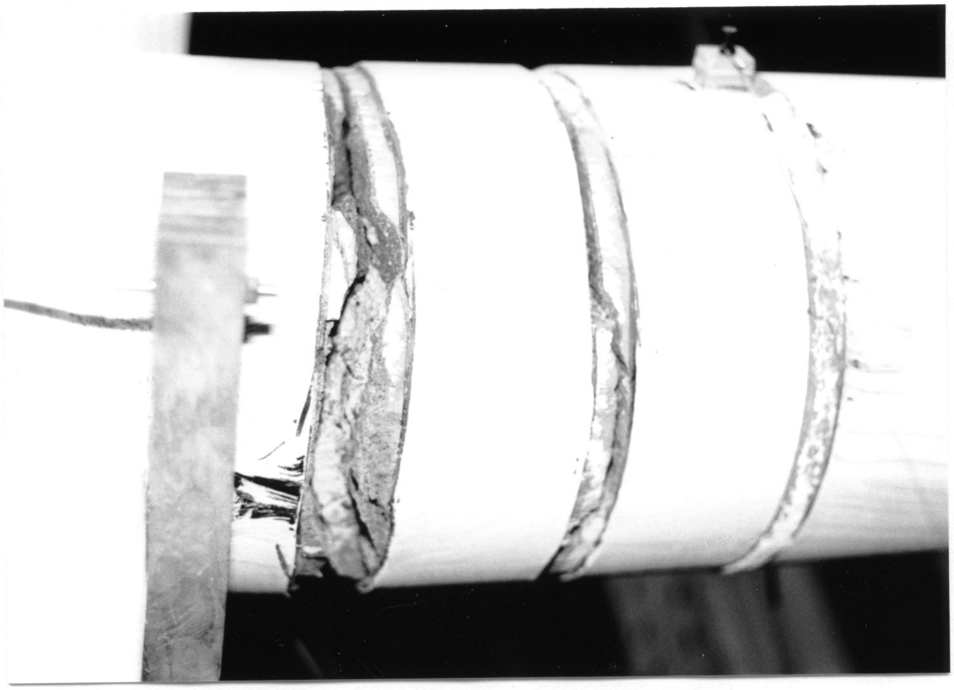


b) Hinge Formation During Positive Loading

Figure 6-10 Retrofitted Specimen RETRO-CD



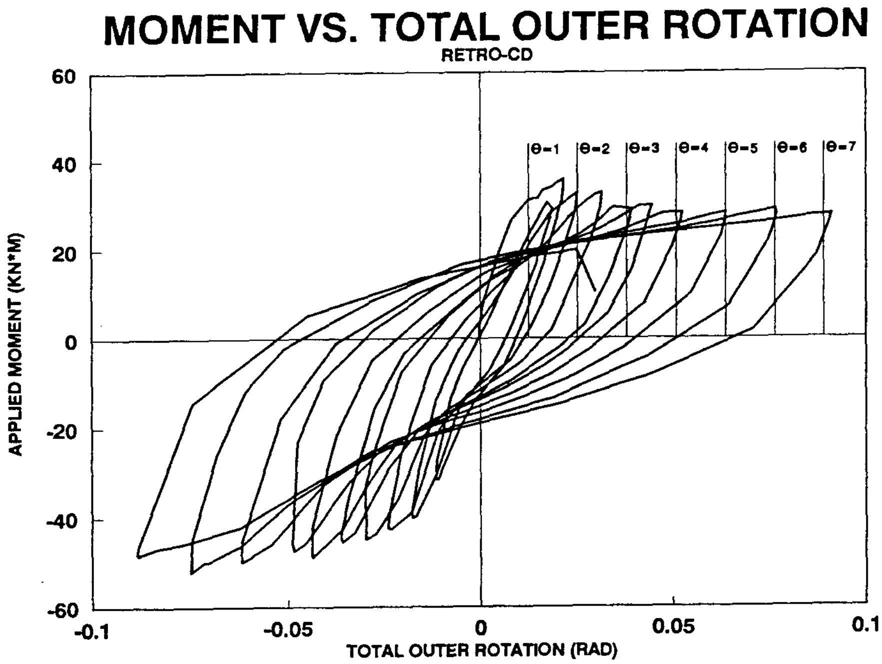
c) Hinge Formation During Negative Loading



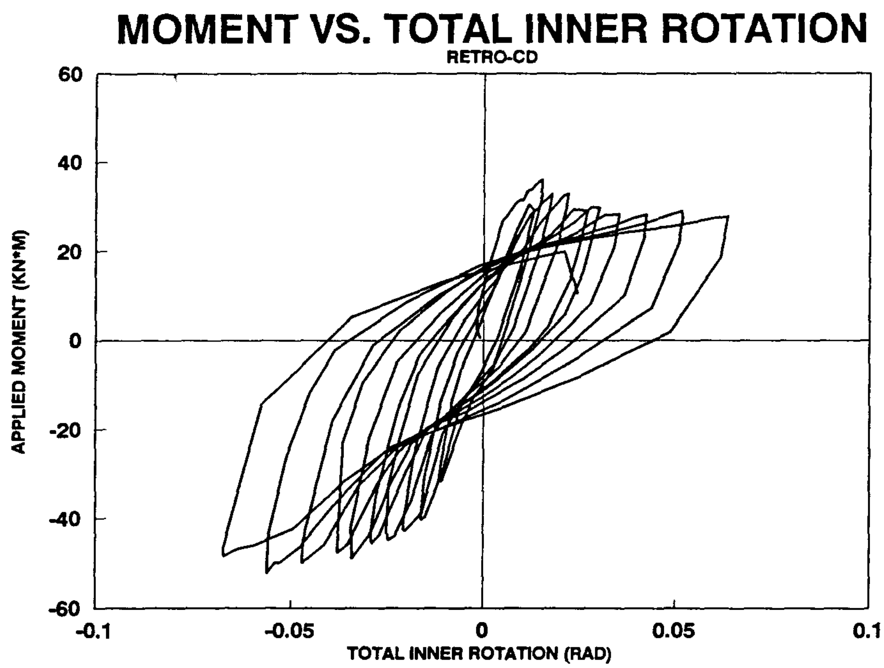
d) Failure

Figure 6-10 Retrofitted Specimen RETRO-CD of equidistant gaps were cut out of the retrofit steel on the beam between the originally designed gap and the joint (fig. 6-10a). The object of this alteration was to reduce the applied moment necessary to cause failure in the beam, and hence move the critical section for negative loading from the column into the beam area. Original concerns about the ability of the retrofitted joint area to withstand excessive cracking and deformation during testing were shown to be unfounded during the earlier tests. This specimen's failure mode was exactly as had been expected and designed for - the development of a plastic hinge within the gap region of the retrofit under both positive and negative loading (fig. 6-10b,c).

Failure of the specimen was caused by flexural yielding and subsequent tensile rupture of the bottom layer of reinforcing steel during positive loading (fig. 6-10d). The extra gaps helped to spread out the yielding length of the reinforcing bars, while still providing enough confinement for the concrete that tensile failure of the bars could be assured. The bottom layer of steel failed in tension during the positive loading portion of the 11th cycle (fig. 6-11). A ductility ratio of $\Theta=7$ was attained during the 10th cycle, and the 11th cycle was undertaken to see whether a ductility ratio of $\Theta=8$ could be secured; this was not achieved. The yielding moment was overshot in the first cycle, and was adjusted from that point onward (fig. 6-11a). A small amount of strength loss was observed during positive loading, but virtually none during negative loading. A small gain in the strength occurred in the two cycles before failure, due to strain hardening of the reinforcement bars. No significant stiffness loss took place, and the pinching behaviour exhibited by this specimen was the most favourable observed in all the specimens. The hysteresis loops were of a rounded shape, without the characteristic pinching effects for positive loading. For

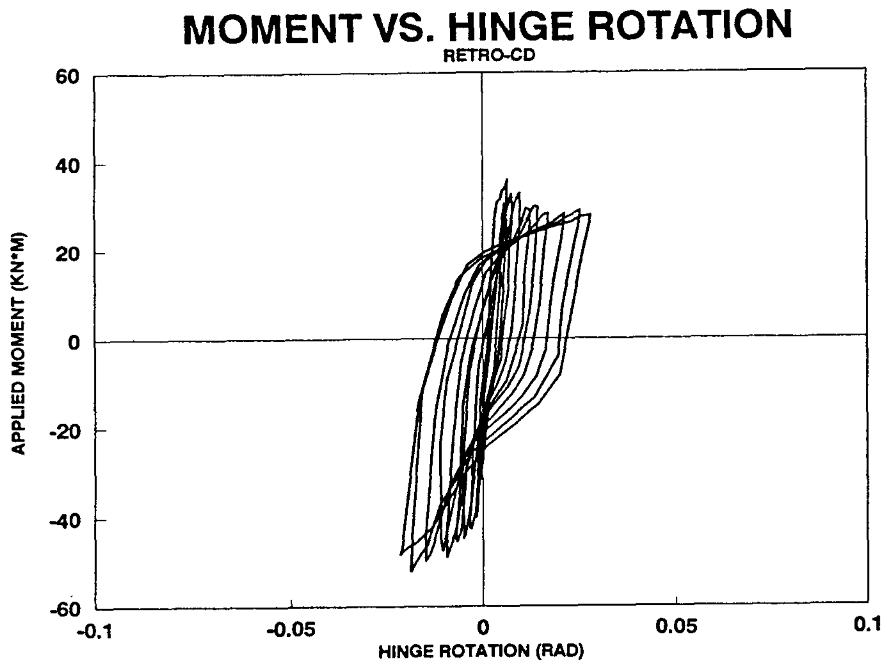


a) Outer Beam Location

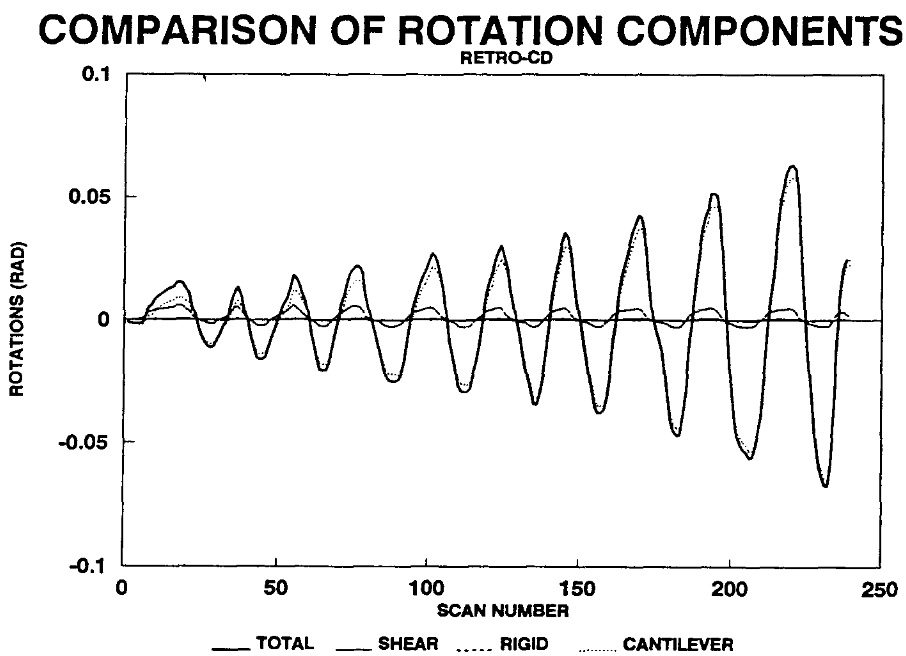


b) Inner Beam Location

Figure 6-11 Rotation: RETRO-CD



c) Hinge at Outermost Retrofit Gap



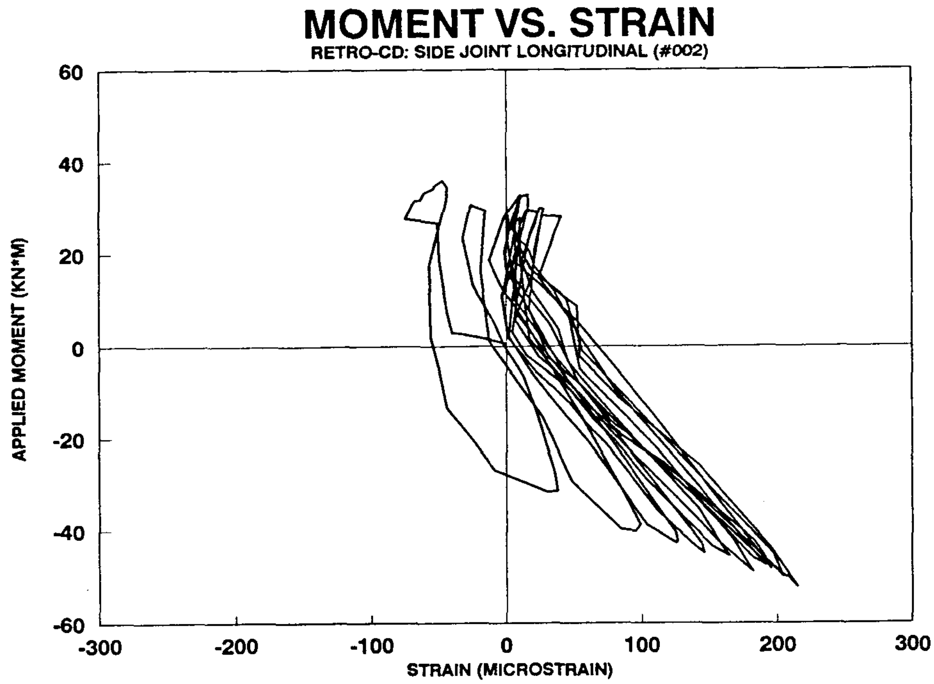
d) Rotation Components

Figure 6-11 Rotation: RETRO-CD

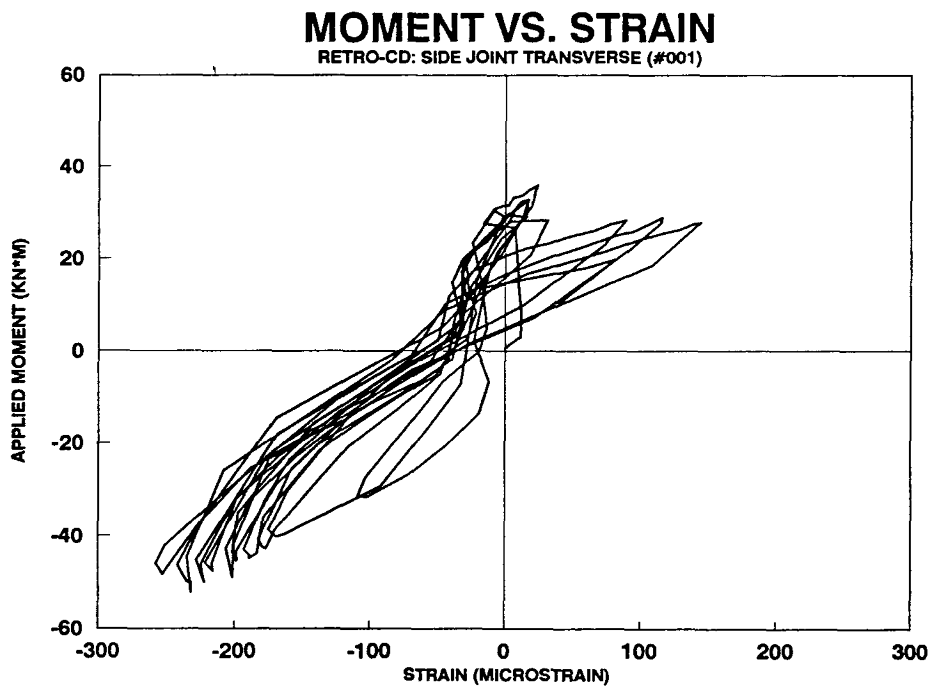
negative loading there was some evidence of pinching, which could be attributed to the loss of bond in the shorter reinforcement bars of the top layer. Most of the bending strain after yielding was concentrated in the first gap, with some deformation in the other two gaps.

The shear deformation contribution to the total rotation was virtually unnoticeable, amounting to less than 1% (fig. 6-11d). The rigid rotation caused by column deformation was constant as expected, since this was due to the elastic deflection of the column. Owing to the addition of the two extra gaps, the total rotation of the joint area included the primary area of the plastic hinge, and hence the plastic deformation of the first and second gaps. As a result, the part of the total rotation that was caused by cantilever action was extremely high. Virtually all of this deformation took place at the gap, rather than in the joint area itself. With such a short distance between the column and the first gap, the actual contribution of elastic cantilever action should have been small. The graph of moment versus total inner rotation (fig. 6-11b) includes the hinging which took place in the first two gaps, whereas the graph of moment versus hinge rotation (fig. 6-11c) shows the deformation caused by the flexural hinge in the 3rd gap. Once again, the curves of shear deformation rotation and rigid rotation cycle about the origin in a characteristically elastic fashion.

There was neither any visual or audible evidence of loss of bond between the concrete and the steel jacket over the retrofit length of the beam adjacent to the column. Localized slip occurred around the edges of the jacket at the gaps. No plastic deformation was detected by the strain gauges; none of the recorded strains reached the 1335 microstrain necessary to indicate yielding (fig. 6-12). The strain patterns recorded by the strain gauges did not indicate any unexpected deformations in either phase of the loading cycle. The

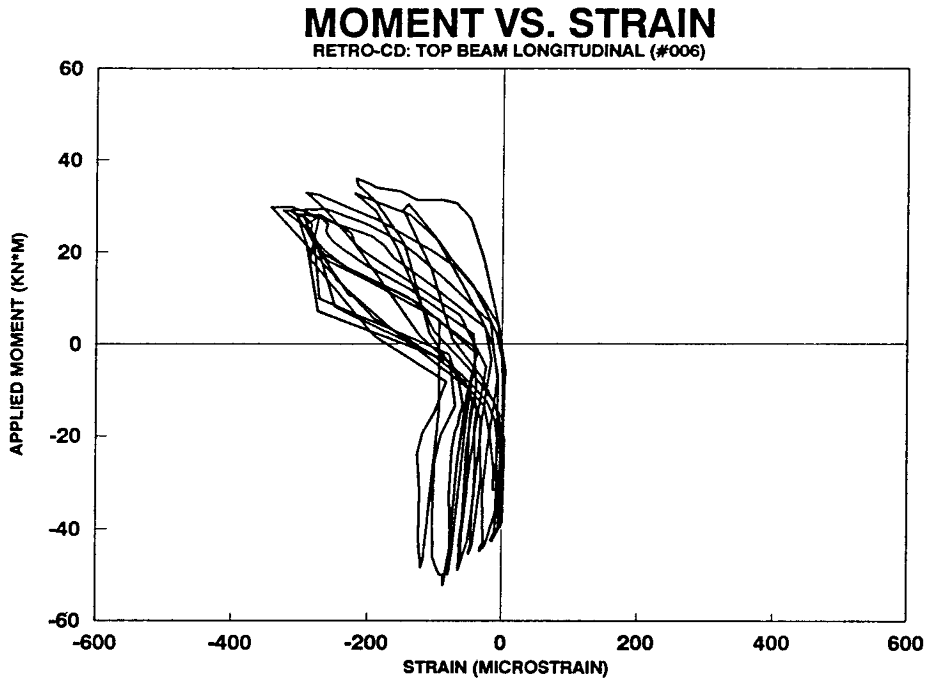


a) Side Joint Longitudinal (SJL)

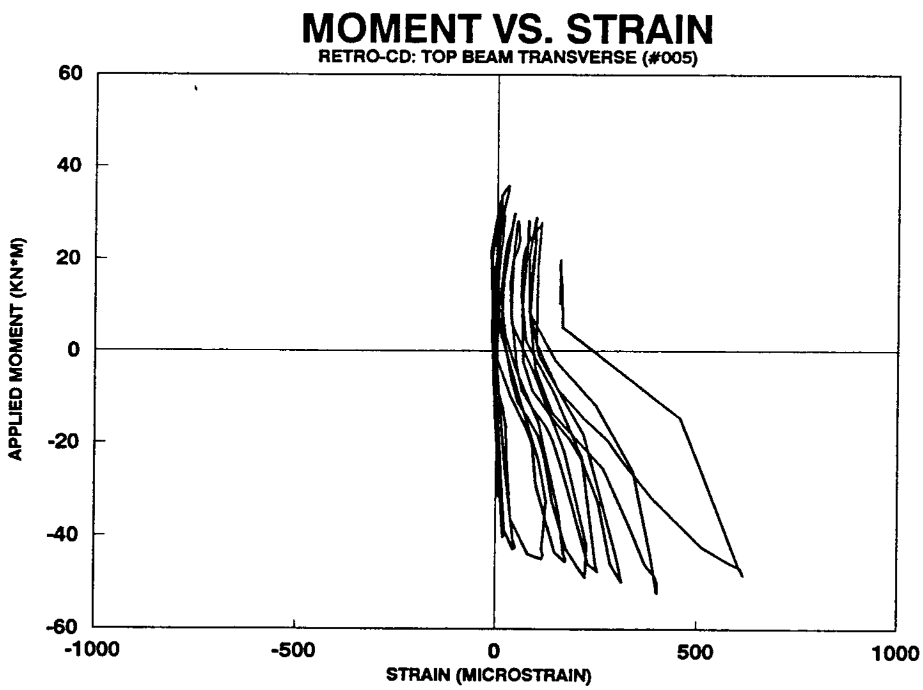


b) Side Joint Transverse (SJT)

Figure 6-12 Strains: RETRO-CD



c) Top Beam Longitudinal (TBL)



d) Top Beam Transverse (TBT)

Figure 6-12 Strains: RETRO-CD

permanent strain effects in the beam extension that had been observed in the three previous specimens were not as prevalent here. Rather, some of those effects were found to appear in the beam section adjacent to the joint, as well as within the joint area itself, as evidenced by the strain values obtained from FCT, RJL, SJL (fig. 6-12a), SJT (fig. 6-12b), TBL (fig. 6-12c) and TBT (fig. 6-12d). There had been some concern that the proximity of the gap to the joint area might unduly affect the integrity of the joint, but the measured levels and distribution of strain in this area suggest that this concern was unwarranted. The joint area was not confined by the steel jacket on all sides and the bond and confinement effects could not be relied upon in this complicated region. Except for the greater strains in the joint retrofit steel, and the greater capacity of the specimen, RETRO-CD behaved similarly to RETRO-CU.

Under positive loading, the applied moment capacity of this specimen was found to be 29.8 kNm, which was very close to the predicted value of 29.5 kNm from a section analysis. Under negative loading, the applied moment capacity was found to be 44.3 kNm, which was significantly smaller than the theoretical capacity of 51.9 kNm. Again, this can be attributed to the inability of the entire top layer of steel to attain its yield stress. From Table 6.4 it may be seen that while the positions of the flexural hinge were accurately predicted, the actual values of the applied yield moment were not.

Table 6-4: RETRO-CD Section Analyses

Failure Location	**M_b (kNm)**	**M_y (kNm)**	**M_{app} (kNm)**
Positive Loading			
Beam - End of Retrofit	16.8	75.6	----
Beam - Gap Region	21.4	28.5	29.8
Column	22.9	63.2	----
Negative Loading			
Beam - End of Retrofit	16.8	75.6	----
Beam - Gap Region	38.9	51.9	44.3
Column	22.9	63.2	----

6.6 SUMMARY OF RETROFIT TESTING

The testing program of the four retrofitted specimens exposed a few flaws in the original designs of the retrofit schemes. Because flexural failure occurred at the end of the retrofit area of the specimen RETRO-SU it was desirable to add extra sleeve length to the extension of specimen RETRO-SD. Another option would have been to cut an extra set of gaps into the retrofit steel in the beam adjacent to the joint. However, the former approach was thought to be the safest in terms of the integrity of the joint area.

Flexural failure in the column outside the retrofitted area of specimen RETRO-CU also necessitated a change in the retrofit design of specimen RETRO-CD. This was accomplished by cutting the two extra gaps into the jacket of the beam, adjacent to the joint.

All of these design changes resulted in a set of unique specimens. This presented difficulties in comparing specimens having similar geometric jackets, since comparisons based upon the initial state of the original specimens were not possible.

Because of excessive column deflection, which resulted from the development of the flexural hinge in the column, the LVDT's were found to be inadequate for measuring joint deformations in specimen RETRO-CU.

It was generally observed that while bond failure occurred in the two square specimens, no significant bond failure occurred in the two circular specimens. All the specimens developed tensile strains in their jackets during testing. However, only the circular jackets were effective in causing significant confinement stress on the core concrete, leading to a generally superior behaviour with regard to pinching.

CHAPTER 7

EVALUATION OF RETROFIT SCHEMES

7.1 INTRODUCTION

Before considering the advantages and disadvantages of the different retrofit schemes under study, it may be useful to summarize what was actually achieved in the retrofit process under evaluation. The detailed discussions that follow may help to avoid pitfalls in the future planning and designing of retrofit schemes.

First and foremost, the retrofit strengthened the deficient or damaged reinforced concrete joint area to such an extent that hinging or failure was deflected to adjacent areas: to the beam or column outside the retrofit, or to the beam within the intentionally weakened "hinge section". Any comparisons of "before" and "after" are thus somewhat misleading because the "problem" was shifted from the most critical link to the next one in line. It is thus useful to consider the retrofit process in its entirety, which involves not only assessing the benefits of introducing strengthening measures, but also of evaluating the behaviour of the post retrofit critical failure zone and of assuring that sufficient ductility is provided to maintain favourable structure response.

7.2 RELIABILITY OF BOND BETWEEN CONCRETE AND RETROFIT STEEL

In the design of the retrofit jacket, the possibility of developing considerable bond between the concrete and the retrofit steel was considered. To break the continuity of the tubes in the longitudinal direction, circumferential gaps were provided in the casing. Ideally, the steel casing would act only as confinement for the concrete, and would not increase the moment capacity of the specimen. In this ideal situation, no longitudinal bond or interaction would exist between the concrete and the retrofit steel. It is, however, impossible to expect lateral confinement of the concrete, resulting from a biaxial stress state in the jacket, without experiencing some amount of mechanical interaction (friction) in the longitudinal direction. This interaction, of course, would increase the moment capacity, perhaps significantly, particularly where the beam jacket is physically connected to the column jacket.

The chemical bond between the concrete and the steel did not appear to be very consistent, as was evidenced in certain areas of the square jacket retrofits, where bond failure occurred early in the tests. A more likely scenario would be the development of a more reliable mechanism through friction or mechanical bond between the concrete and the retrofit steel. However, in the case of the retrofit tests, there was no conclusive proof of the existence of any mechanical bond that could have resulted in an increase in the moment capacity. Although, the moment capacities of all of the specimens did experience an increase, it remains unclear which mechanisms were involved. No longitudinal tensile strains were evident in three of the four specimens in the retrofit area between the gap and the joint (RETRO-CU being the single exception). If sufficient bond of any kind had existed, it would have increased the moment capacity of the specimens and would have been detected under negative loading as a tensile strain by the longitudinal strain gauge situated at the top of the beam. In fact, during the tests this strain gauge registered compressive strains under negative loading in all cases except for RETRO-CU, indicating a complete loss of bond in the longitudinal direction. The only explanation for this situation could be the influence of the transverse Poisson effect from the circumferential confinement strains. Without any contribution to the moment capacity from the steel jacket through bond, flexural yielding should have taken place at the beam-to-column interface.

The actual situation appears to be one that is intermediate between the two extremes. The moment capacity of the section increased, but not because of a strongly positive mechanical or chemical bond between the concrete and steel. Rather, the bond appears to have been fairly weak, but sufficient to provide a certain amount of friction between the two surfaces, and enough to move the critical section along the length of the beam to the retrofit gap. This would imply a fair amount of slippage between the concrete and steel surfaces, so that all longitudinal tensile strains would be relieved. Because of cracking of the concrete in the tension region, it cannot be concluded whether slippage did indeed occur in the retrofit section between the gap and the joint for the square specimens (as large amounts of concrete spalling took place in this area during the rotation of the flexural hinge that formed at this point). A fair amount of slippage was, however, evident in the RETRO-CD specimen.

7.3 IMPROVEMENT IN DUCTILITY

The overall improvement in ductility of the specimens was substantial regardless of the jacket shape used in the retrofit. The original unretrofitted specimens RCBC1 and RCBC2 were unable to withstand even two cycles of loading at a ductility level of $\Theta=2$. In comparison, even the square retrofits (RETRO-SU and RETRO-SD) provided a substantial improvement of ductility, provided that failure occurred within the retrofitted zone. For RETRO-SU, there was still substantial strength and stiffness at $\Theta=3$, before failure actually occurred outside of the retrofit zone. For RETRO-SD, flexural failure occurred after $\Theta=6$ had been successfully attained, showing a very significant improvement.

The circular retrofit specimen RETRO-CU experienced a rotational ductility of about $\Theta=5$ before a flexural failure took place in the gap region. This has to be qualified because the onset of hinging in the column prompted a change in the intended loading cycle to force a failure in the retrofit area. If testing had proceeded with a full cyclic load (in both positive and negative directions) as originally planned, failure would certainly have occurred in the column outside the retrofit area, before this level of ductility was reached. RETRO-CD showed a remarkable improvement in ductility by successfully attaining a rotational ductility of $\Theta=7$ before flexural failure occurred.

It should be noted that the true extent of the improvement in ductility for RETRO-SU and RETRO-CU cannot be fully assessed, since in both cases failure mechanisms developed outside the retrofit zones, which affected the results of the intended experiments.

7.4 POSITIONING THE GAP FOR PLASTIC HINGE DEVELOPMENT

The gap in the beam retrofit was initially incorporated in the design concept in response to the anticipated increase in the moment capacity from chemical and/or mechanical bond between the retrofit steel and the concrete. The purpose of the gap was to contain failure within the retrofitted zone of the specimen, and also to limit the resulting increase in moment capacity. The gap was placed away from the joint area, due to the uncertainty of whether the joint area would be able to withstand extensive cracking without loss of bond and shear strength, even when retrofitted. It should be kept in mind that the joint area was not entirely surrounded by the steel jacket, and placing a gap very close to the beam-to-column interface would remove some of the confinement protection provided by the jacket. Once cracking occurred within the gap area during the formation of a plastic hinge, it was necessary to prevent this network of cracks from extending into the joint area, which was shown to behave poorly during the unretrofitted tests. This was of greater concern for the square jacket retrofits which, because of their geometry, were not expected to benefit from the confinement to the same extent as the circular jacket retrofits.

Two major considerations thus governed the positioning of gaps: reduction of the moment capacity, which would be most effective with a gap close to the joint, and preservation of the integrity of the joint zone, which called for an uninterrupted casing close to the joint. A third consideration entered at a later stage, namely the placement of multiple gaps to provide an extended hinge zone. The higher moment capacities of retrofitted sections resulted in failure mechanisms outside of the retrofit areas, which prompted modification of the retrofit design for subsequent tests: (a) the retrofit sleeve of RETRO-SD was lengthened in response to an earlier failure mechanism developing outside the retrofit zone of RETRO-SU, and (b) extra gaps closer to the joint were placed in the beam retrofit of RETRO-CD in response to the column failure mechanism and relatively low ductility levels developed in RETRO-CU. The desired behaviour was subsequently observed for RETRO-SD and RETRO-CD. No detrimental behaviour occurred in the joint area of RETRO-CD despite the close proximity of the retrofit gaps to the joint area.

7.5 DIMENSIONS OF THE RETROFIT JACKET

The thickness of the steel jacket was found to be sufficient for the purposes of this particular study. The basis of design was to replace the missing transverse steel in the joint area, on a volume basis, with an equivalent amount of steel in the form of a steel jacket. A major consideration dictating the use of a steel jacket was ease of fabrication. Weldability of the casing required a minimum thickness to avoid excessive distortions and burn-through when joining the component parts of the steel jacket. The weld needed to be able to withstand the high stress concentrations that would occur in the beam-to-column joint area; this could be a severe requirement, especially under repeated yield cycles.

The fact that the retrofit steel had a yield strength of 267 MPa rather than the specified 400 MPa did not alter the outcome of the experiment. Ideally, the steel should not add moment capacity to the specimen, so a lower yield strength actually proved to be an advantage in this particular case. In the design, the thickness of the jacket was chosen on the basis of replacing 400 MPa transverse steel stirrups with an equivalent amount of 400 MPa steel jacketing. Shear failure was not considered a problem with these tests, and the purpose of the jacket was entirely to confine the concrete core. The 267 MPa steel fulfilled that purpose without yielding. The 400 MPa steel would have behaved in much the same manner since the elastic properties of steel do not vary noticeably with grade.

The length of the jacket sleeves and the placement of the gaps proved to be the critical dimensions of the steel casing to avoid failure mechanisms occurring outside the retrofit zone. Two retrofit possibilities arose: extend the jackets sufficiently to force a failure in the joint itself, or provide a weakened hinge zone in a non-critical area. In both square and circular jacket cases, if the retrofit gaps could be moved closer to the joint, the dimensions of the jacket as originally designed would be acceptable. In the case of RETRO-SD, this was viewed as an unnecessary risk, and the sleeve extention was opted for. The retrofit lengths were based on the dimensions of the specimen after the retrofit; therefore the circular and square retrofit sleeves had different lengths.

7.6 CONFINEMENT EFFECTS

There was no evidence of an increase in compressive strength of the concrete due to triaxial confinement by the jacket. On the other hand, all failures of the retrofitted specimens took place at sections that were not completely confined. At the retrofit gap, confinement could not be relied upon, because the jacket did not fully enclose the concrete. The measured moments at yield were generally consistent with predictions based on the conventional unconfined strength of the concrete. Had the concrete been able to develop a larger compressive strength, the yield moment would have been even greater. Concrete needs to be in triaxial compression to develop this extra compressive strength. In the beam section of the specimens, this could only occur within the retrofit jacket. Even there, only a small portion of the cross-sectional area would actually be in compression. The only way to test this would be to force a failure in the beam portion of the specimen across a section that is entirely encased in the retrofit steel. In general it may be stated that the increase in concrete compressive strength due to confinement is only an issue in heavily loaded columns. In beams, the increase in bond and shear strength are of greater importance.

The most obvious confinement effect provided by the steel jacket was the containment of the core, preventing the concrete from spalling and falling away. The concrete was kept in place throughout the testing sequence; even in the gap area the arching effect was sufficient to contain the concrete. With the aid of mechanical bond (friction), the failure zone was concentrated in the gap zone. A general bond failure between the beam reinforcement steel and the column concrete in the joint area was thus avoided.

7.7 RATING THE RETROFIT SCHEMES

Considering the efficiency of the retrofit schemes, all the specimens behaved satisfactorily, with a slightly better performance being exhibited by the circular retrofits. The square retrofits did, however, behave well enough to merit consideration as a reasonable retrofit scheme. Evidently, the increase in the moment capacity provided by the concrete-to-steel bond, and the containment provided by the jacket, regardless of its geometric shape, were the overriding factors in the improvement of ductility. Placing the gap closer to the joint helped the effectiveness of the jacket as designed.

RETRO-CD provided the greatest retrofit improvement, with RETRO-SD a close runner-up. Failure was observed to be entirely within the retrofit region of these two specimens. RETRO-SU and RETRO-CU were affected by failure modes outside the retrofit region, and their performances should thus not be compared to RETRO-CD and

RETRO-SD. RETRO-CU developed a larger ductility than RETRO-SU, but was hampered by the failure in the column under negative loading. RETRO-SU developed reasonable ductility under positive loading, until flexural and shear failure outside the retrofit zone under negative loading took place.

CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 SUMMARY

The pre-1971 Canadian design standards for reinforced concrete construction lacked specific guidelines for seismic design; for example, details relating to ductility did not exist in the CAN3-A23.3-M66 (1966) standard. As a result, structures that were designed and built before changes in the 1970's are poorly detailed for ductility, which may have dire consequences in the event of a major earthquake. The primary focus of this study centered on the improvement of ductility of such deficient reinforced concrete frame structures, especially the ductility of the beam-to-column joint area.

The purpose of this study was to determine the suitability of a possible retrofit method to strengthen and improve the ductility of damaged or deficient beam-to-column joints. The retrofit method considered involved the use of a steel jacket to encase the joint; the gap between the steel jacket and concrete core was filled with cement grout. Two shapes of retrofit jacket were tested in this study: a circular steel jacket and a square steel jacket.

The test specimens was based on a structure of about half standard size designed according to the Canadian building code (NBCC, 1970) and concrete standard (CSA, 1966). The specimens themselves consisted of a subassembly of two column halfs and half a beam. The two initial tests of the reinforced concrete beam-to-column joint specimens confirmed that the detailing for ductility was inadequate. These specimens exhibited poor behaviour under cyclic loading.

The joint specimens were then retrofitted with a 3.0 mm thick steel grout filled jacket. Four retrofitted specimens were tested: two were the initial previously tested and damaged specimens, the other two were fresh specimens. The two damaged specimens were retrofitted with a square and circular jacket, and the same was done for the two undamaged specimens.

8.2 CONCLUSIONS

In all the retrofitted specimens the joint area itself was sufficiently strengthened to deflect failure to adjacent areas. When considering overall behaviour, each specimen exhibited an improvement in ductility, although only two of the specimens actually developed a failure mode entirely within the retrofit region. Even the two specimens which developed partial premature failures outside the retrofit jacket exhibited improved ductility behaviour in the retrofit region where hinging took place within the retrofit gap. It was generally observed that a circular jacket retrofit was more beneficial than a square jacket retrofit. In practical terms, however, the difference between the two shapes was not significant.

A side effect of this retrofit method was the increase in moment capacity of the sections. This was a result of a composite action between the concrete and steel jacket, as well as of a probable increase of the compressive strength of the core concrete. This increase in the moment capacity should be minimized to avoid undue distress in other regions of the structure that have not been designed for such large moments. To limit the increase in moment capacity of the joint area, it is recommended that retrofit gaps be placed close to the beam-to-column joint area. A number of gaps may be advisable to avoid concentrated yielding and thus reduced ductility of the reinforcement bars within the gap areas. For the dimensions of the specimens in this study three gaps seemed to be sufficient. When using square jackets, care should be taken to avoid placing the gaps too close to the joint area. The joint area has been shown to be weak, and without the added benefit of the radial confinement provided by a circular retrofit jacket this joint area, which is effectively within the column, can be significantly affected by a plastic hinge forming too close to it.

On the other hand, placing the gaps closer to the joint area would ensure plastic hinging in the retrofit area and would avoid the need for an increase of the retrofit length to avoid failures outside the retrofit region. If this cannot be accomplished, the lengths of the sleeves must be increased in order to avoid failure at the end of the retrofit jacket, in either the beam or the column. A jacket thickness designed to replace the missing transverse steel, on a volume basis, was found to be sufficient. For practical reasons during construction (eg. welding) a minimum thickness would be advisable.

Since none of the retrofitted specimens failed in the same manner as the unretrofitted ones, no conclusions can be drawn pertaining to the effectiveness of the proposed retrofit method in the immediate joint area. Follow-up tests are necessary to determine the amount of strengthening and added ductility needed in that region.

CHAPTER 9

RECOMMENDATIONS FOR FURTHER STUDY

9.1 CLARIFICATIONS AND MODIFICATIONS TO THIS RETROFIT STUDY

A few of the material properties that were important to this study are still unclear, and some work should be undertaken to quantify their effects on the results obtained. Most importantly is the strength of the composite action or bond between the retrofit steel and the concrete under bending. Another important material property that should be determined is the increase of concrete compressive strength under bending action in the beam as well as in the joint area.

Some of these effects can be examined through a retrofit study which is similar to the one described in this thesis, but having more instrumentation. Additional placement of strain gauges in sensitive areas of the retrofit jacket, such as in the joint and gap areas, would go a long way toward determining the strain patterns within the steel jacket. A set of strain gauges placed on the longitudinal reinforcement bars would also help to determine when and where yielding is taking place within the concrete reinforcement steel.

Scale factors could also make a difference. The sizes of the specimens in this study were limited by physical constraints of equipment and space in the Structures Laboratory at the University of British Columbia. Specimens of nearly full scale would be desirable, as the scales of the specimens and the retrofit jackets would not necessarily increase uniformly. A set of specimens should be tested in which all failure modes take place within the retrofit area. This would require that the specimens be designed with the gaps suitably located. A finite element analysis of a model could be conducted to predict the behaviour of the retrofitted joint once the requisite material properties have been determined.

9.2 BEYOND THE SIMPLE BEAM-TO-COLUMN JOINT

The retrofit scheme proposed seems promising, as it achieved the goal of improving the ductility capacity of beam-to-column joints which were originally poorly detailed for ductility. However, the simplicity of the tested joint has only limited usefulness. Columns with only a single framing beam are rarely found, and then usually only in bridges. Cyclic lateral loading is the simplest of possible motions that may affect a joint of this type. In the case of a bridge structure, the motions during an earthquake may be much more complex, and may include motions parallel to the bridge deck, which would apply torsional motion as well as lateral motion at 90 degrees to that observed in this study. To determine these effects, a scale model of a single or double bent frame should be tested on a shake table.

Beam-to-column joints in other structures tend to be of greater complexity. Exterior joints also have transverse beams framing into the column, and interior joints have four beams joining at the column. Modifications to this study to include effects of transverse beams are straigtforward enough, but require a larger laboratory set-up. Of much greater importance is the presence and effect of slabs in a reinforced concrete structure. Most buildings using this type of construction incorporate columns, longitudinal and transverse beams, as well as slabs. The presence of slabs significantly increases the complexity of the retrofit problem. Confinement of the column in the joint region can still be achieved with the presence of a slab, but the slab interferes with the retrofit of the beam, and a reasonable plan must take into account the presence of the slab in the joint area.

The results of this study indicate promising possibilities in the use of steel jacket retrofits for the improvement of ductility of reinforced concrete beam-to-column joints. The above recommendations can be used to extend this study, and to determine the usefulness of this retrofit method for various kinds of reinforced concrete construction.

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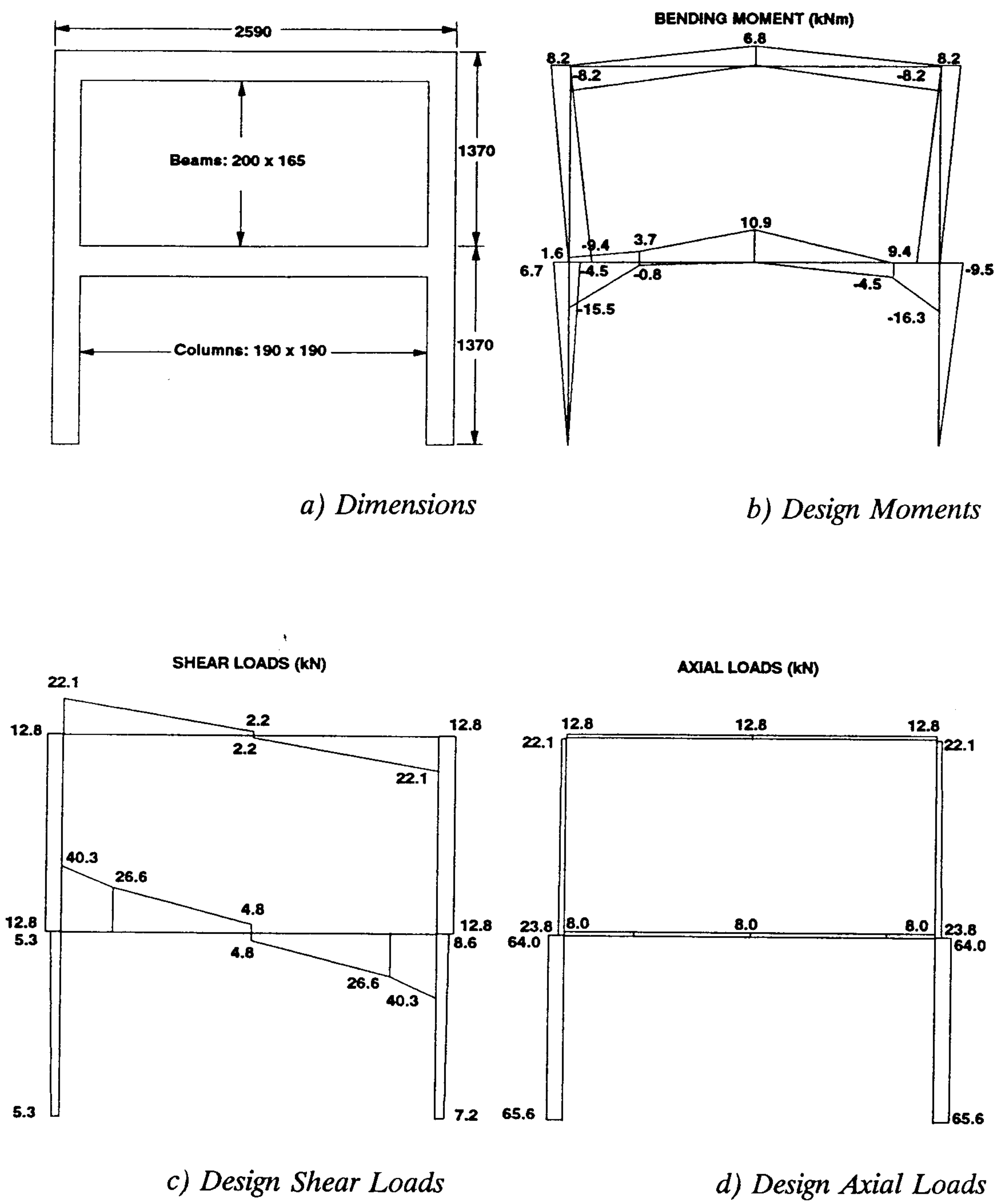
APPENDIX A

STRUCTURE AND SPECIMEN DESIGN

The structure was designed according to the 1970 National Building Code of Canada and the CAN3-A23.3-M66 Standard for Reinforced Concrete Design (1966).

This appendix presents a summary of the design bending moments, axial loads and shear loads of the structure (fig. A-1), as well as the design specifications for the specimen used in the tests (fig. A-2).



a) Dimensions

b) Design Moments

c) Design Shear Loads

d) Design Axial Loads

Figure A-1 Frame Design

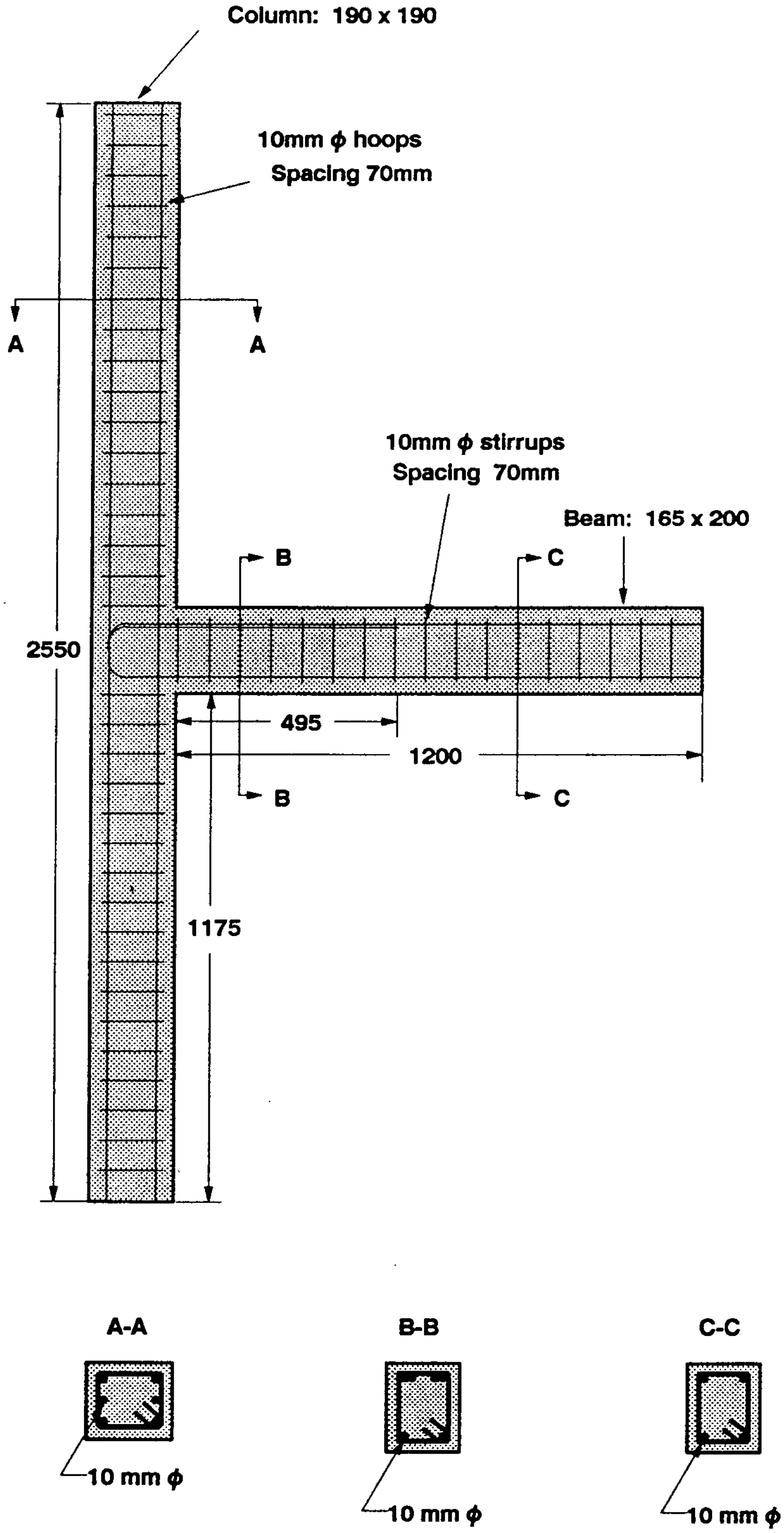


Figure A-2 Specimen Design

APPENDIX B

MATERIAL PROPERTIES

Table B-1 presents the compressive strengths of the concrete and grout as well as the tensile strengths of the reinforcing steel and the retrofit steel used in the tests. Figures B-1 to B-6 show the tensile coupon test results obtained from the reinforcing steel and jacket steel. For the reinforcing bar specimens, the standard 0.2% offset strain method was used to determine a value for the yield strain used in the calculations.

Table B-1: Material properties of concrete and steel.

Concrete Strength: Unretrofitted specimens	Test #1	25.1 MPa
	Test #2	27.5 MPa
	Average	26.3 MPa
Concrete Strength: Retrofitted specimens	Test #1	29.3 MPa
	Test #2	31.3 MPa
	Average	30.3 MPa
Grout Strength	Test #1	29.1 MPa
	Test #2	28.3 MPa
	Test #3	38.1 MPa
	Test #4	29.8 MPa
	Average	31.3 MPa
Reinforcing Steel Yield Strength	Test #1	535 MPa
	Test #2	513 MPa
	Test #3	551 MPa
	Test #4	664 MPa
	Average	566 MPa
Retrofit Steel Yield Strength	Test #1	267 MPa
	Test #2	267 MPa
	Average	267 MPa

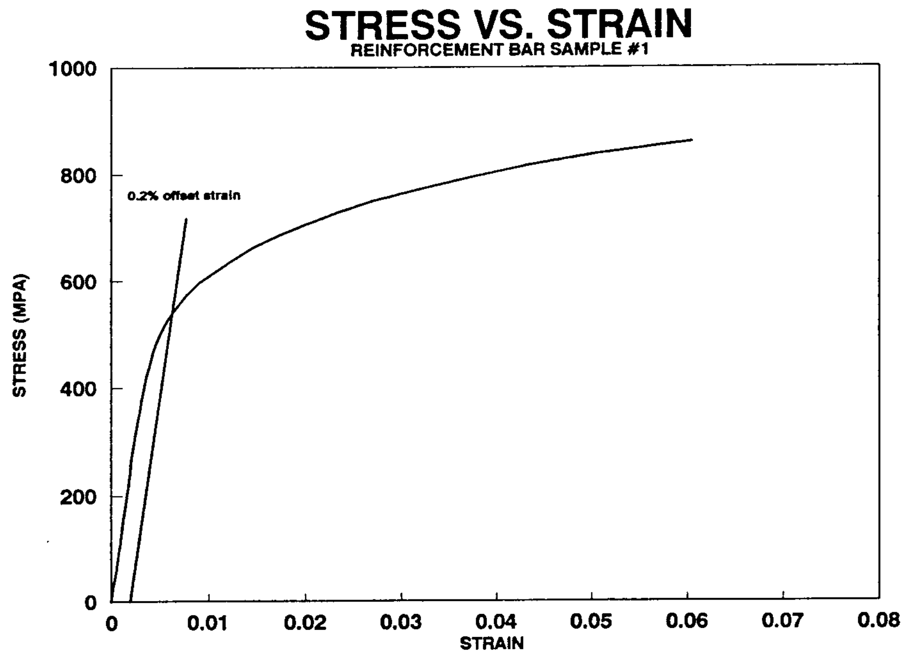


Figure B-1 Reinforcement Bar #1

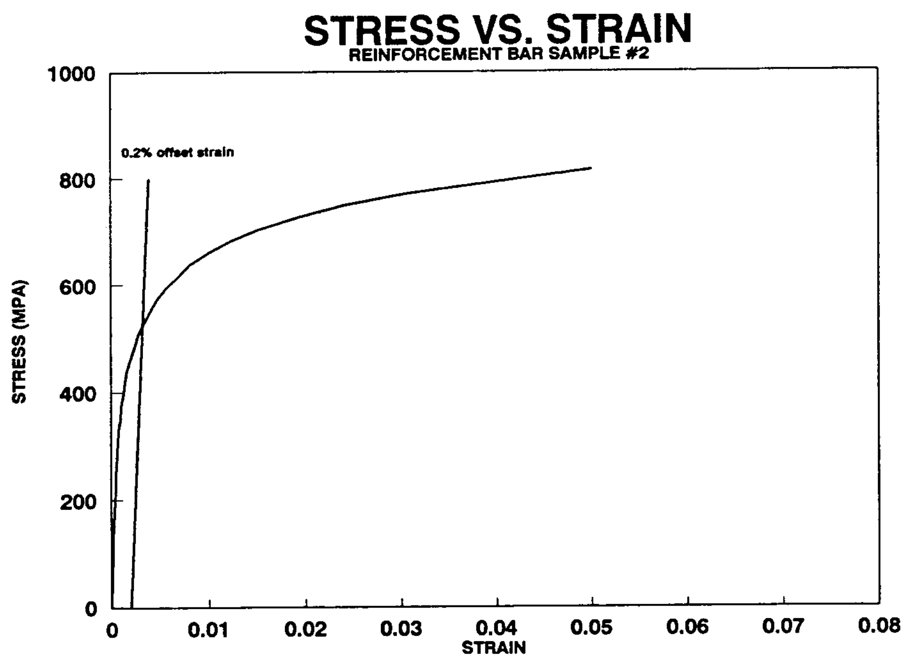


Figure B-2 Reinforcement Bar #2

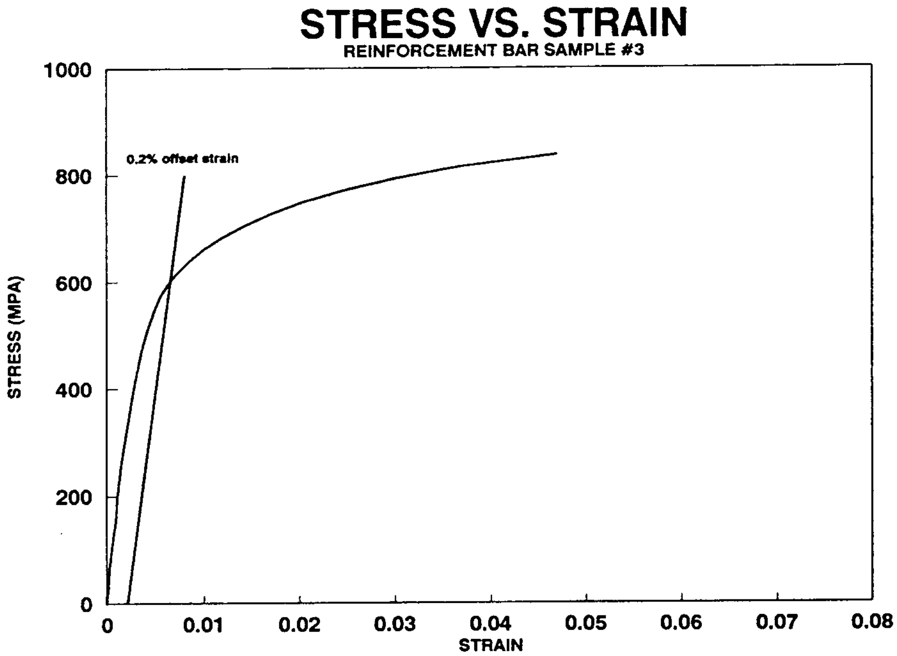


Figure B-3 Reinforcement Bar #3

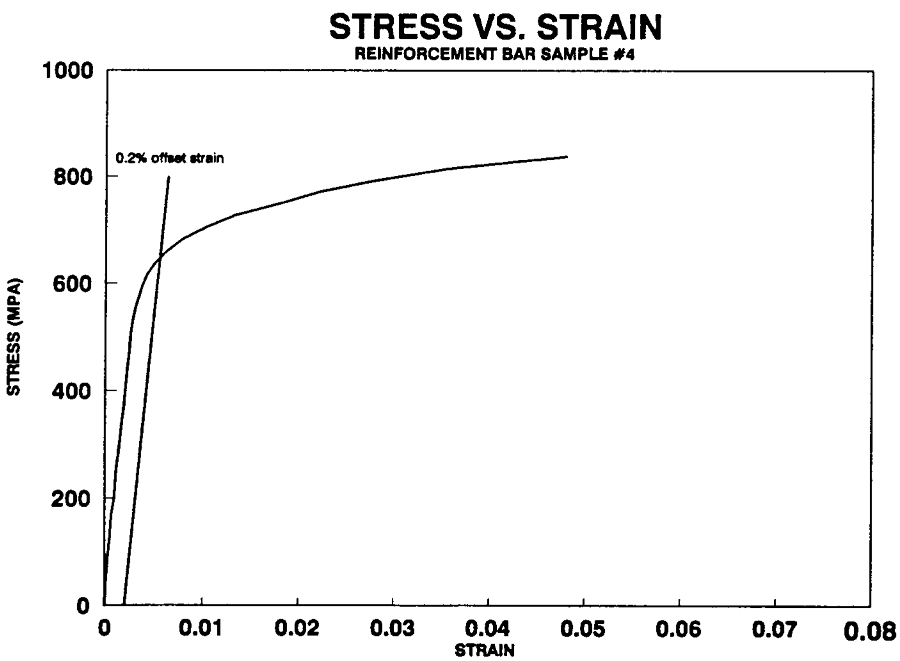


Figure B-4 Reinforcement Bar #4

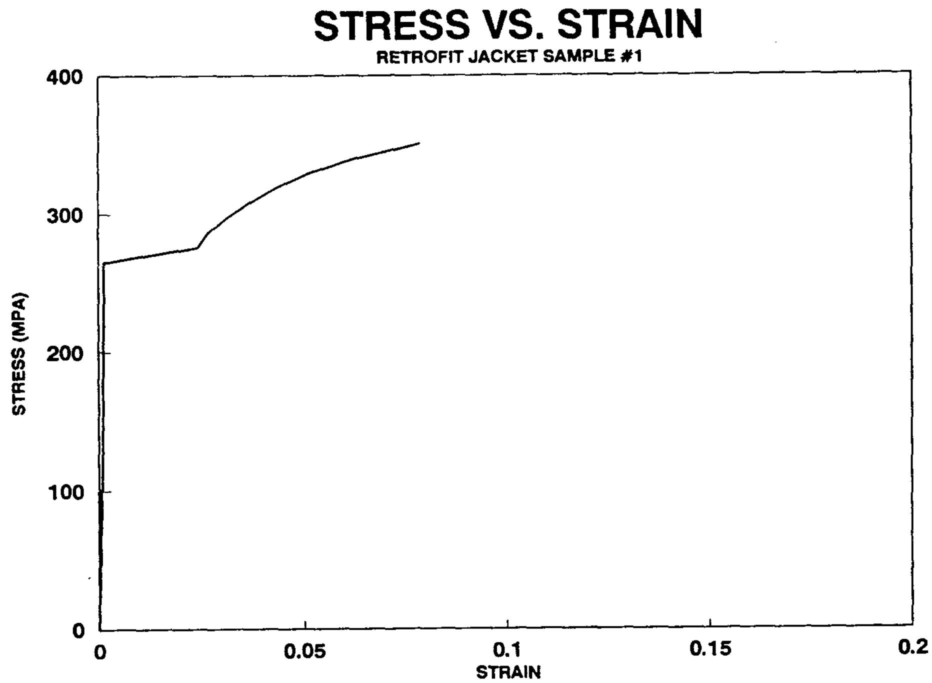


Figure B-5 Retrofit Jacket #1

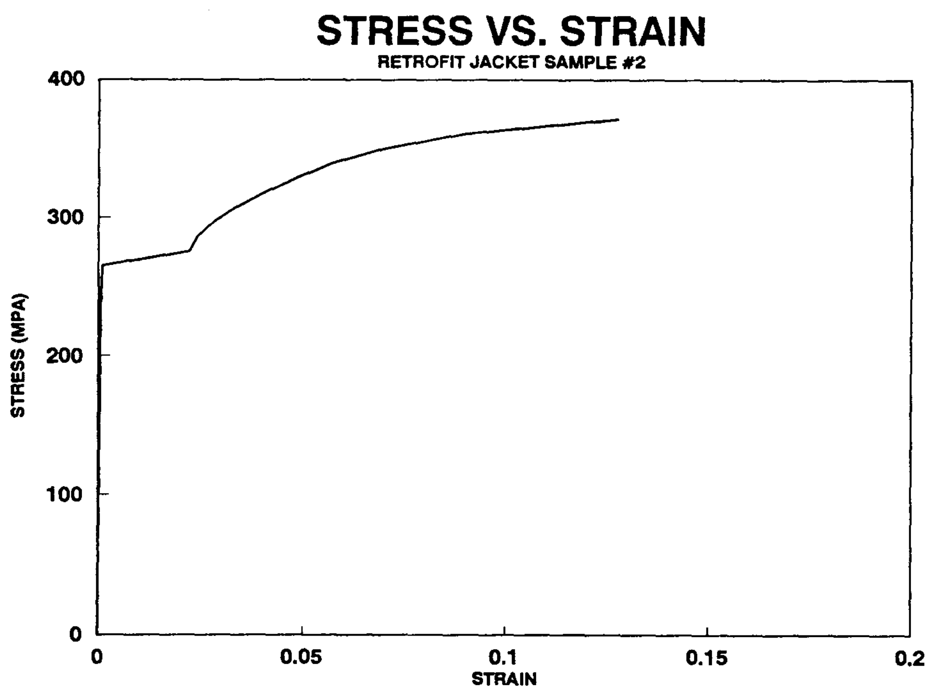


Figure B-6 Retrofit Jacket #2

APPENDIX C

DATA FOR UNRETROFITTED SPECIMENS

Appendix C illustrates all the data recorded for specimens RCBC1 and RCBC2, the two tests of the normal unretrofitted beam-to-column joint. The data are organized for each of the two data sets in the following order: total rotation, comparison of rotation components, cantilever rotation, rigid rotation, shear rotation and shear & rigid rotation.

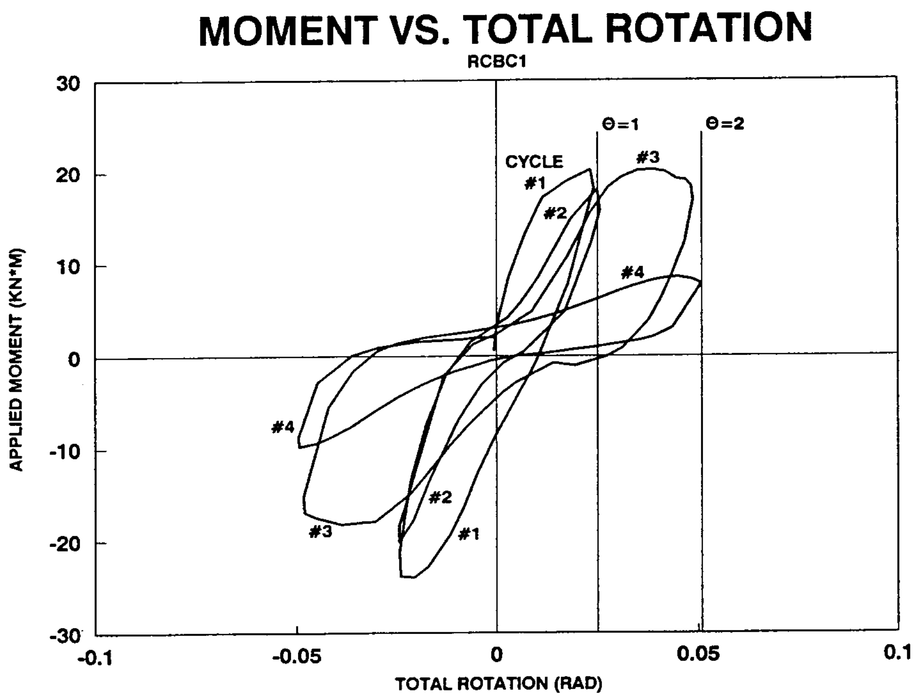


Figure C-1 RCBC1: Total Rotation

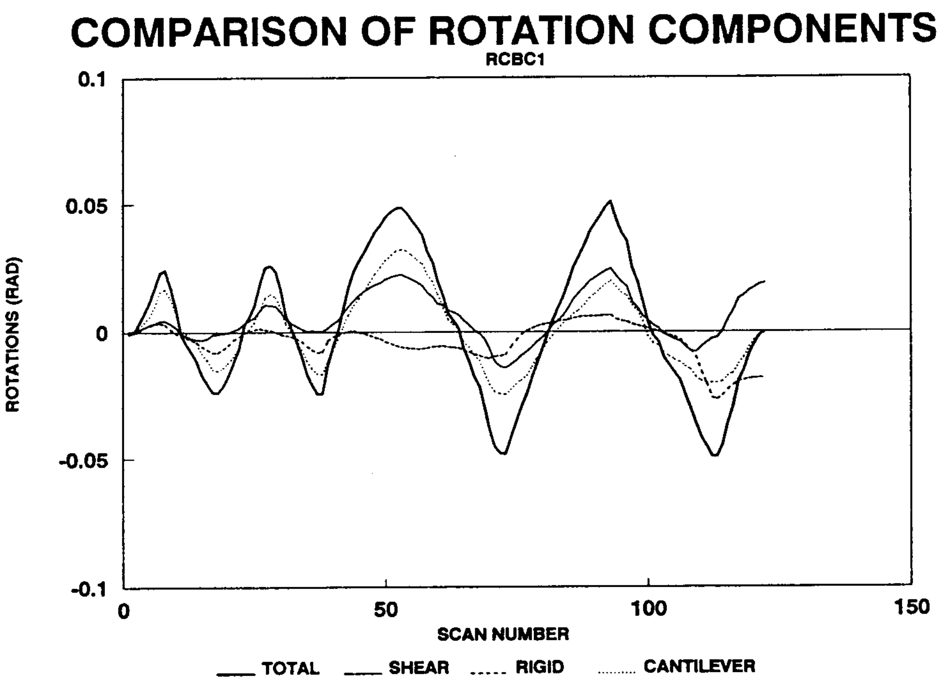


Figure C-2 RCBC1: Rotation Components

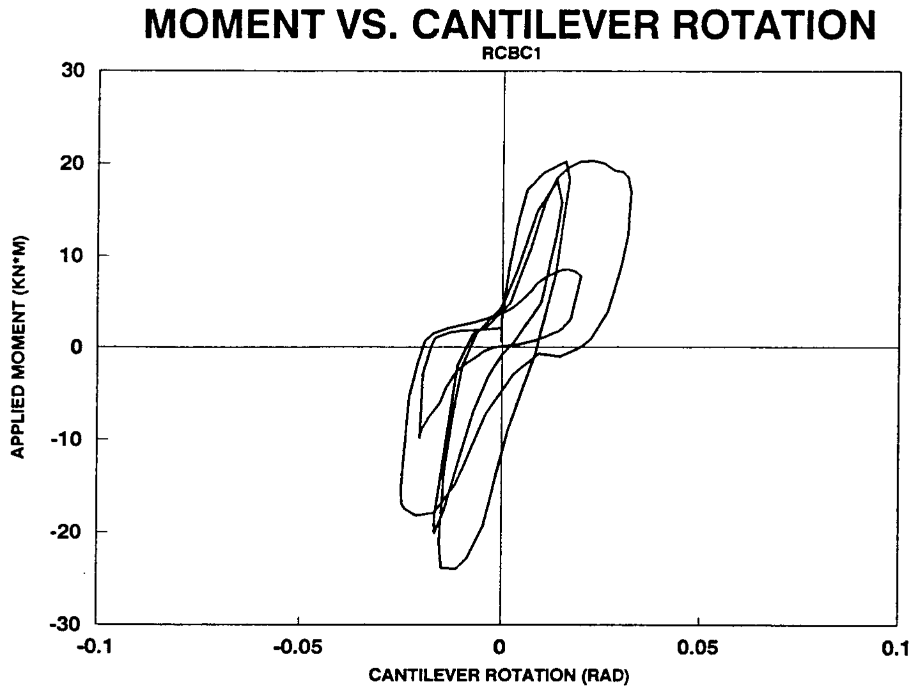


Figure C-3 RCBC1: Cantilever Rotation

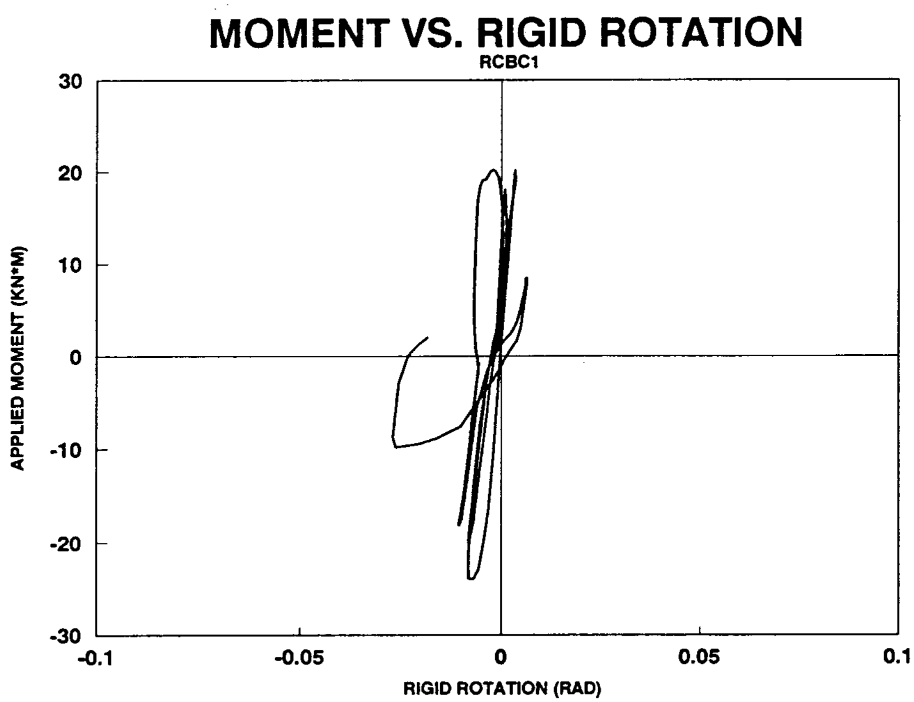


Figure C-4 RCBC1: Rigid Rotation

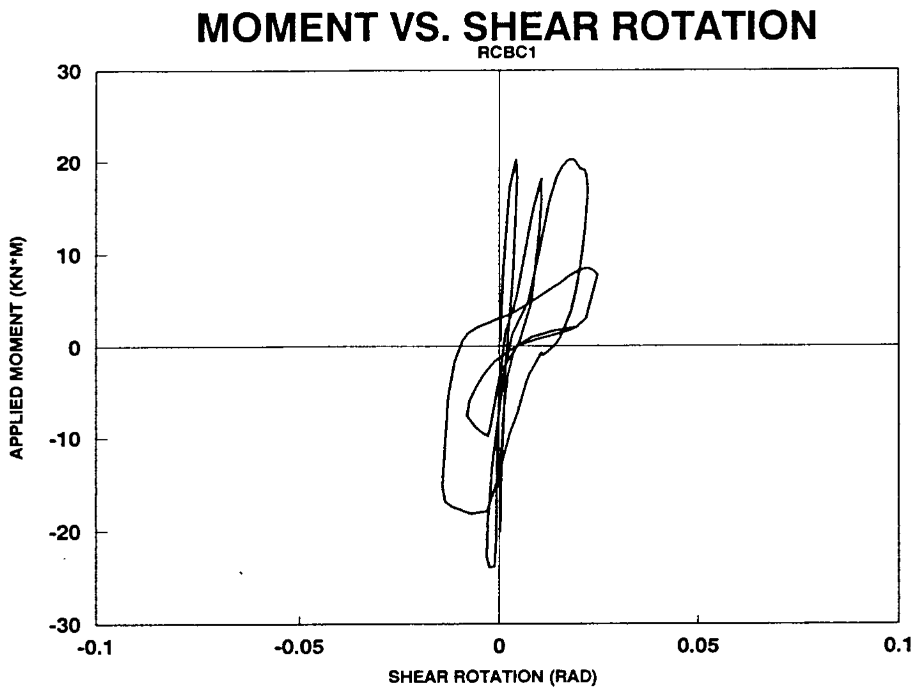


Figure C-5 RCBC1: Shear Rotation

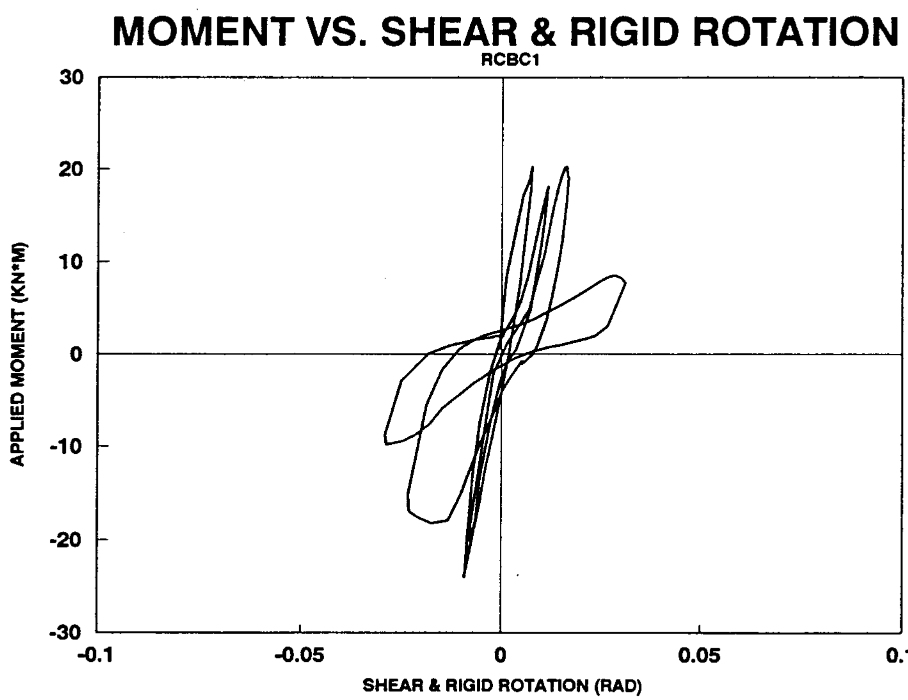


Figure C-6 RCBC1: Shear and Rigid Rotation

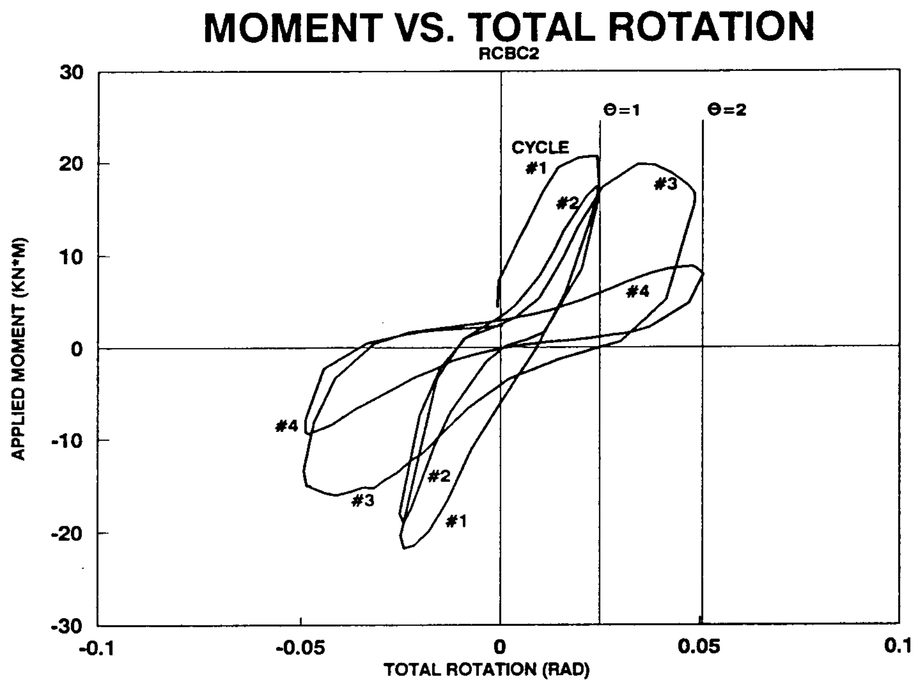


Figure C-7 RCBC2: Total Rotation

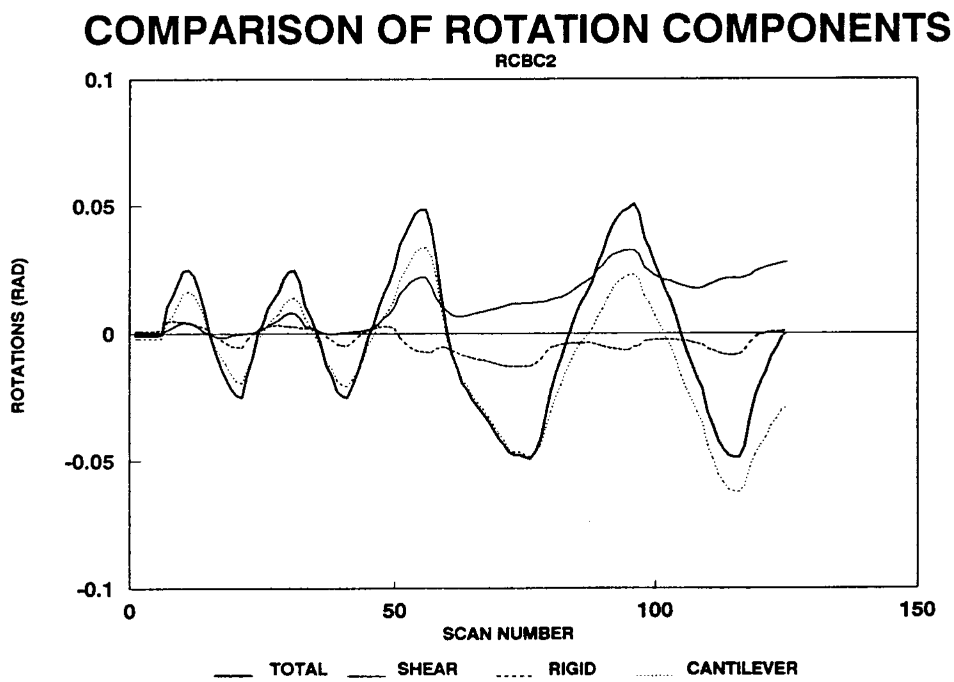


Figure C-8 RCBC2: Rotation Components

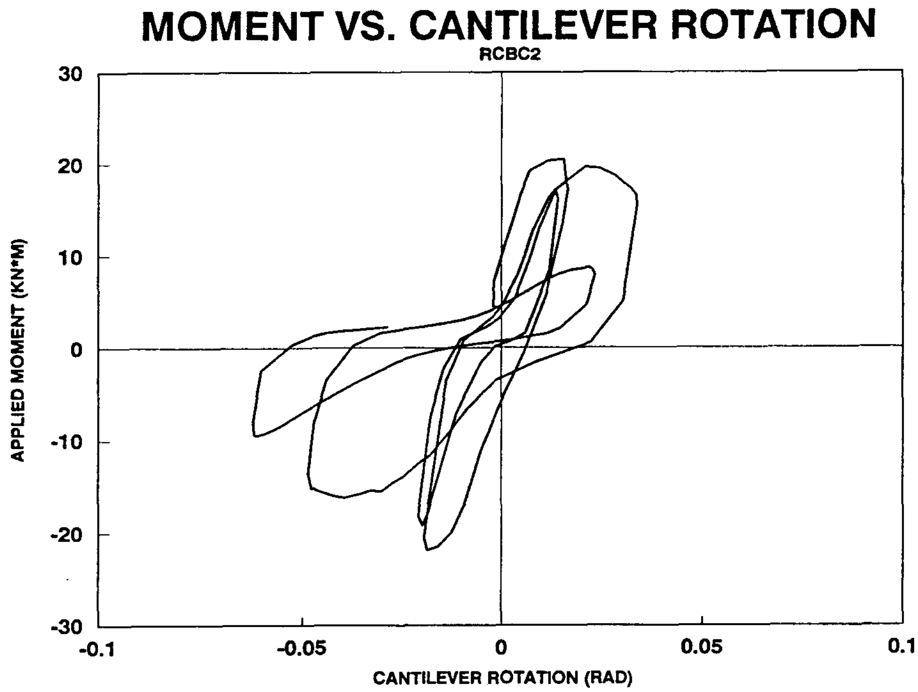


Figure C-9 RCBC2: Cantilever Rotation

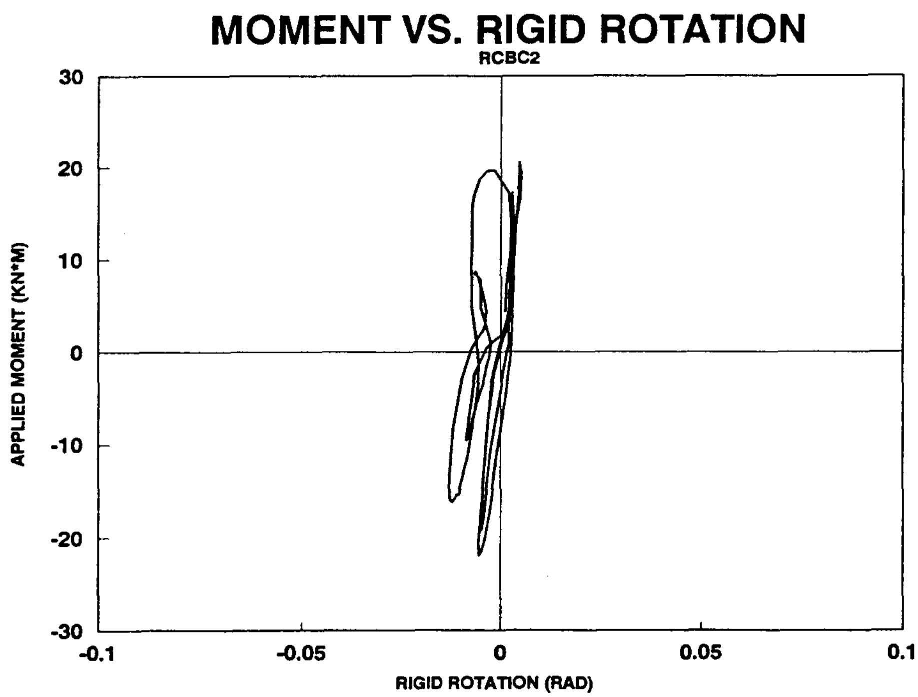


Figure C-10 RCBC2: Rigid Rotation

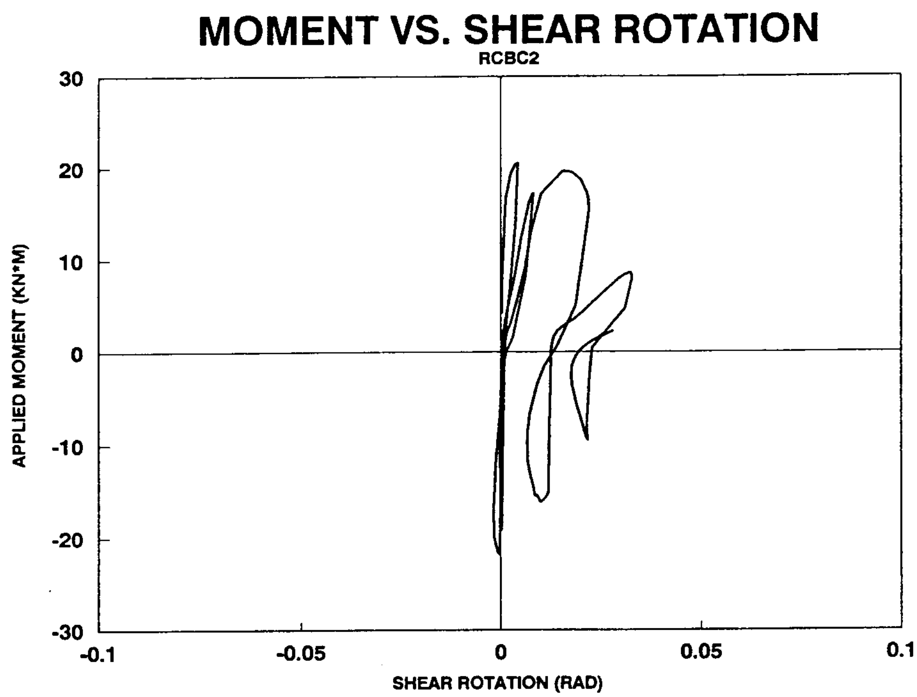


Figure C-11 RCBC2: Shear Rotation

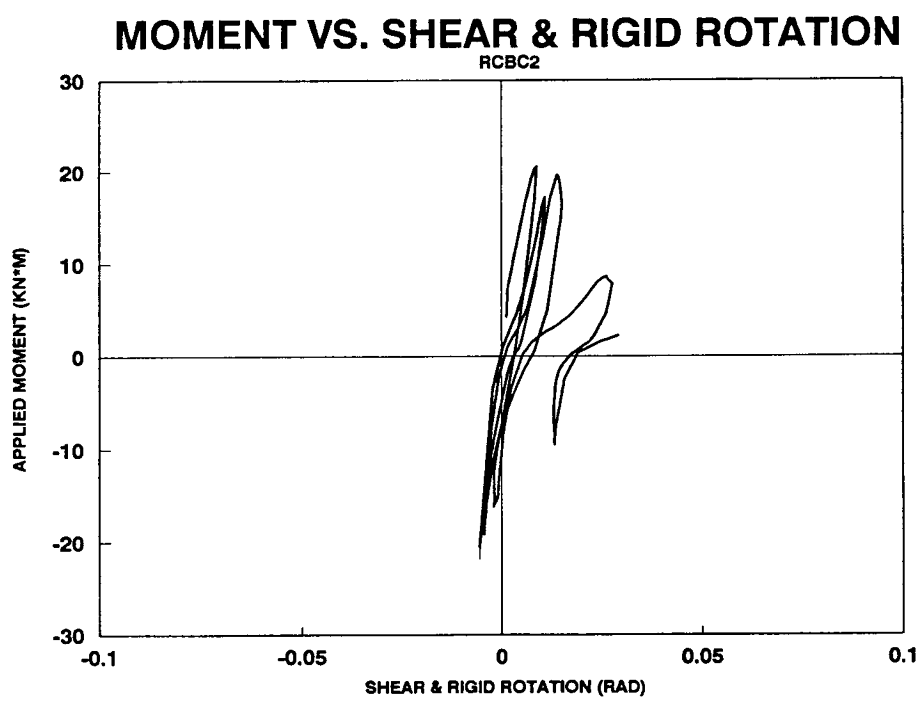


Figure C-12 RCBC2: Shear and Rigid Rotation

APPENDIX D

DATA FOR RETROFITTED SPECIMENS

All data recorded for specimens RETRO-SU, RETRO-CU, RETRO-SD and RETRO-CD are presented in this Appendix. The data are organized by specimen, and for each specimen the data are in the following order: total outer rotation, total inner rotation, hinge rotation, rotation components, cantilever rotation, rigid rotation, shear rotation, shear & rigid rotation and all of the strain gauge data.

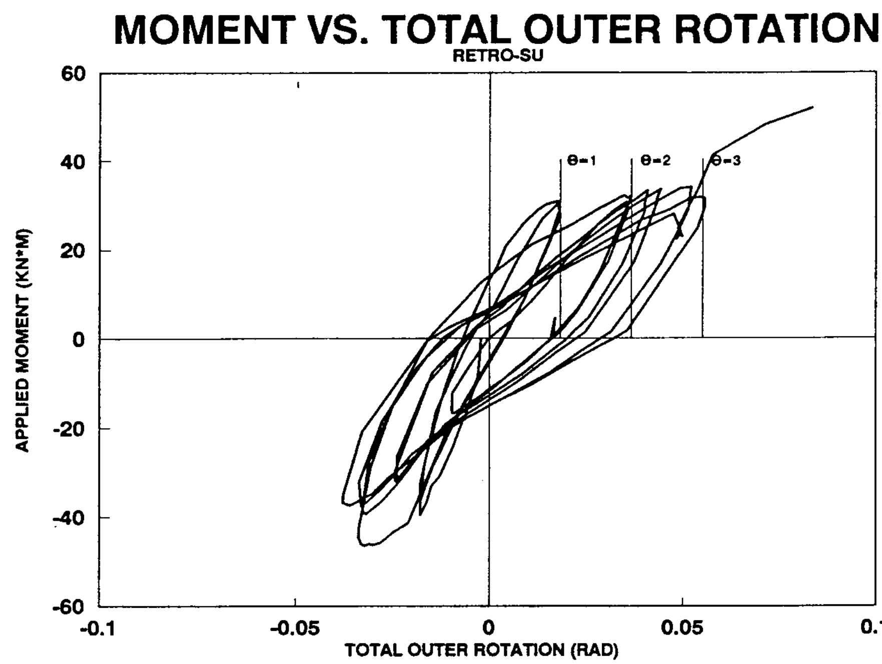


Figure D-1 RETRO-SU: Total Outer Rotation

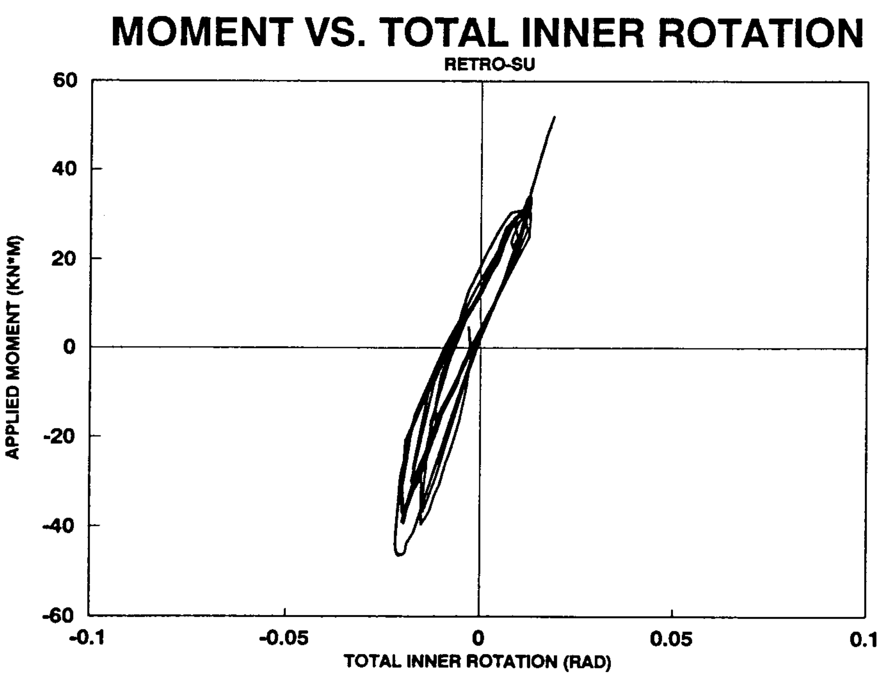


Figure D-2 RETRO-SU: Total Inner Rotation

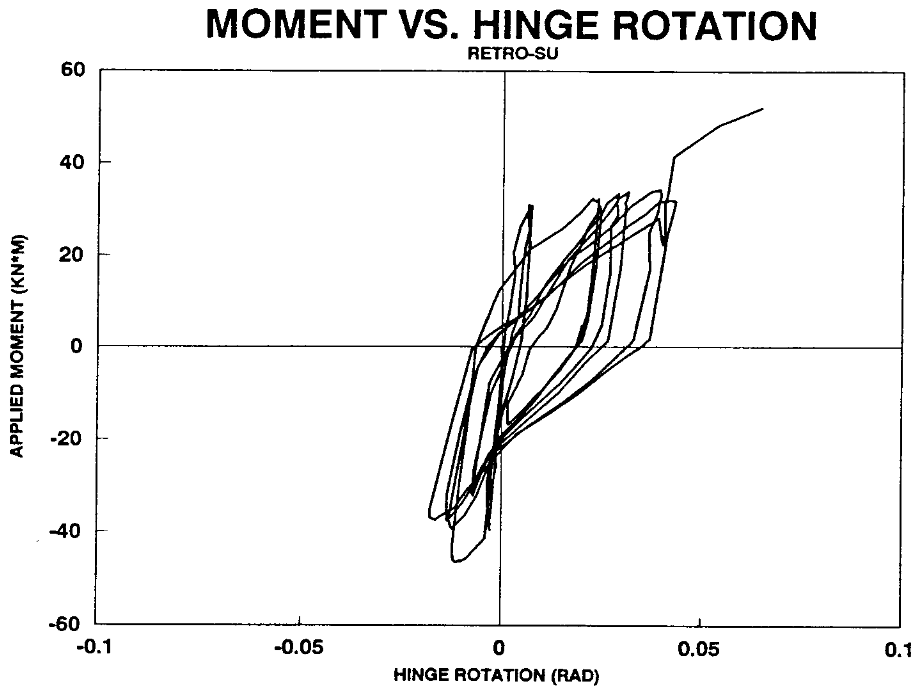


Figure D-3 RETRO-SU: Hinge Rotation at Gap

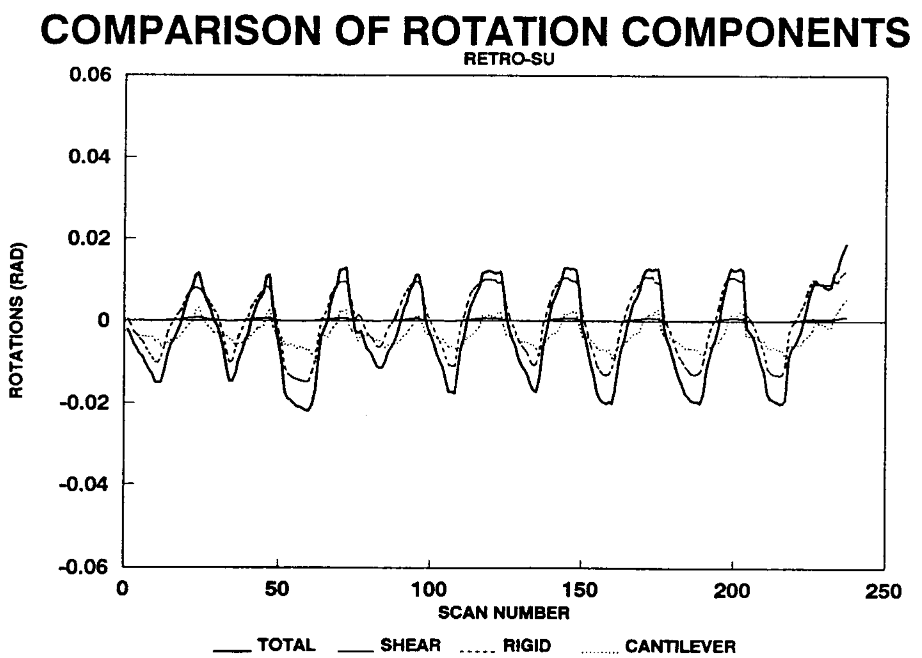


Figure D-4 RETRO-SU: Rotation Components

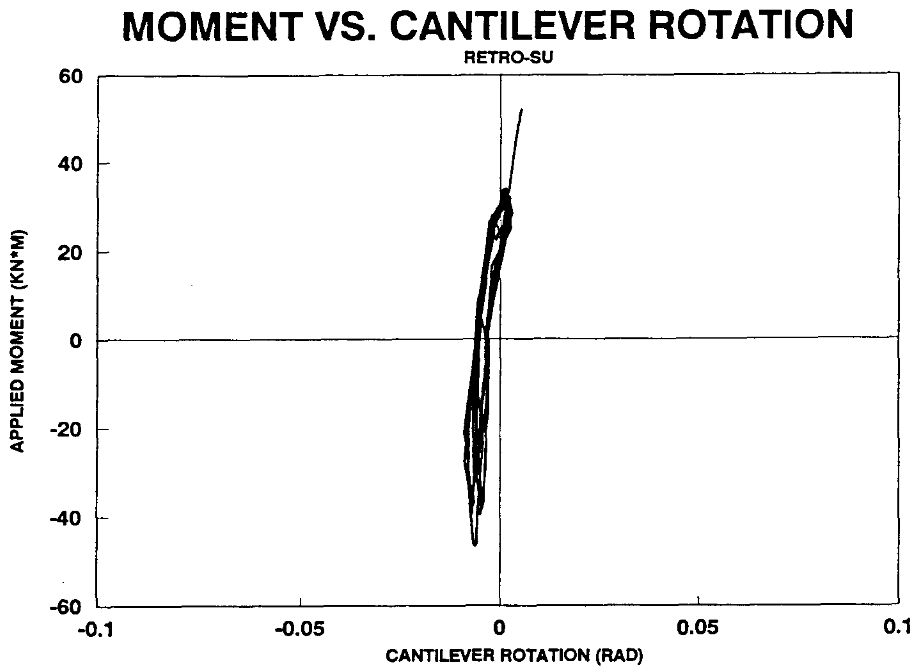


Figure D-5 RETRO-SU: Cantilever Rotation

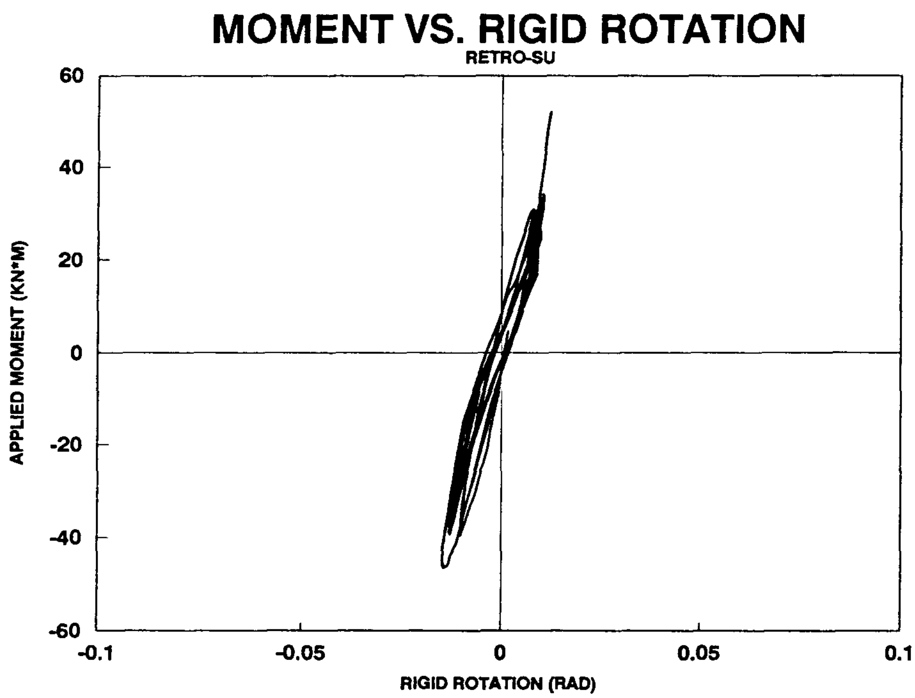


Figure D-6 RETRO-SU: Rigid Rotation

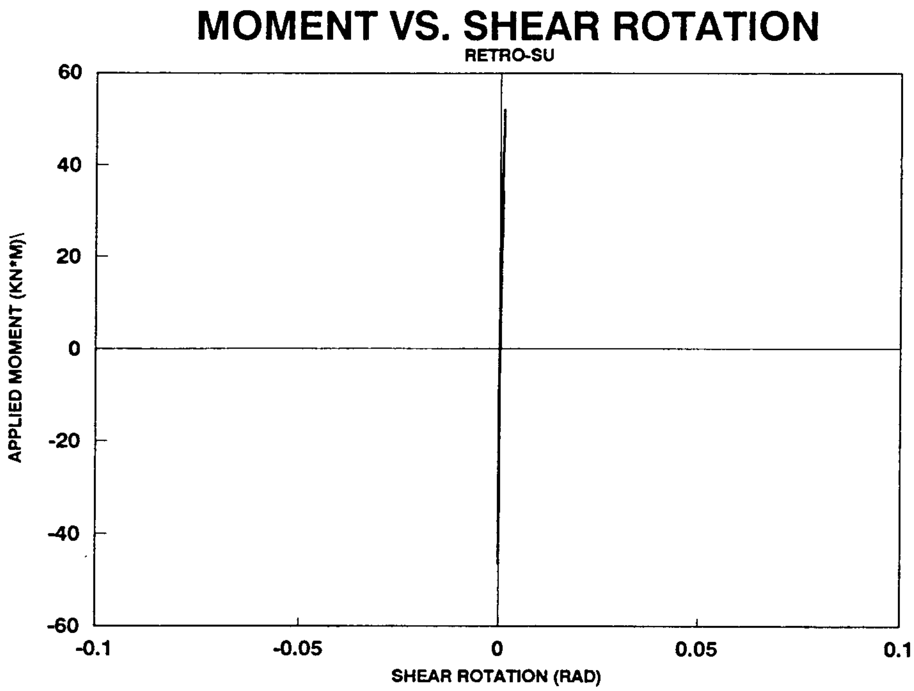


Figure D-7 RETRO-SU: Shear Rotation

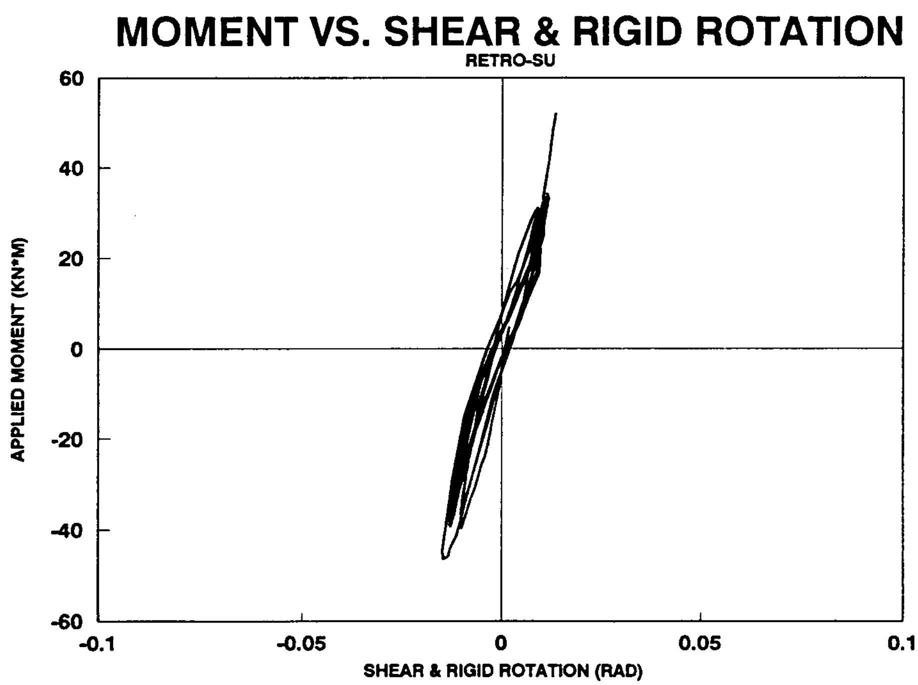


Figure D-8 RETRO-SU: Shear and Rigid Rotation

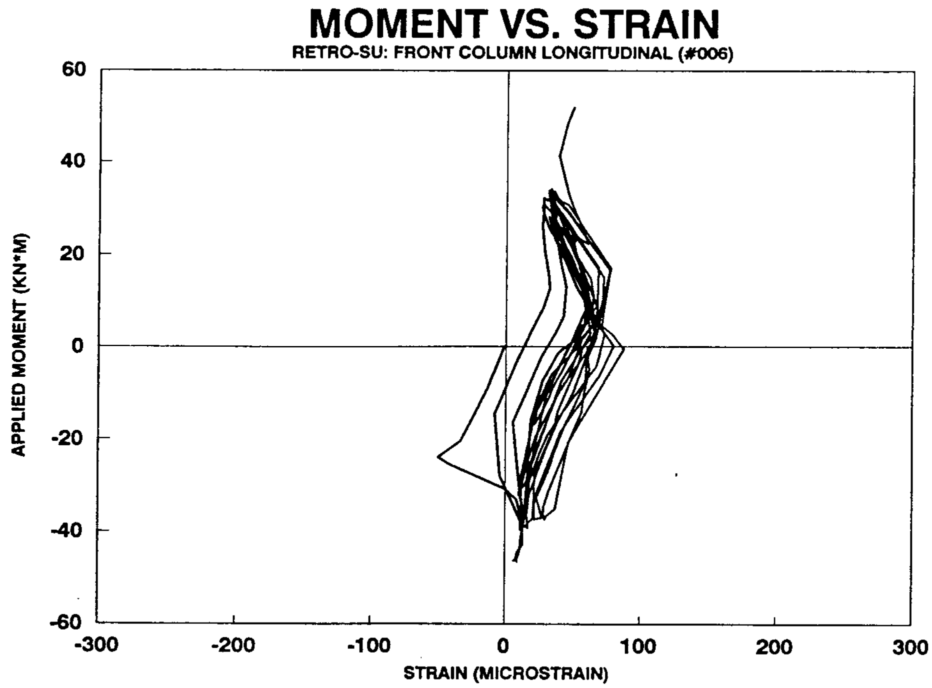


Figure D-9 RETRO-SU: Front Column Longitudinal (FCL)

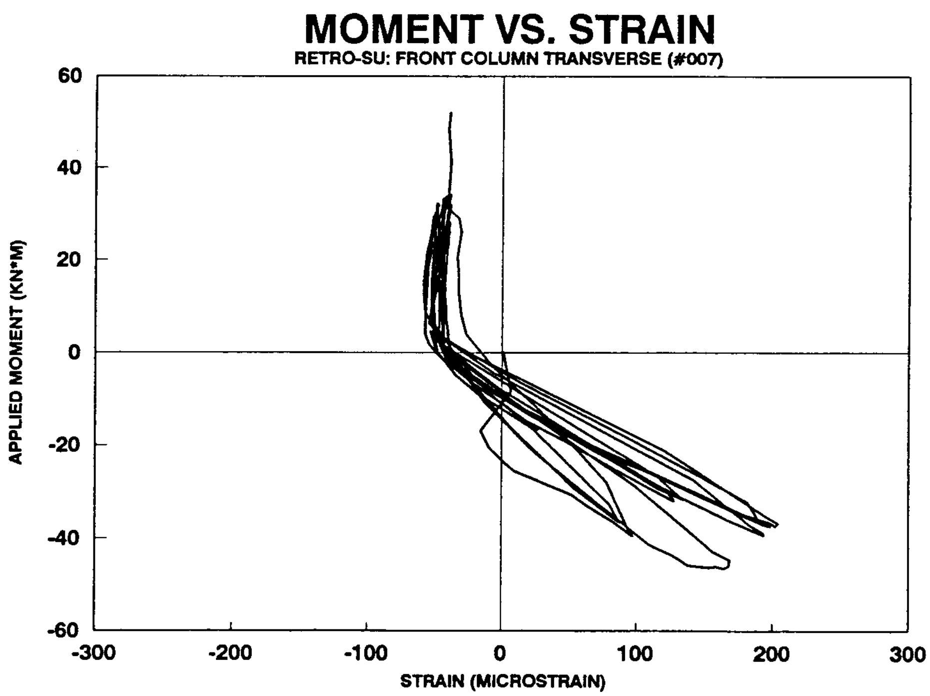


Figure D-10 RETRO-SU: Front Column Transverse (FCT)

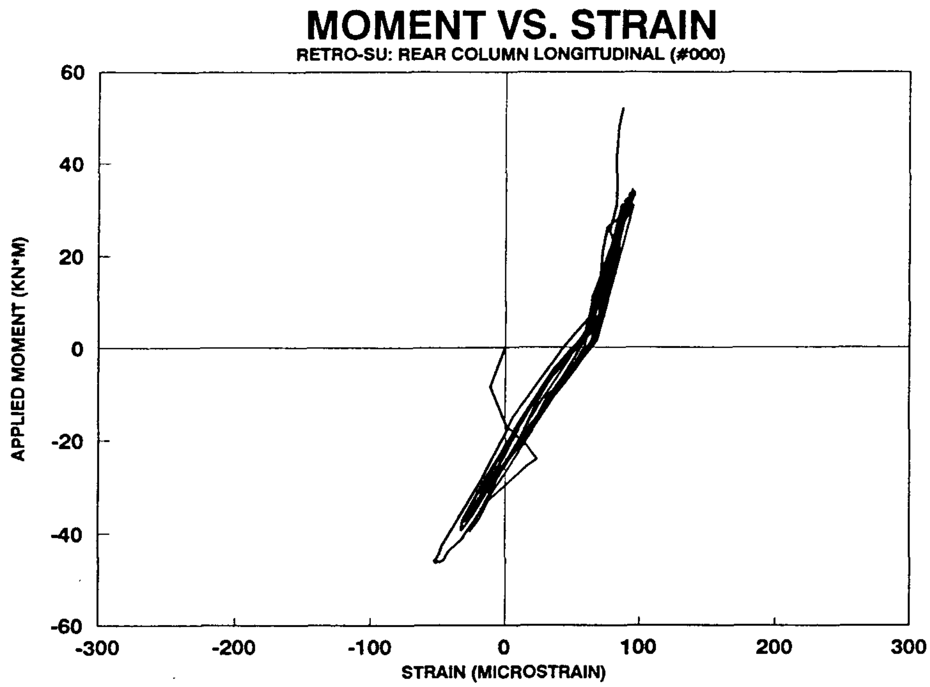


Figure D-11 RETRO-SU: Rear Column Longitudinal (RCL)

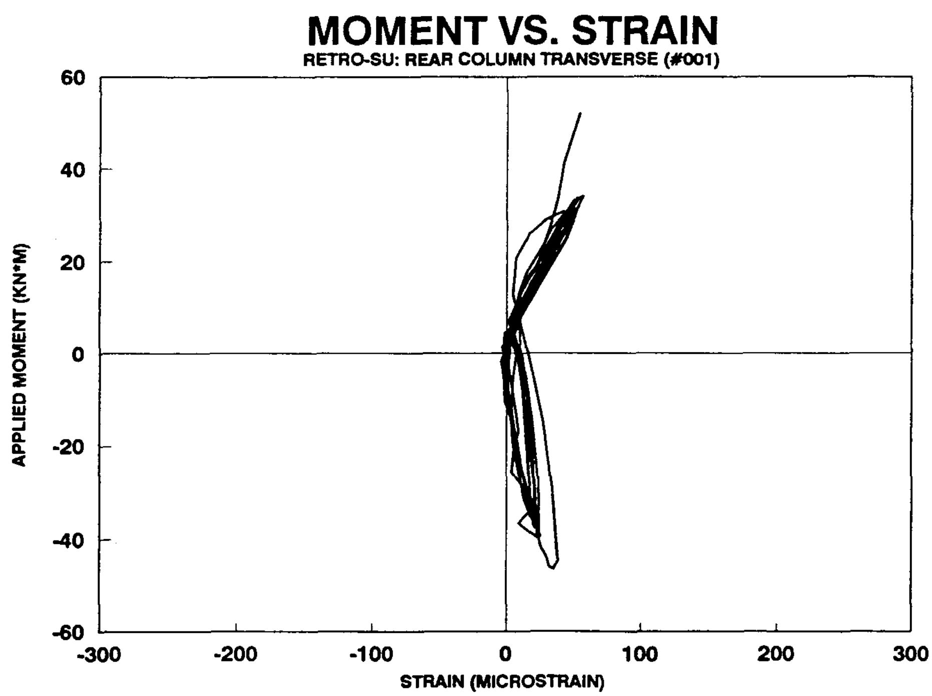


Figure D-12 RETRO-SU: Rear Column Transverse (RCT)

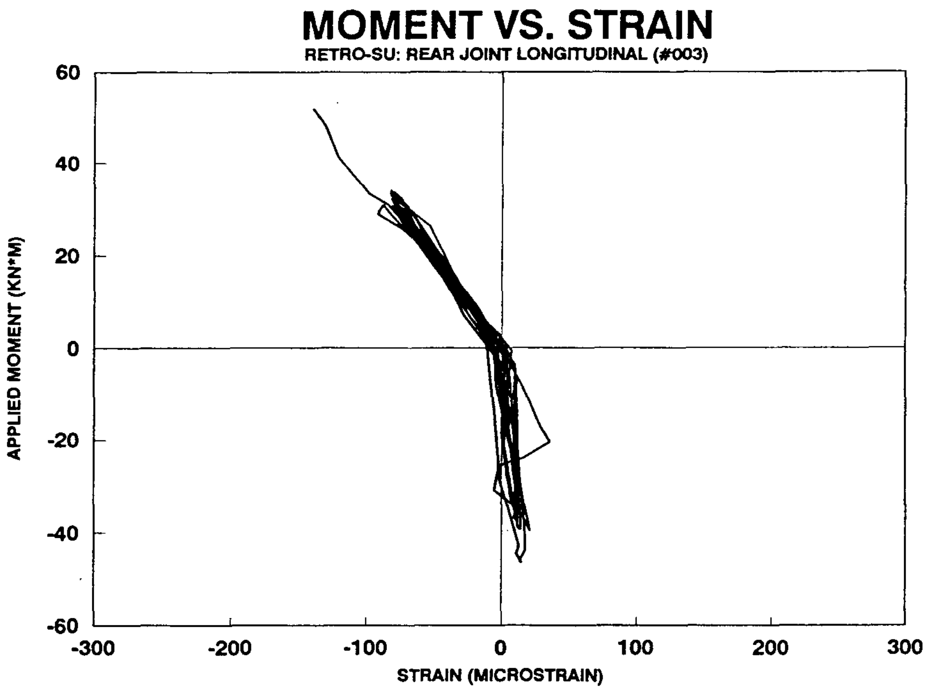


Figure D-13 RETRO-SU: Rear Joint Longitudinal (RJL)

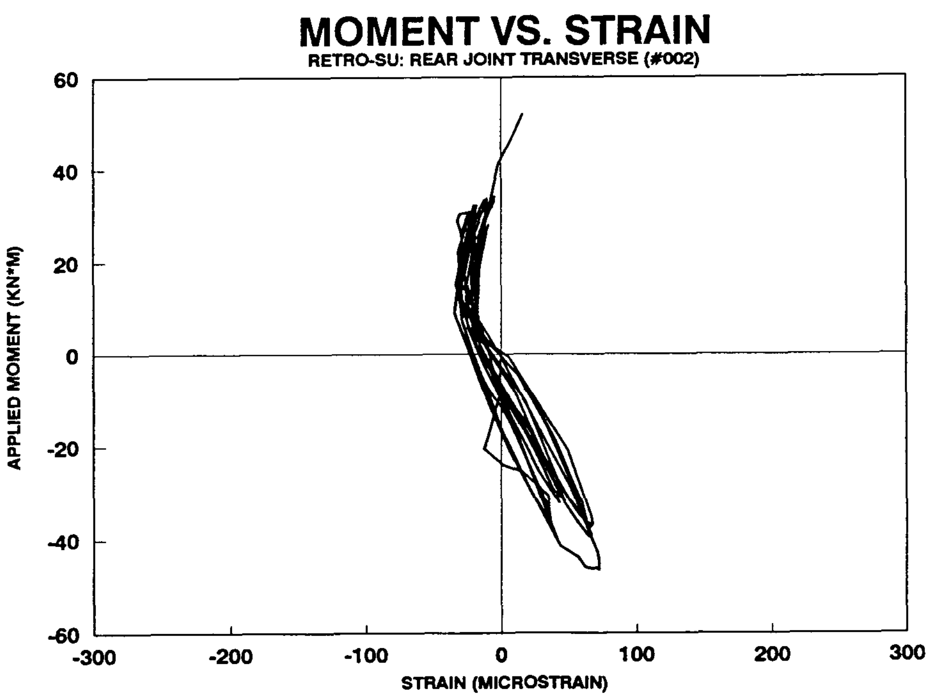


Figure D-14 RETRO-SU: Rear Joint Transverse (RJT)

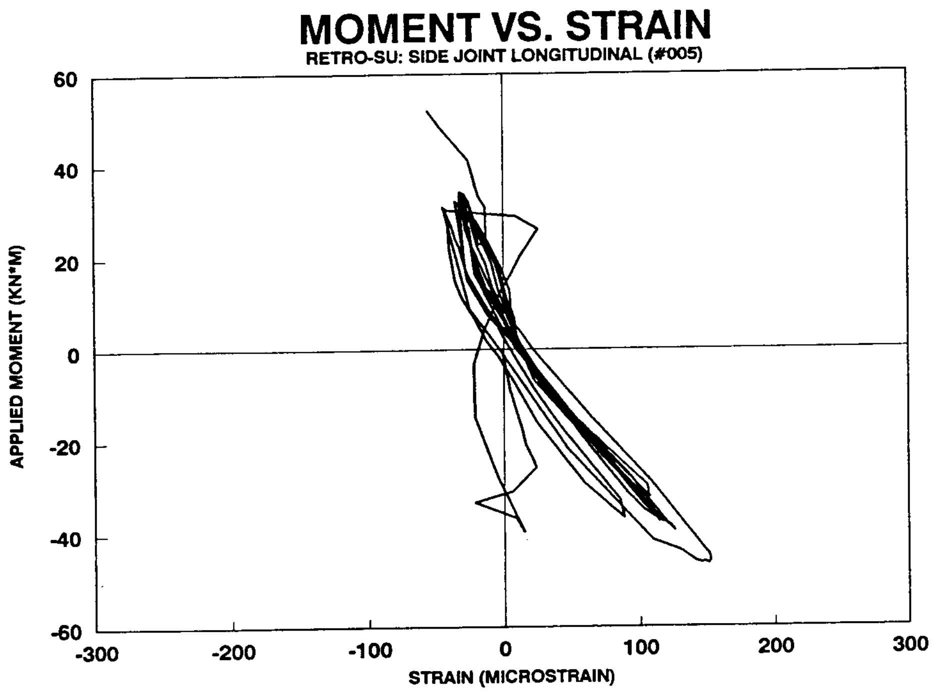


Figure D-15 RETRO-SU: Side Joint Longitudinal (SJL)

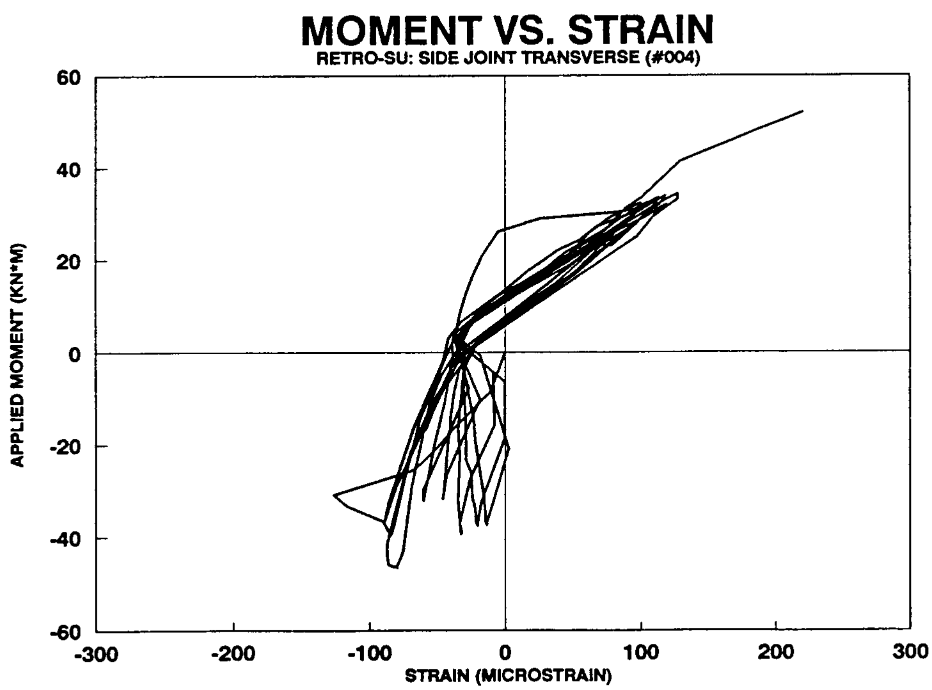


Figure D-16 RETRO-SU: Side Joint Transverse (SJT)

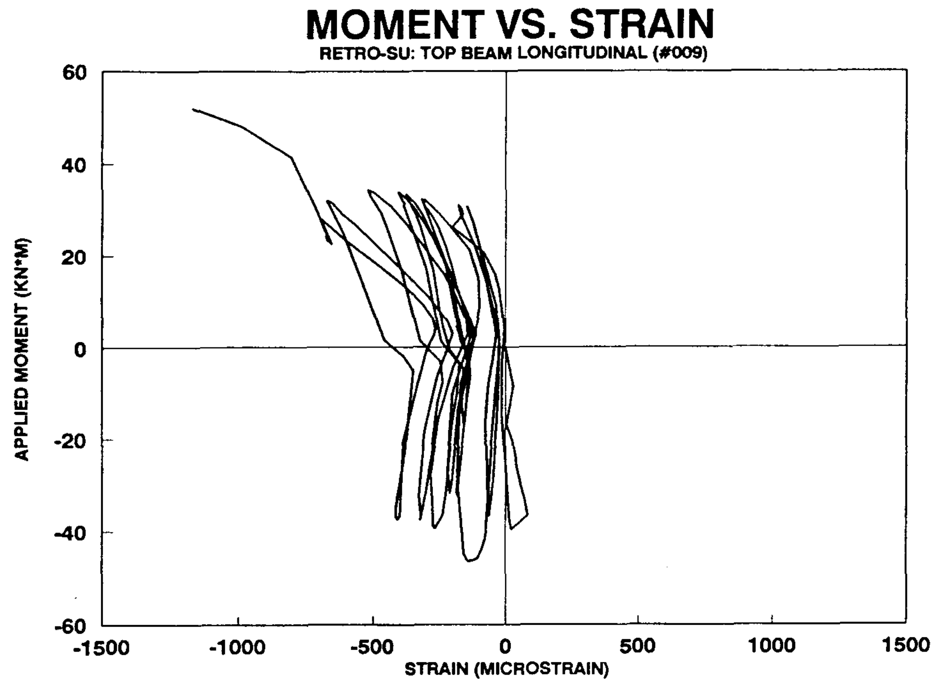


Figure D-17 RETRO-SU: Top Beam Longitudinal (TBL)

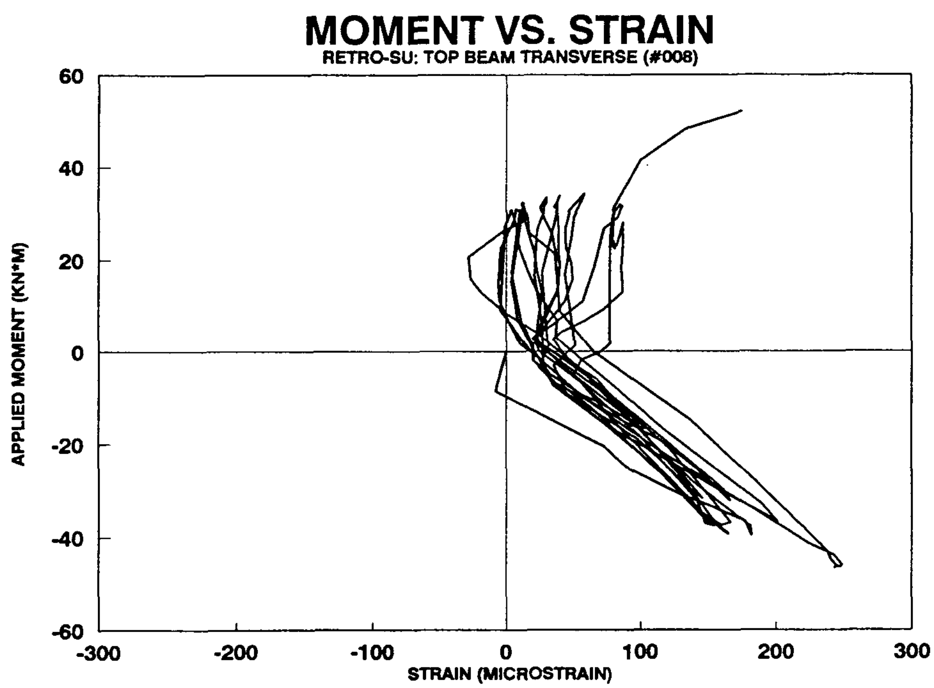


Figure D-18 RETRO-SU: Top Beam Transverse (TBT)

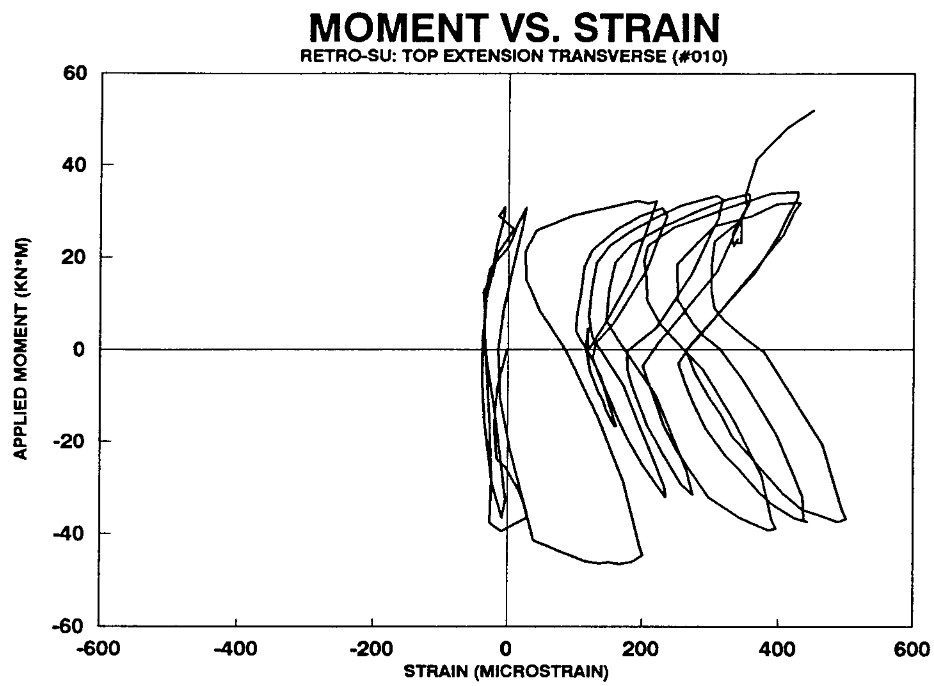


Figure D-19 RETRO-SU: Top Extension Transverse (TET)

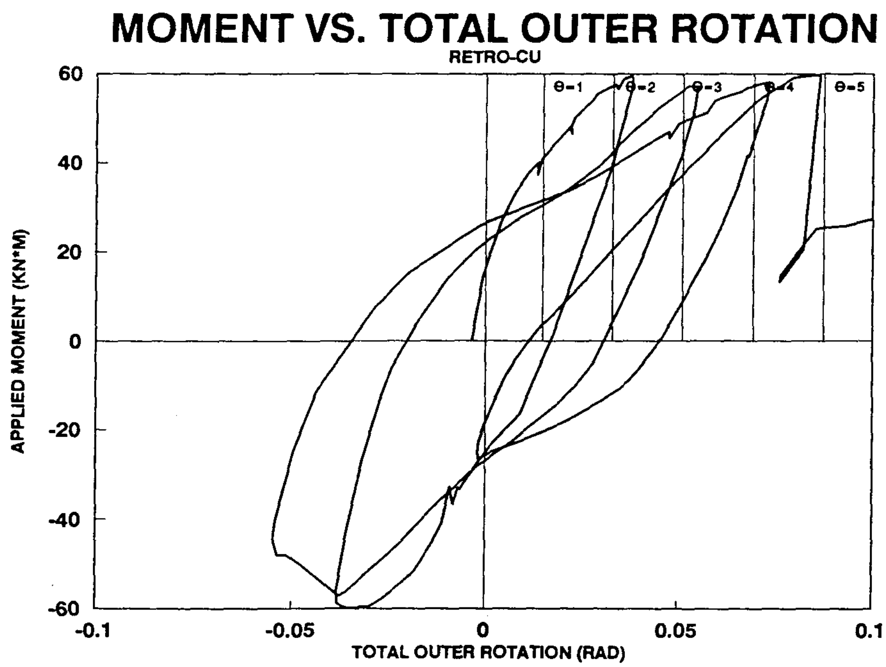


Figure D-20 RETRO-CU: Total Outer Rotation

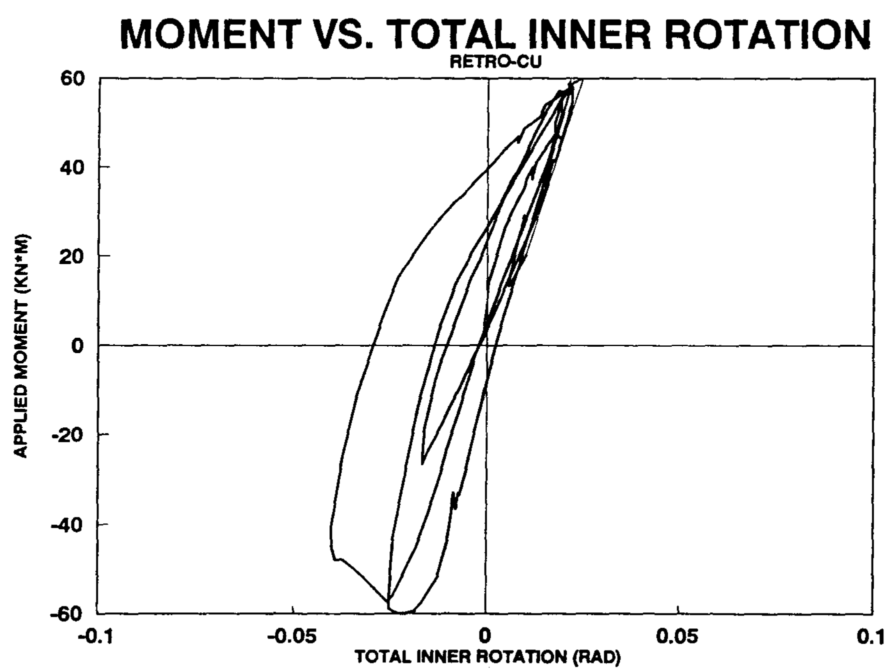


Figure D-21 RETRO-CU: Total Inner Rotation

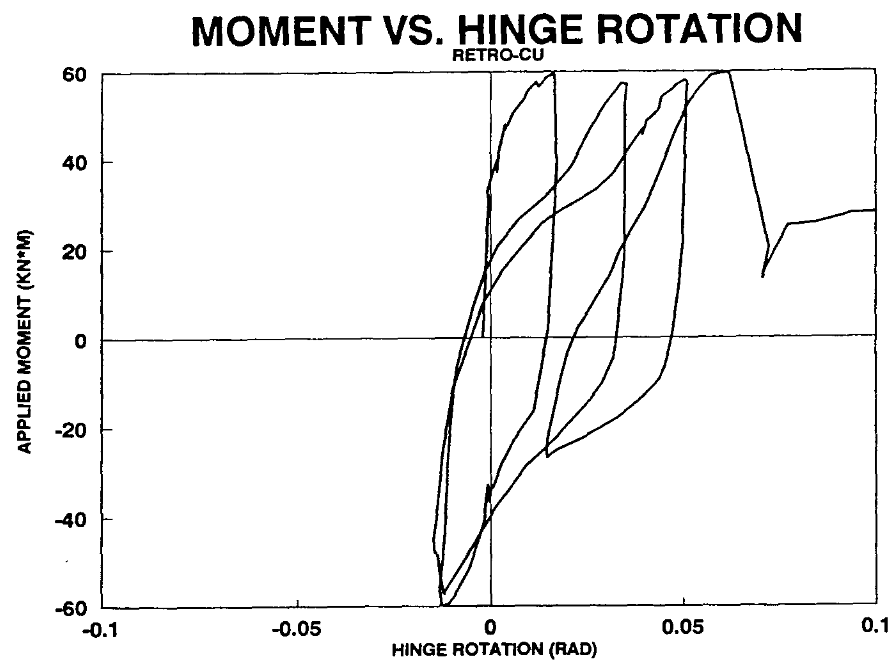


Figure D-22 RETRO-CU: Hinge Rotation at Gap

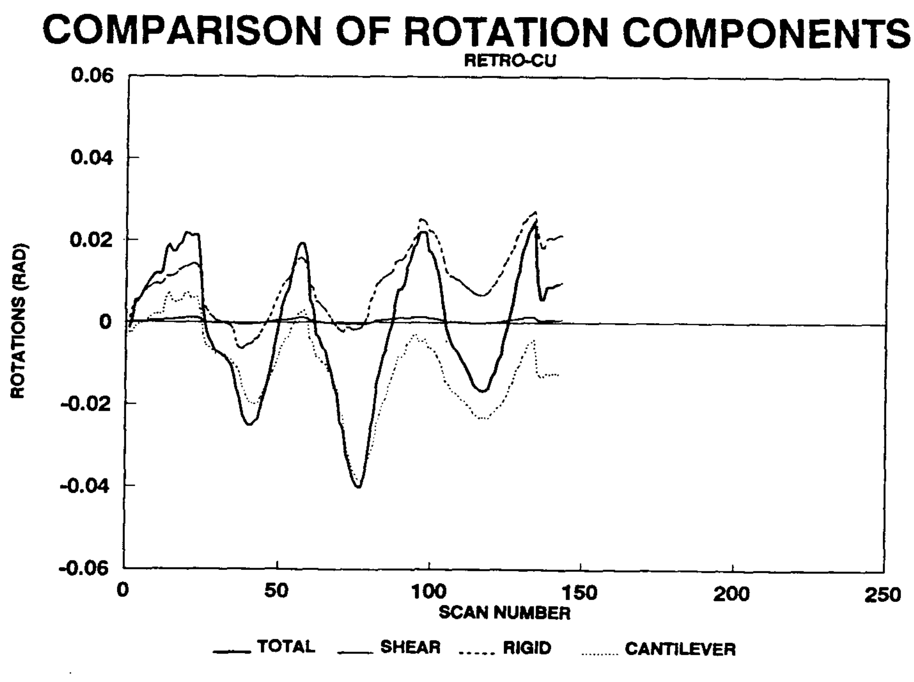


Figure D-23 RETRO-CU: Rotation Components

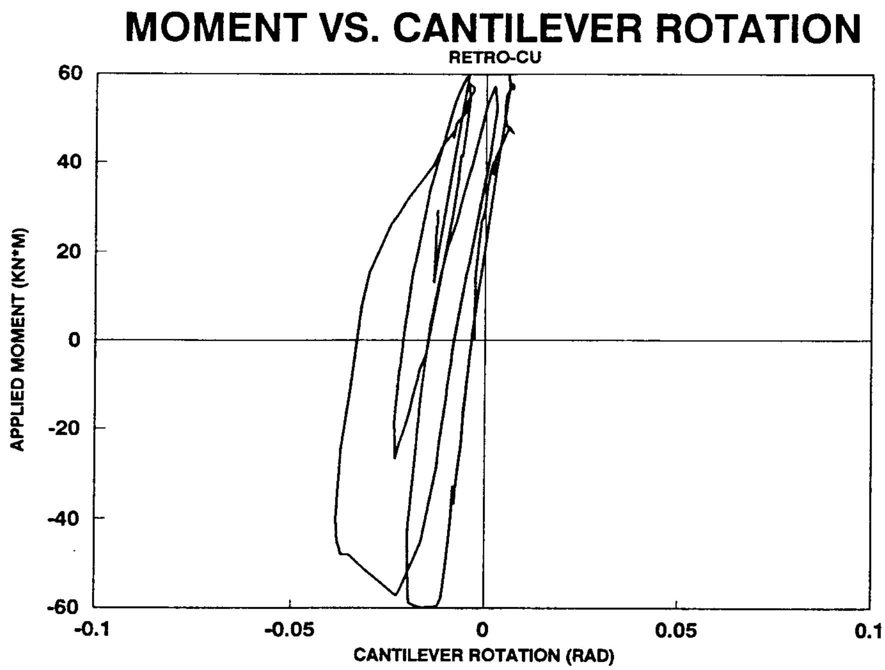


Figure D-24 RETRO-CU: Cantilever Rotation

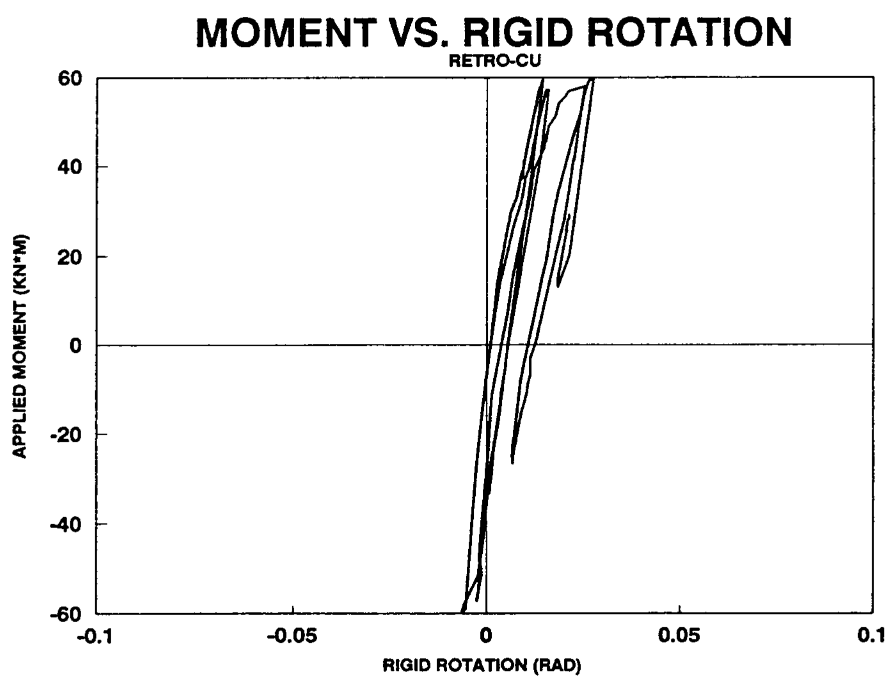


Figure D-25 RETRO-CU: Rigid Rotation

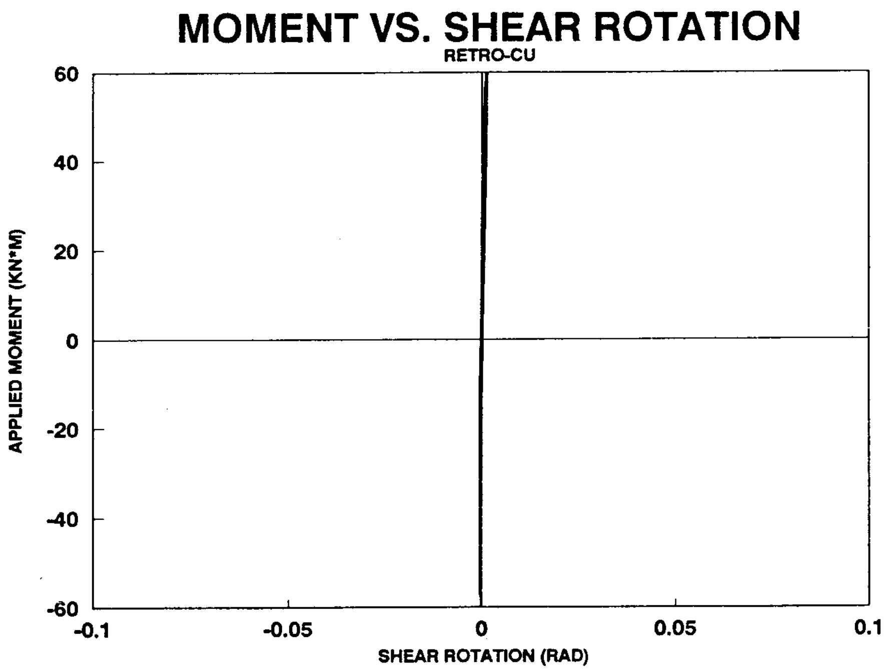


Figure D-26 RETRO-CU: Shear Rotation

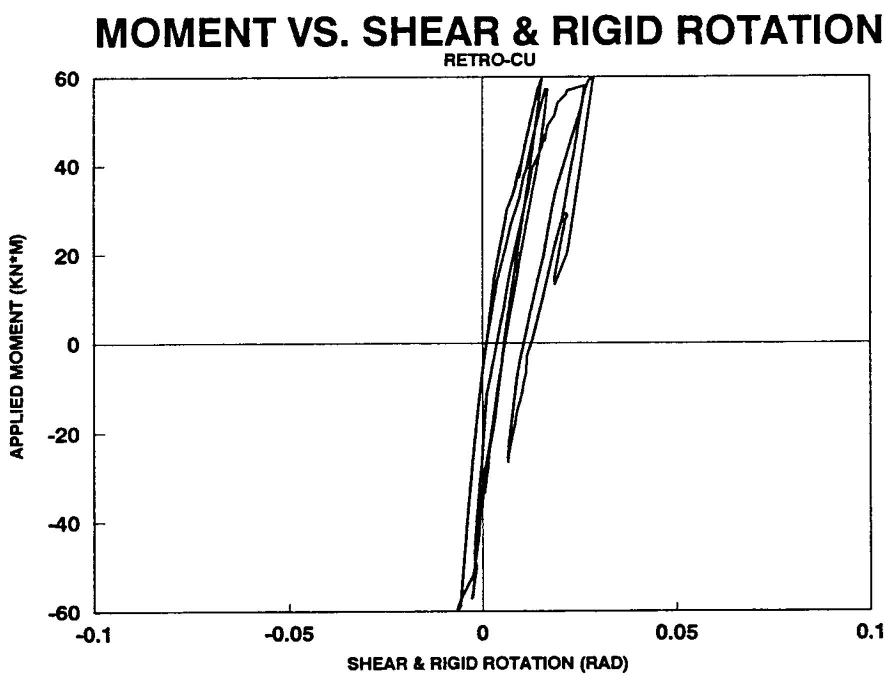


Figure D-27 RETRO-CU: Shear and Rigid Rotation

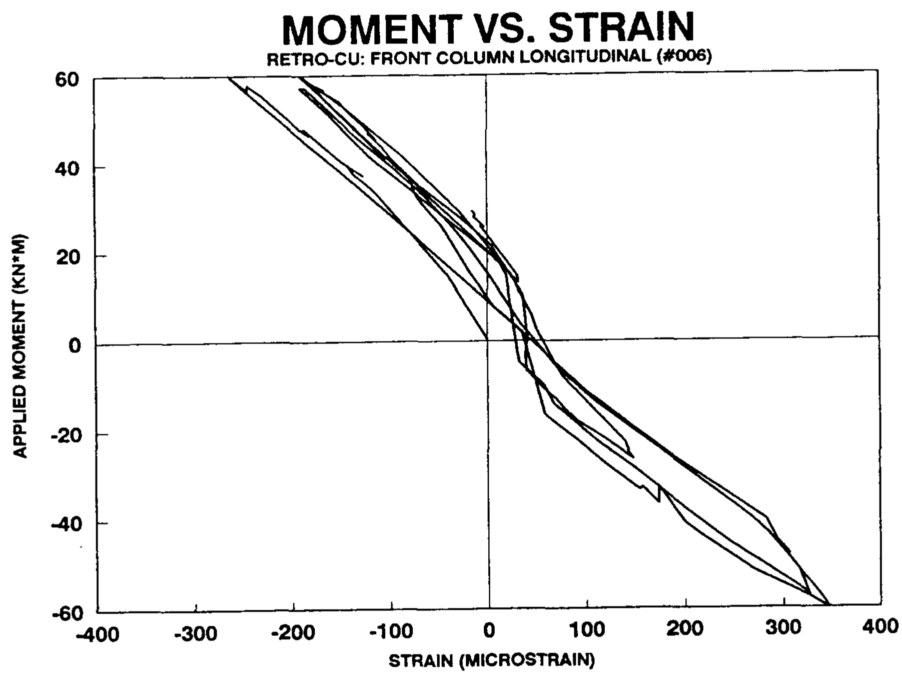


Figure D-28 RETRO-CU: Front Column Longitudinal (FCL)

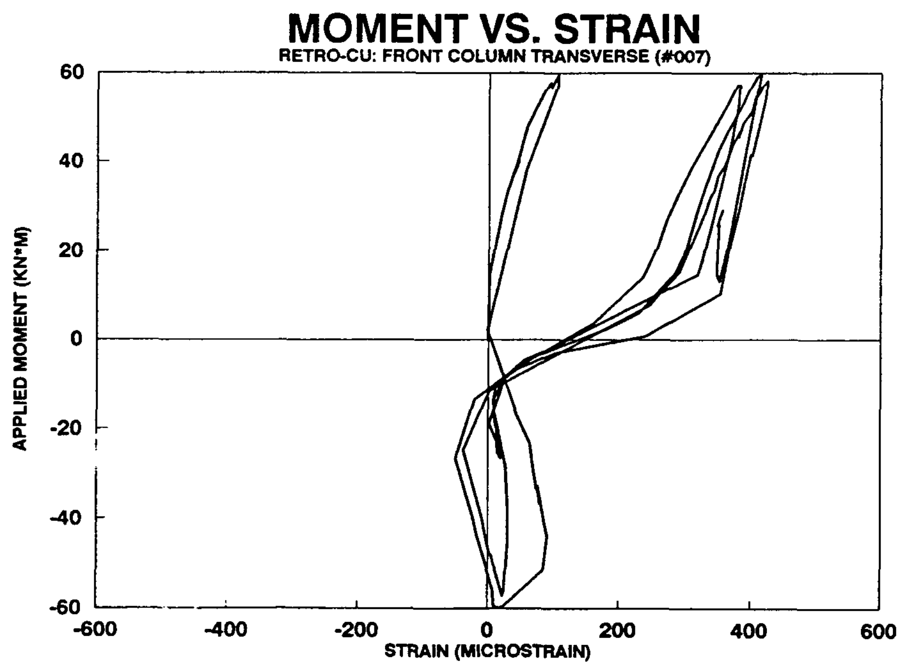


Figure D-29 RETRO-CU: Front Column Transverse (FCT)

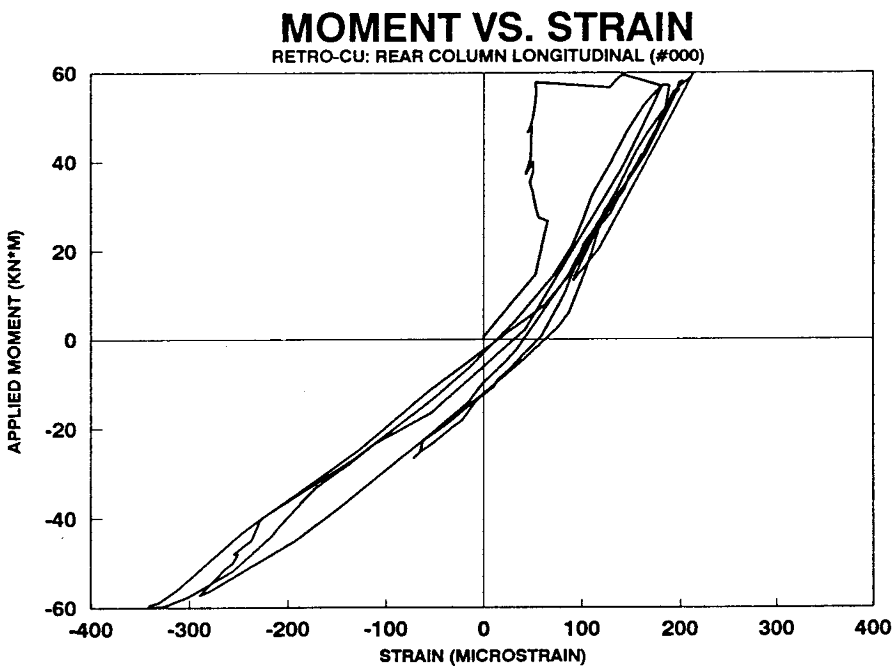


Figure D-30 RETRO-CU: Rear Column Longitudinal (RCL)

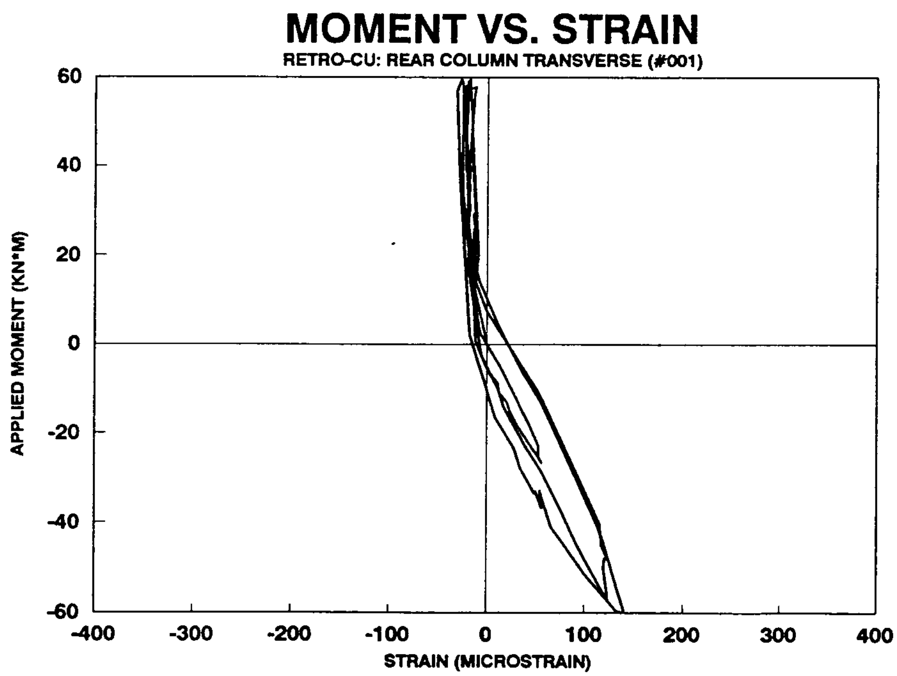


Figure D-31 RETRO-CU: Rear Column Transverse (RCT)

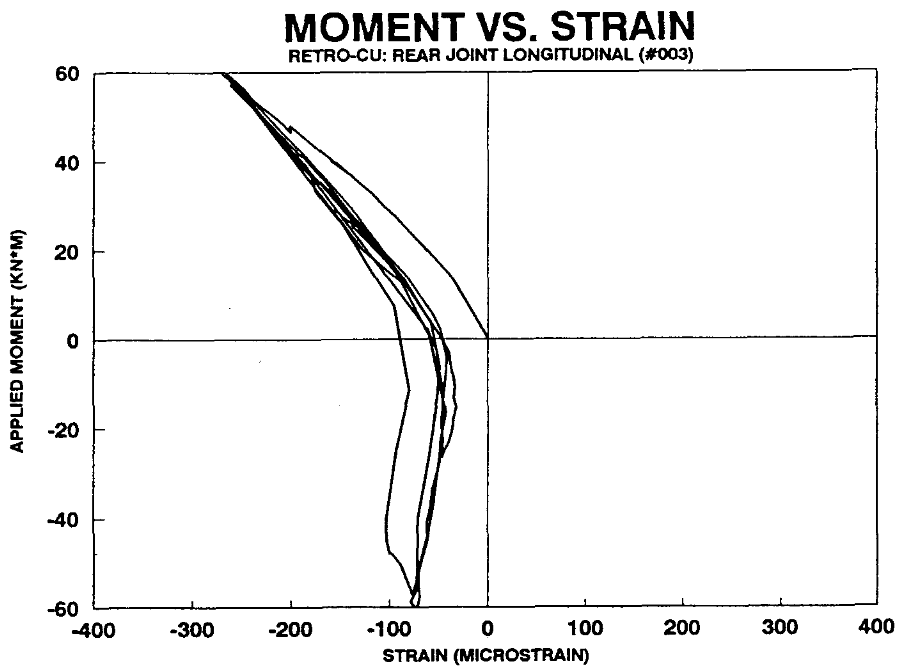


Figure D-32 RETRO-CU: Rear Joint Longitudinal (RJL)

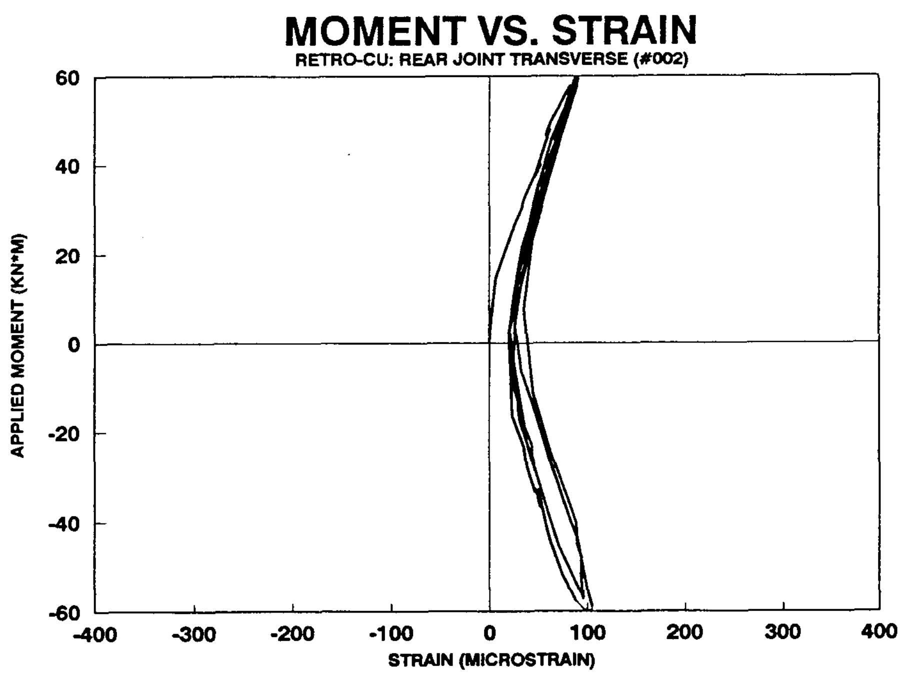


Figure D-33 RETRO-CU: Rear Joint Transverse (RJT)

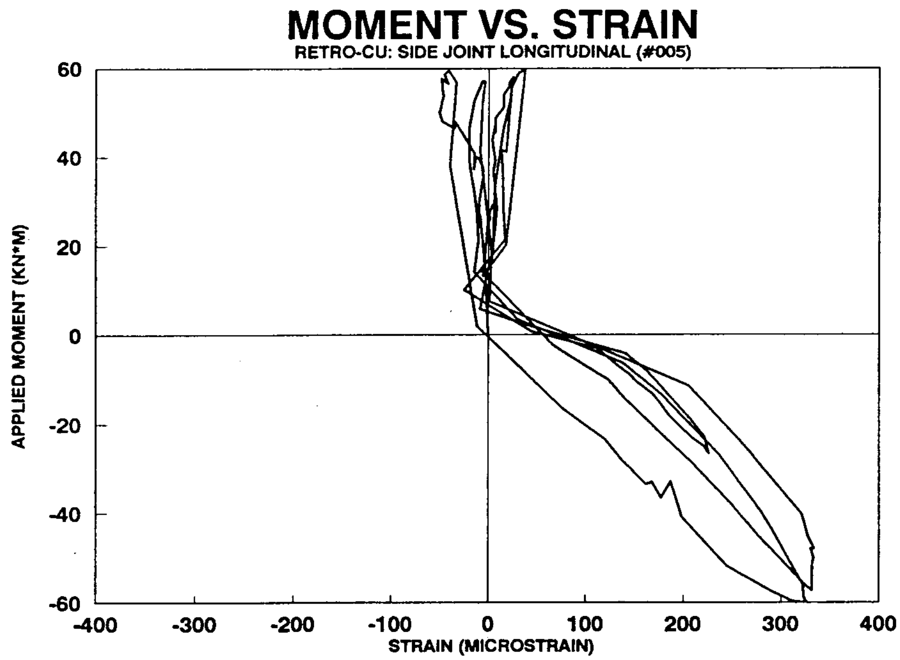


Figure D-34 RETRO-CU: Side Joint Longitudinal (SJL)

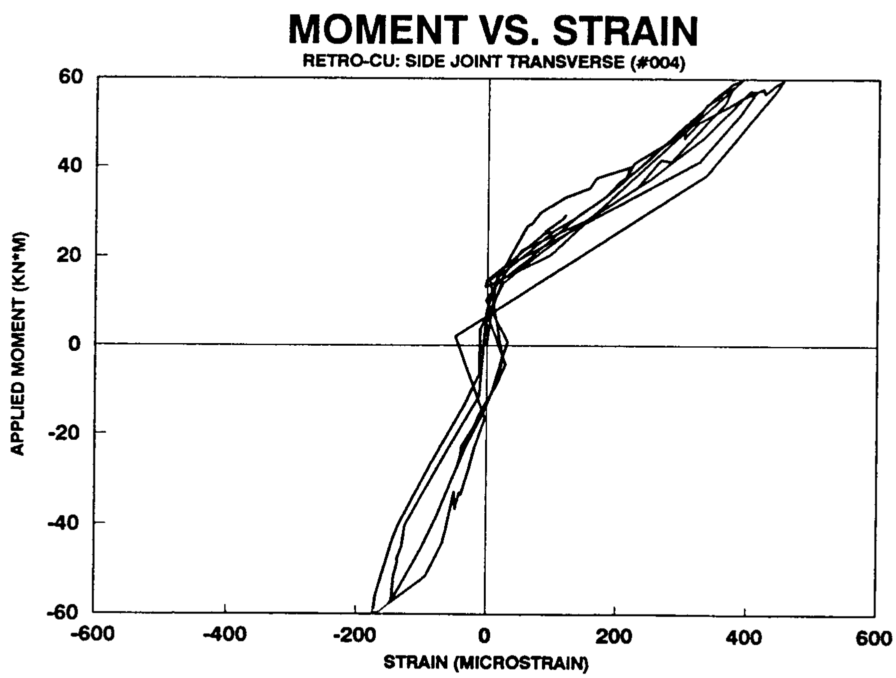


Figure D-35 RETRO-CU:· Side Joint Transverse (SJT)

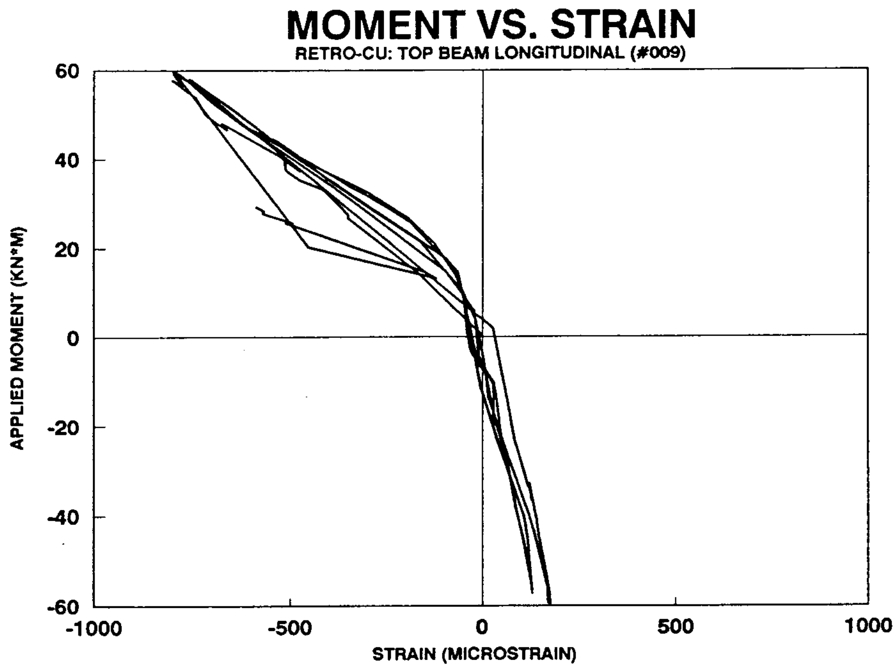


Figure D-36 RETRO-CU: Top Beam Longitudinal (TBL)

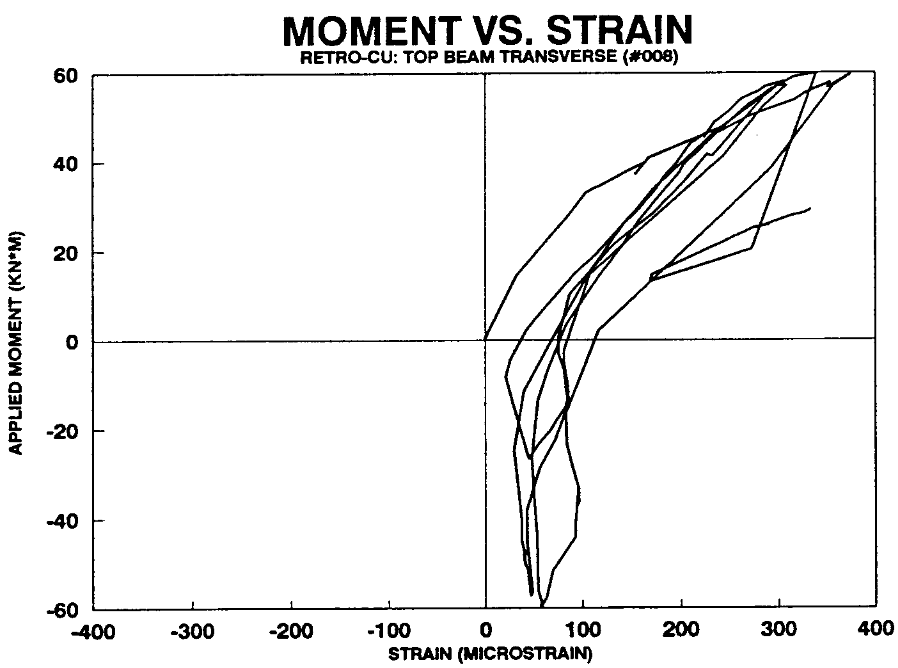


Figure D-37 RETRO-CU: Top Beam Transverse (TBT)

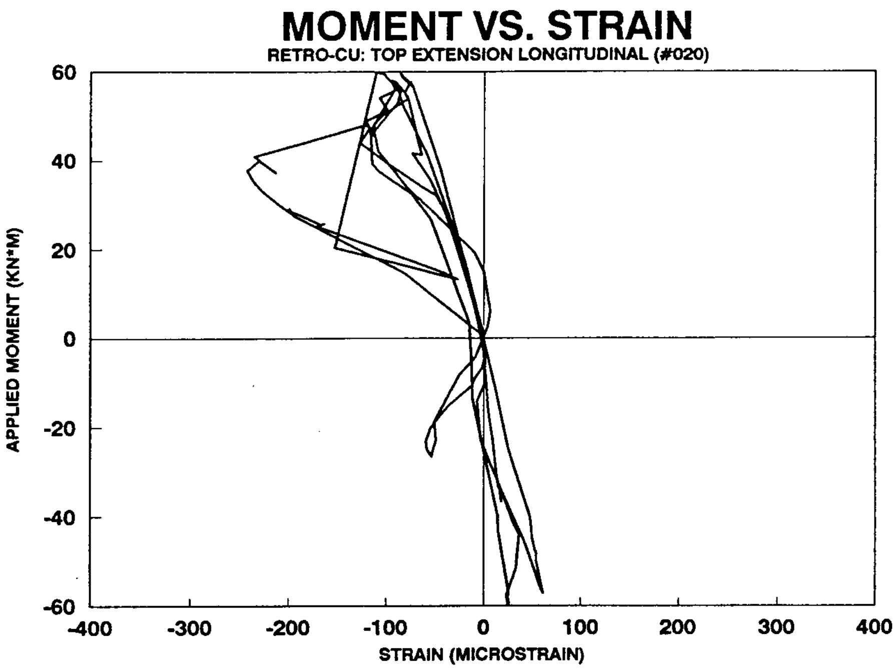


Figure D-38 RETRO-CU: Top Extension Longitudinal (TEL)

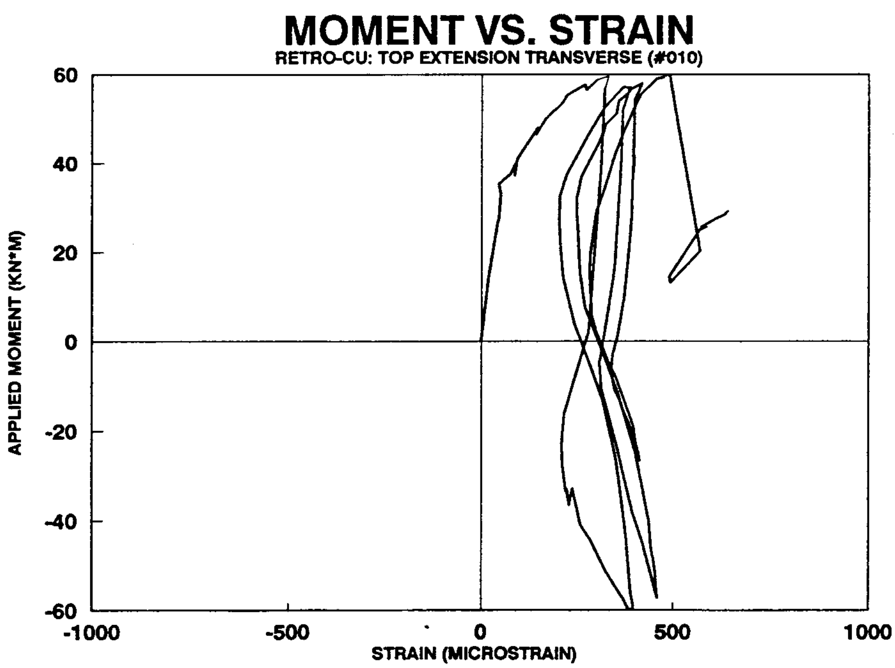


Figure D-39 RETRO-CU: Top Extension Transverse (TET)

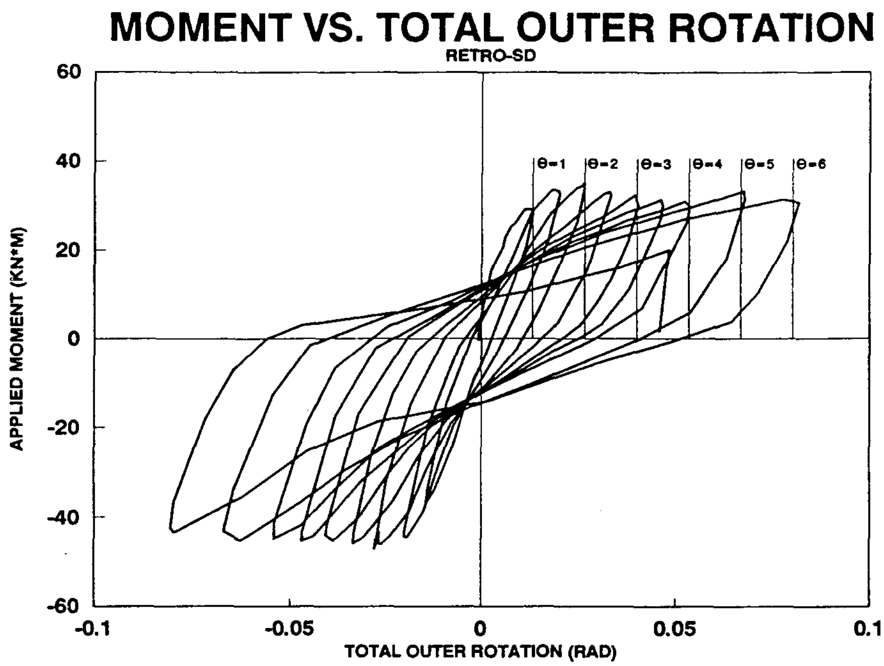


Figure D-40 RETRO-SD: Total Outer Rotation

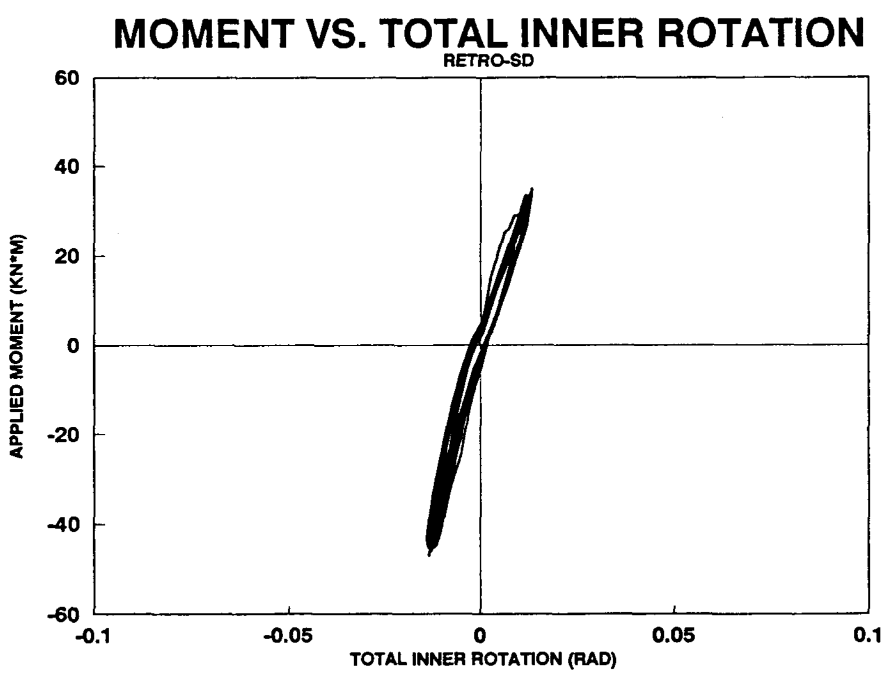


Figure D-41 RETRO-SD: Total Inner Rotation

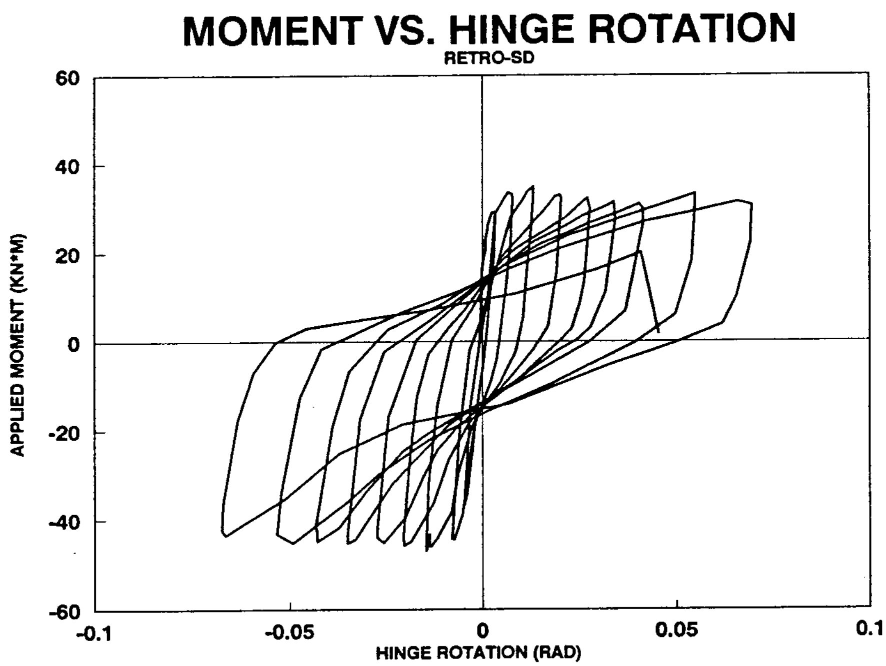


Figure D-42 RETRO-SD: Hinge Rotation at Gap

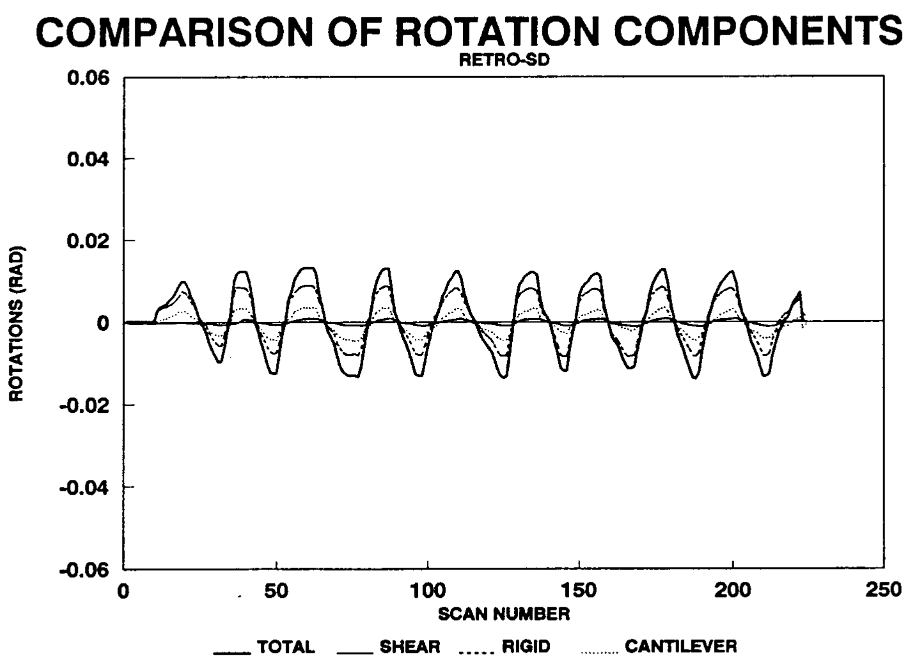


Figure D-43 RETRO-SD: Rotation Components

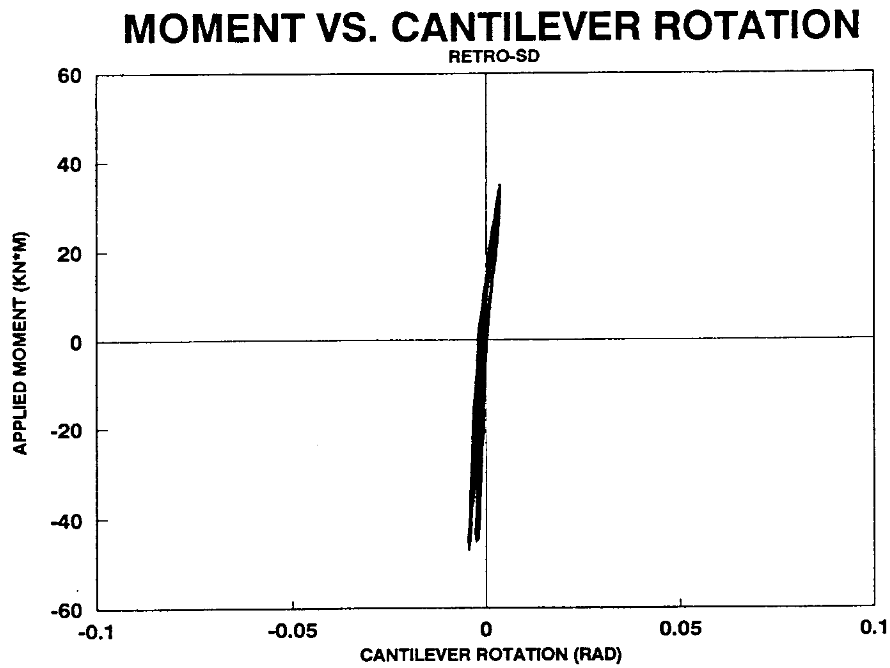


Figure D-44 RETRO-SD: Cantilever Rotation

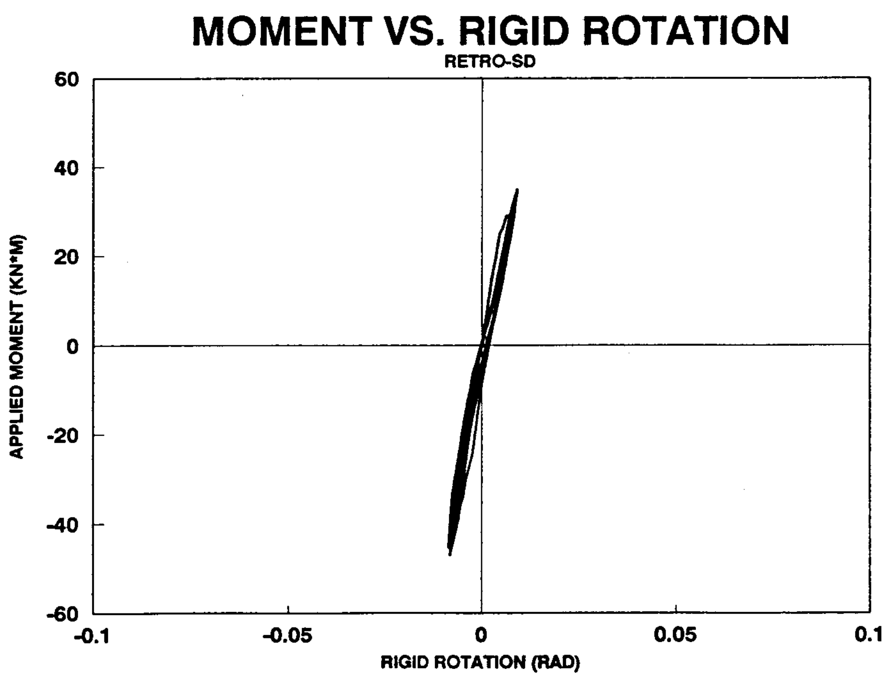


Figure D-45 RETRO-SD: Rigid Rotation

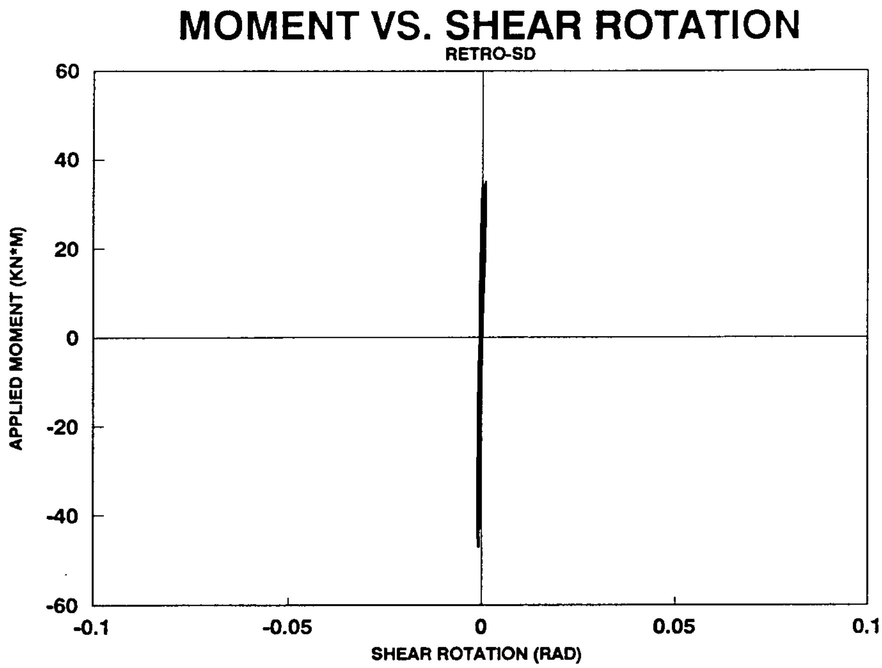


Figure D-46 RETRO-SD: Shear Rotation

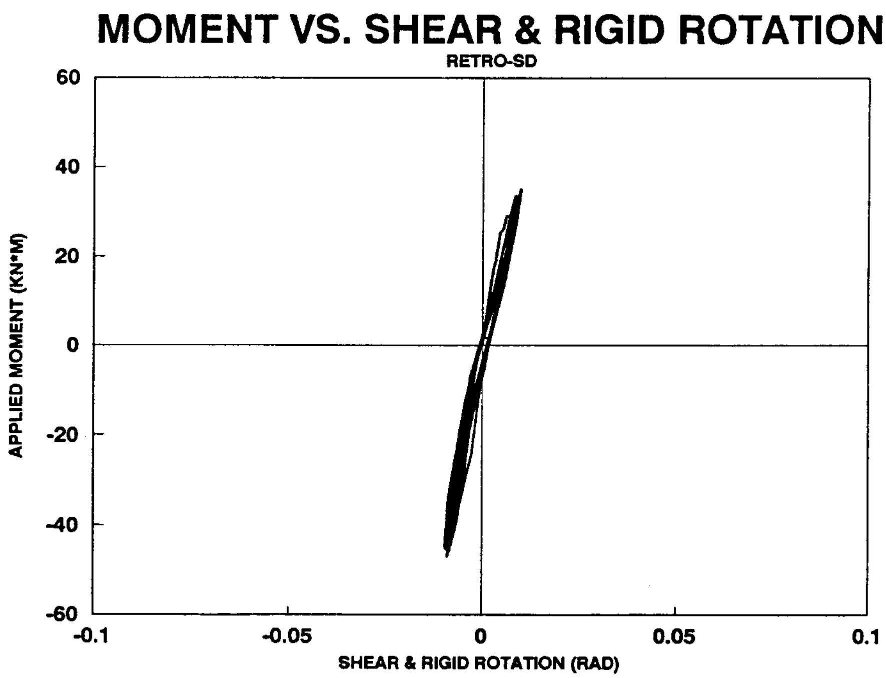


Figure D-47 RETRO-SD: Shear and Rigid Rotation

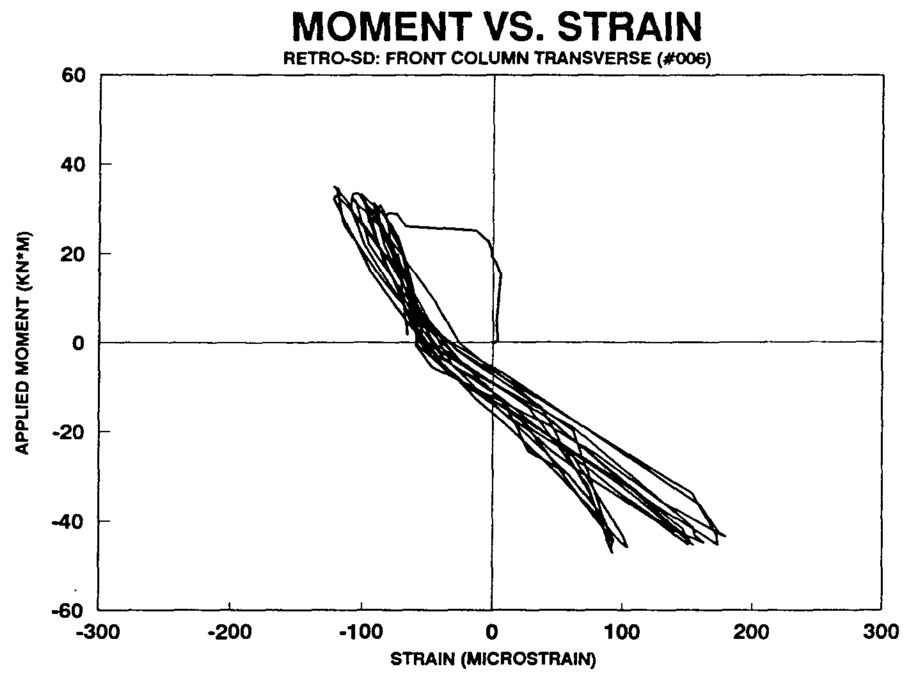


Figure D-48 RETRO-SD: Front Column Transverse (FCT)

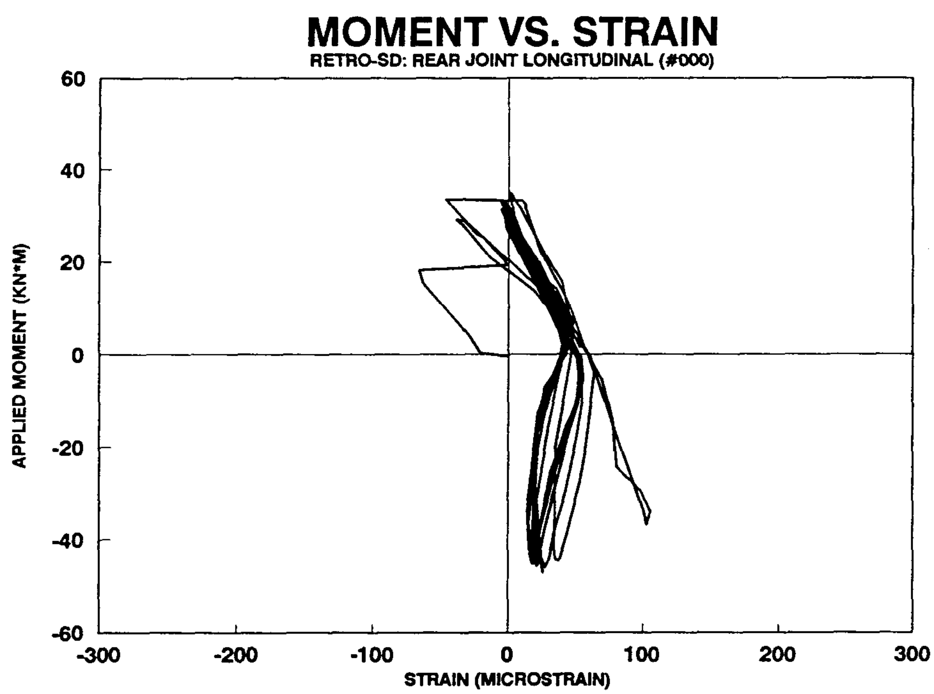


Figure D-49 RETRO-SD: Rear Joint Longitudinal (RJL)

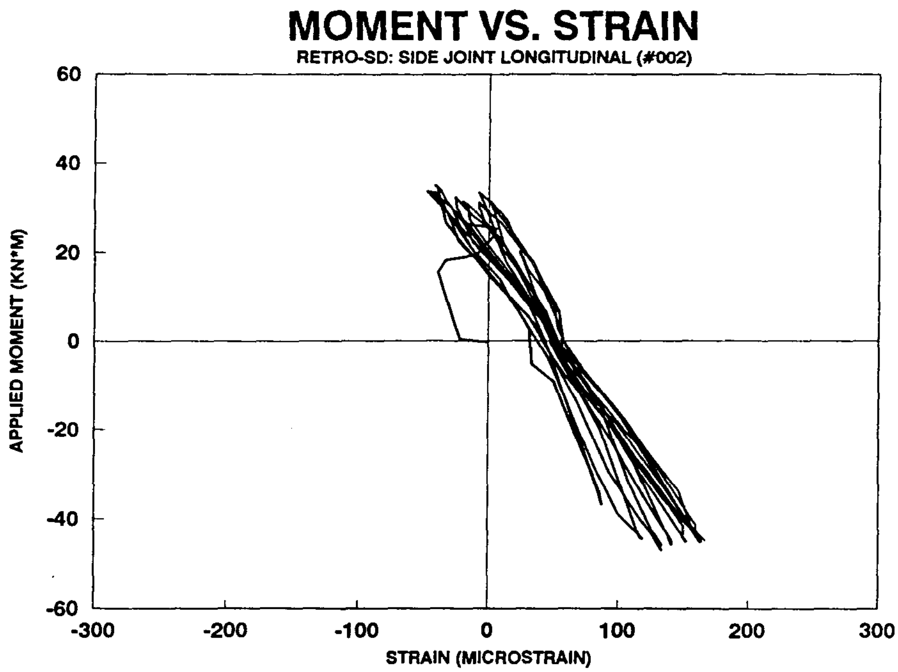


Figure D-50 RETRO-SD: Side Joint Longitudinal (SJL)

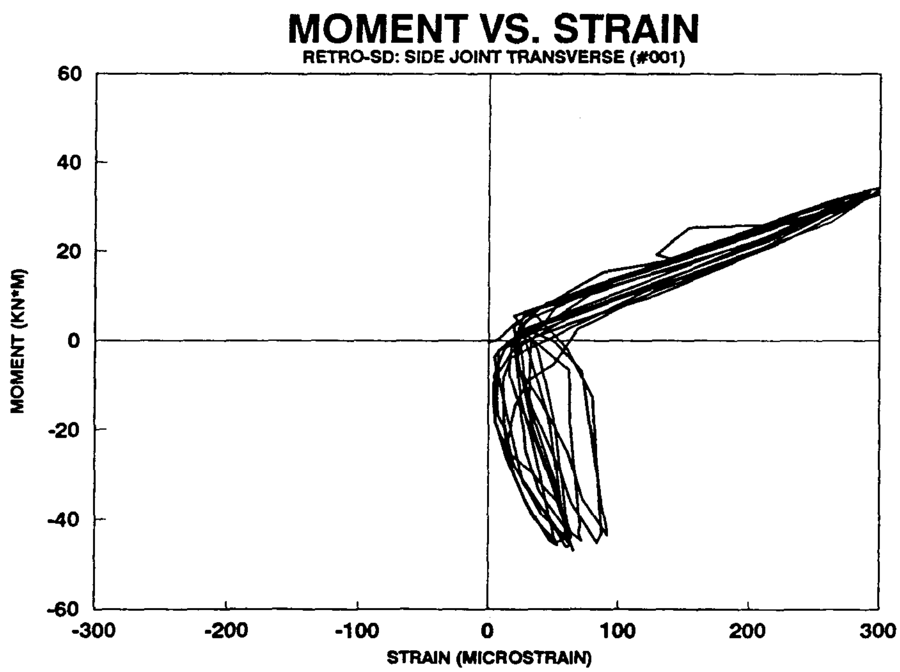


Figure D-51 RETRO-SD: Side Joint Transverse (SJT)

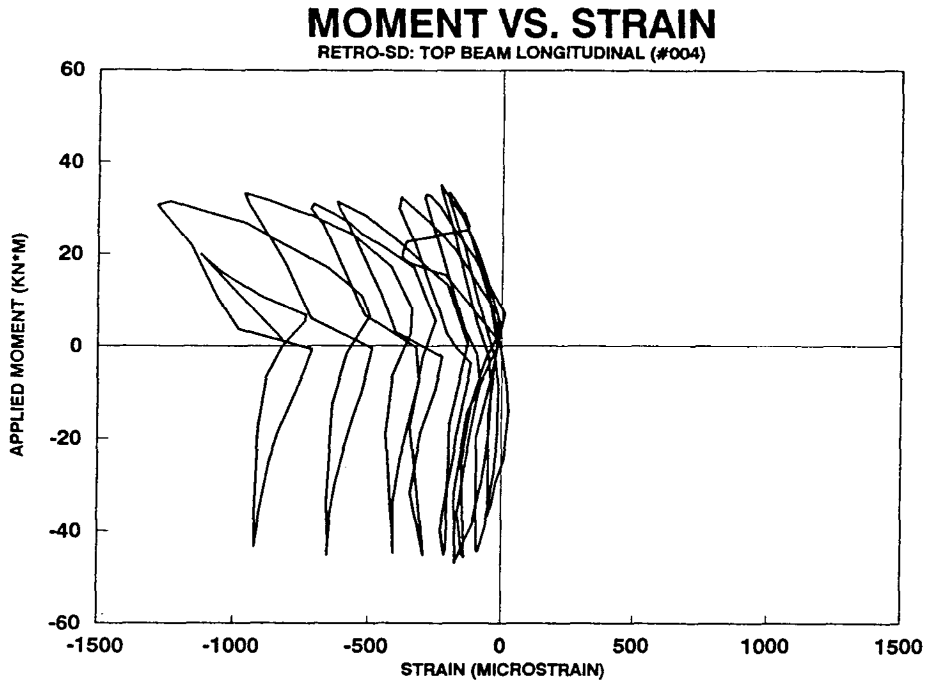


Figure D-52 RETRO-SD: Top Beam Longitudinal (TBL)

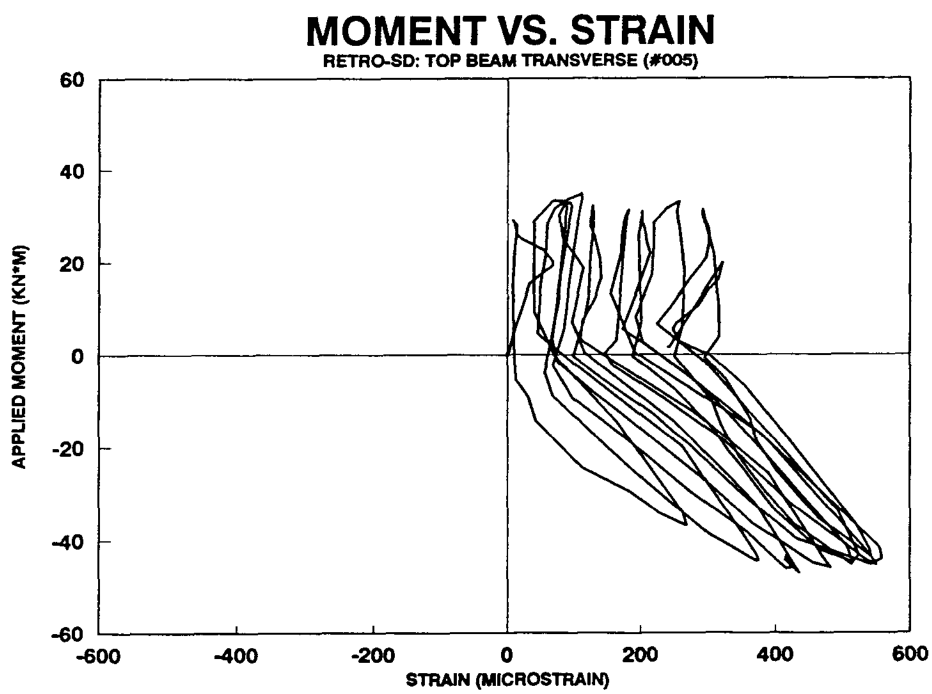


Figure D-53 RETRO-SD: Top Beam Transverse (TBT)

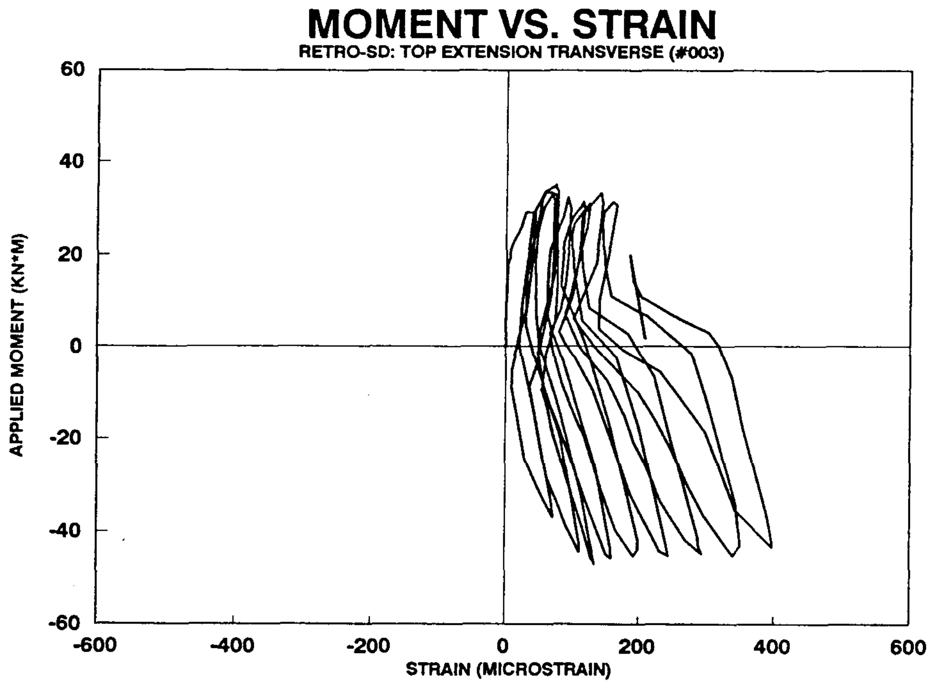


Figure D-54 RETRO-SD: Top Extension Transverse (TET)

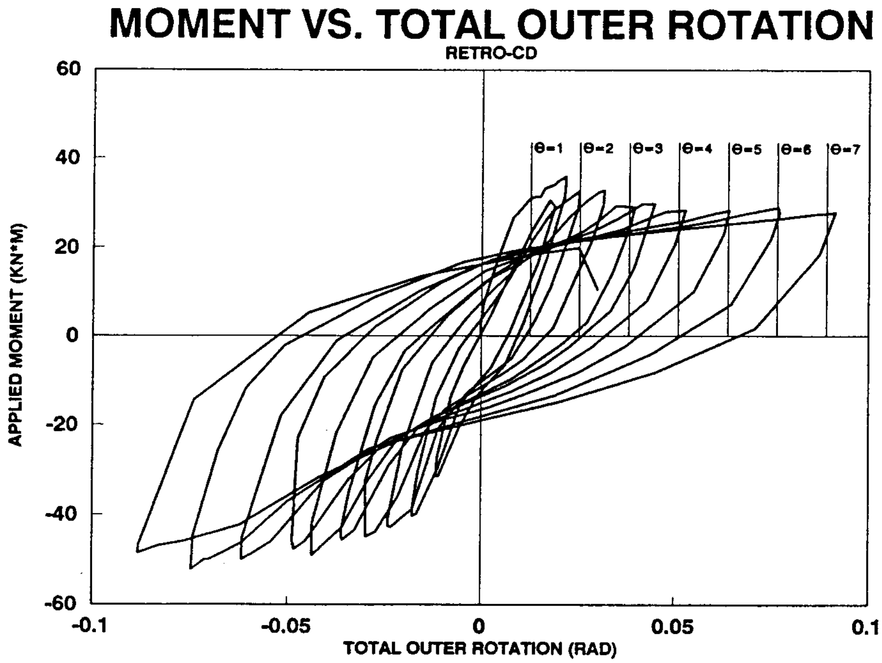


Figure D-55 RETRO-CD: Total Outer Rotation

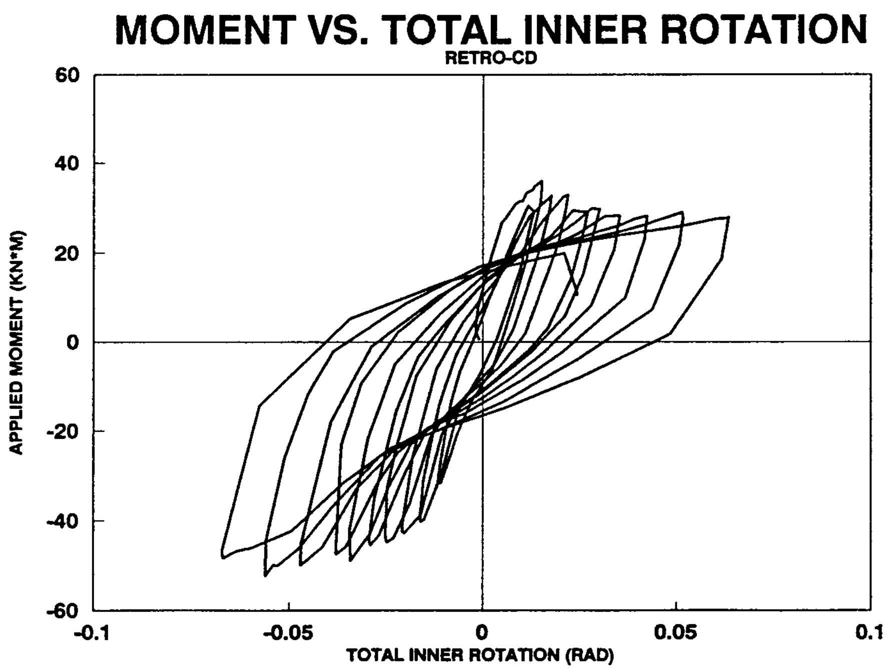


Figure D-56 RETRO-CD: Total Inner Rotation

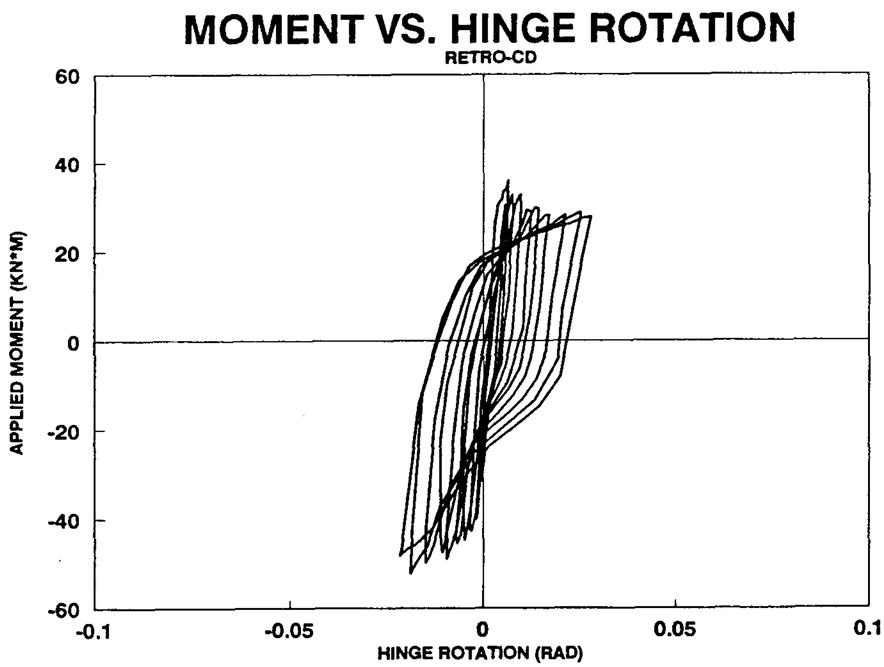


Figure D-57 RETRO-CD: Hinge Rotation at Gap

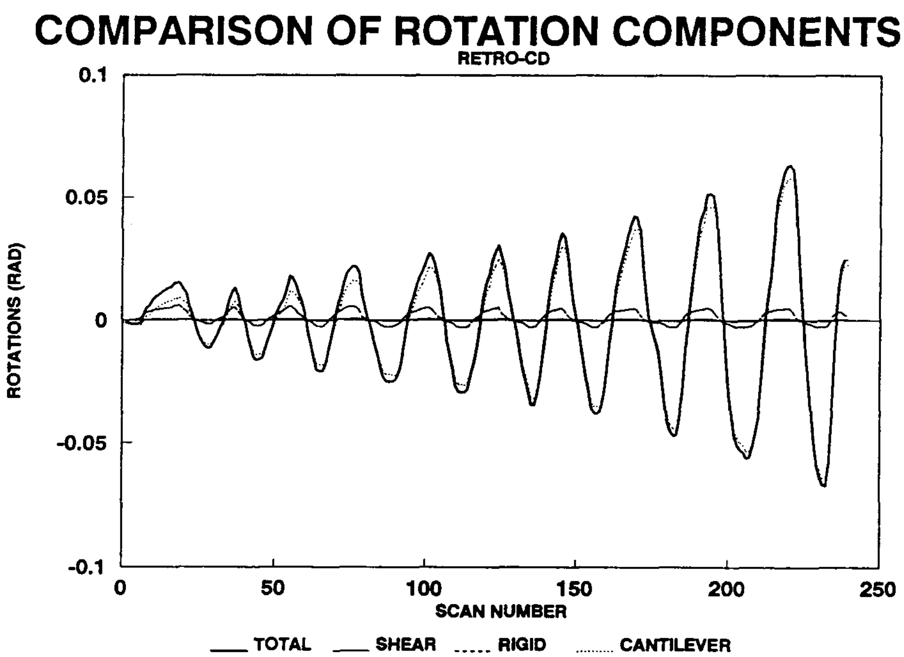


Figure D-58 RETRO-CD: Rotation Components

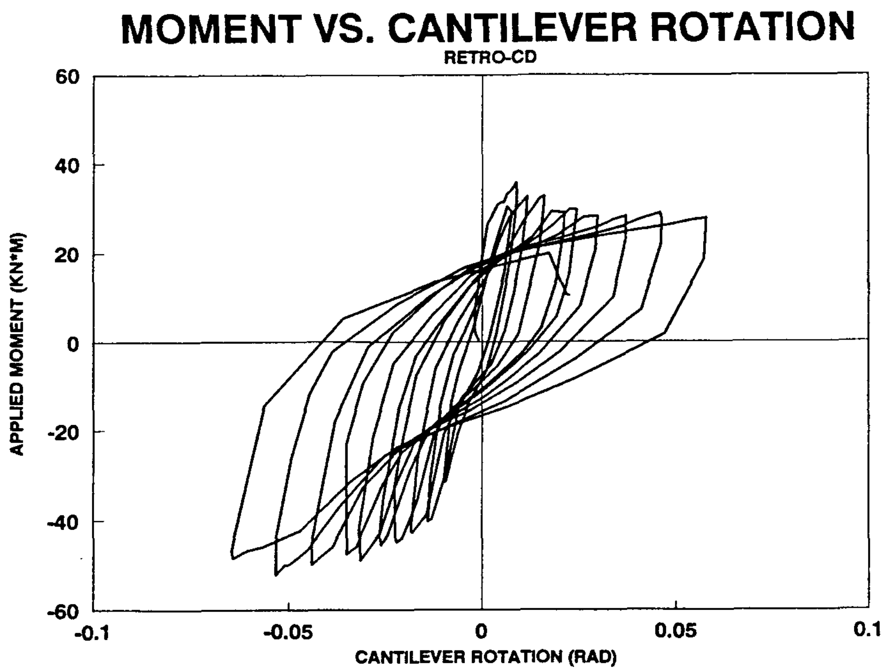


Figure D-59 RETRO-CD: Cantilever Rotation

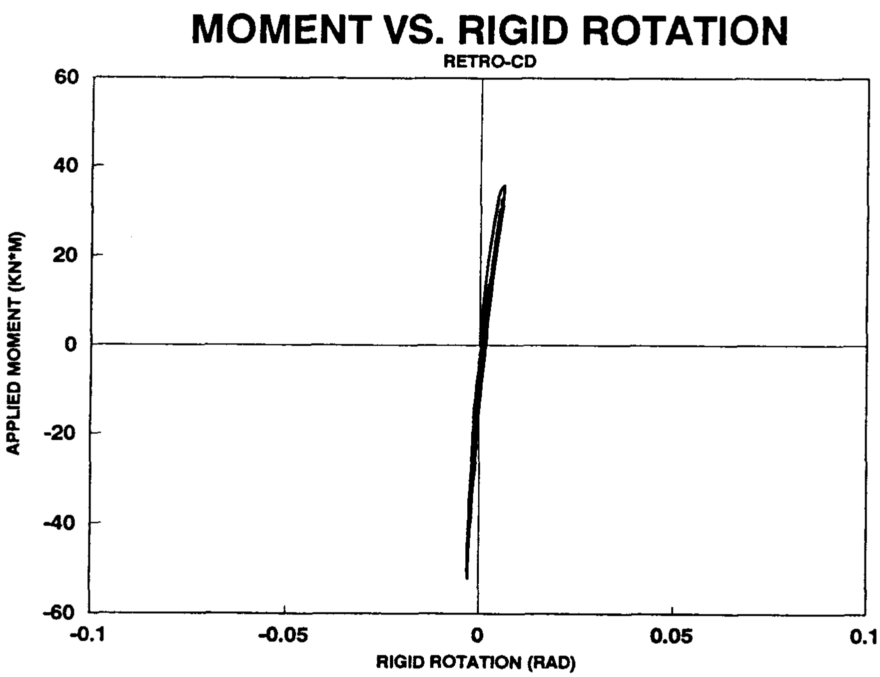


Figure D-60 RETRO-CD: Rigid Rotation

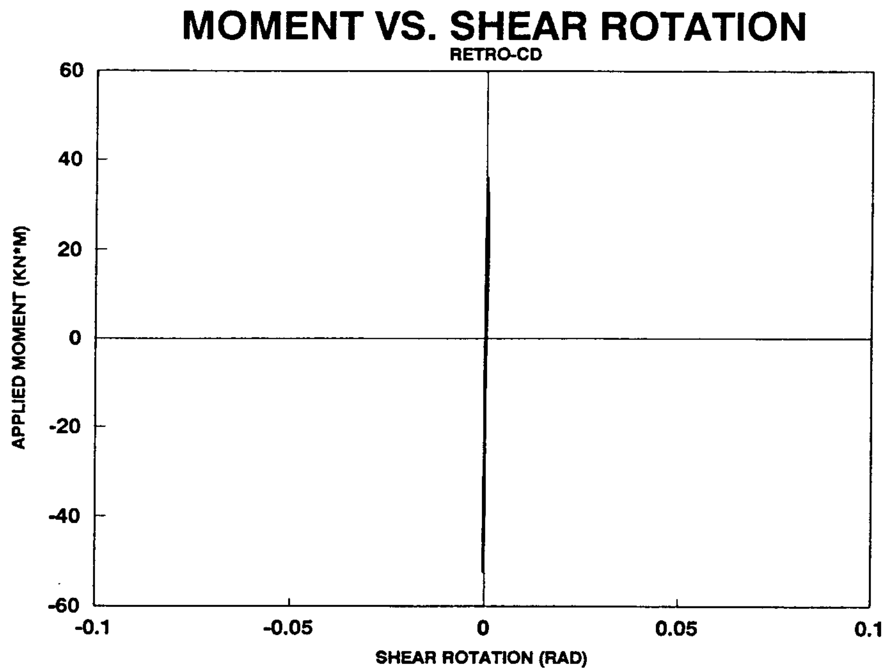


Figure D-61 RETRO-CD: Shear Rotation

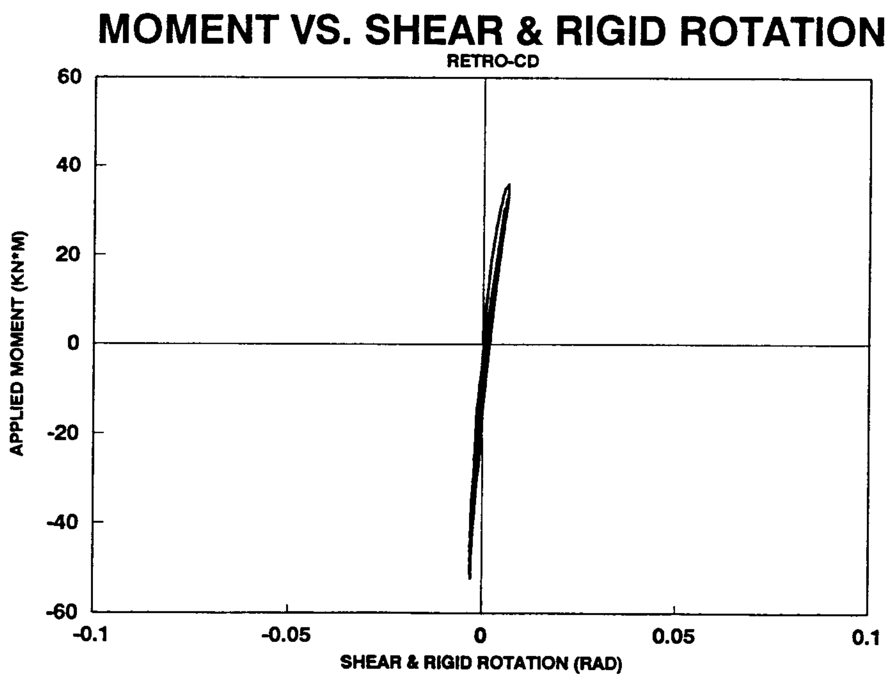


Figure D-62 RETRO-CD: Shear and Rigid Rotation

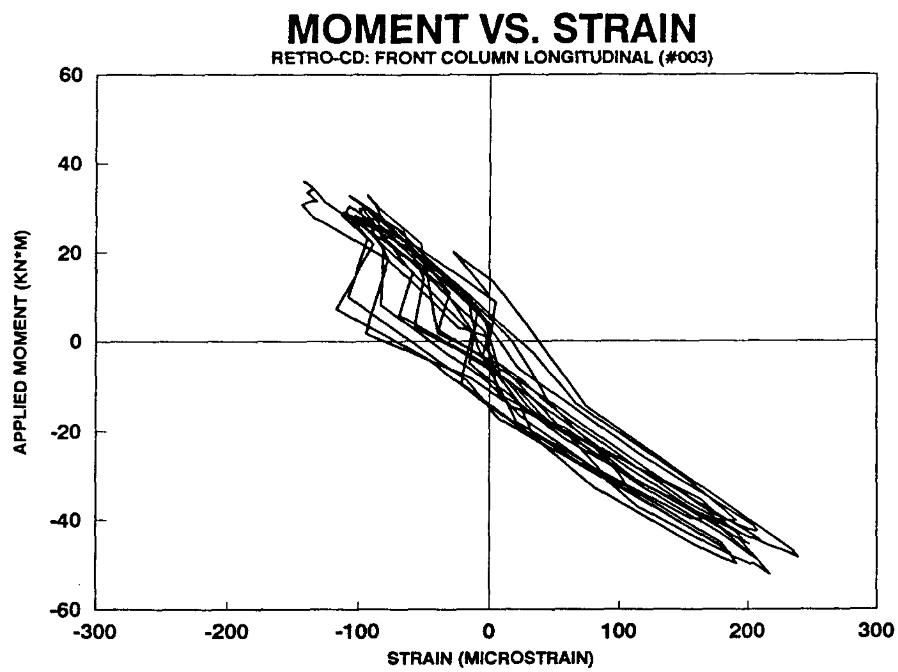


Figure D-63 RETRO-CD: Front Column Longitudinal (FCL)

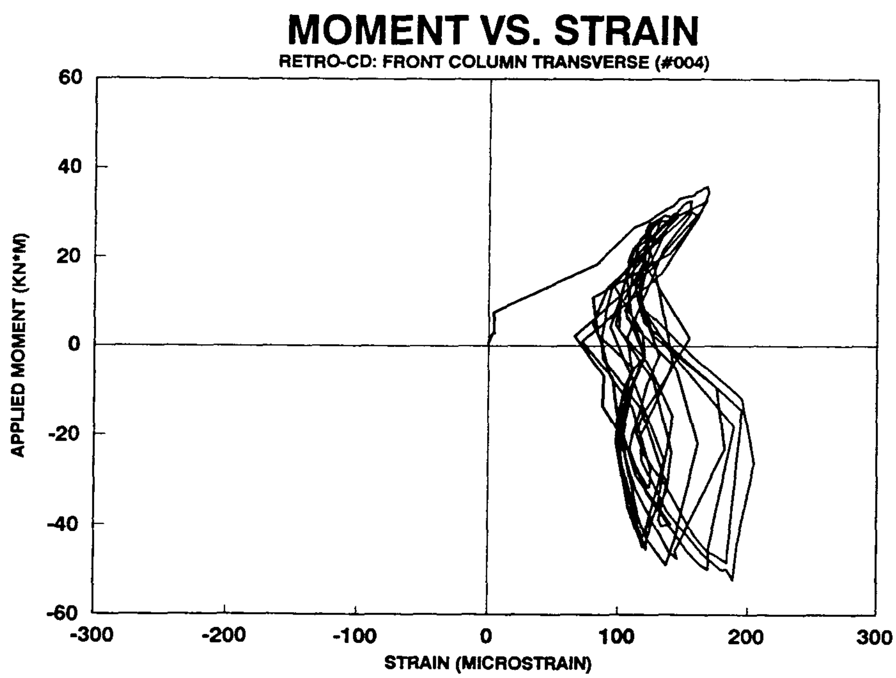


Figure D-64 RETRO-CD: Front Column Transverse (FCT)

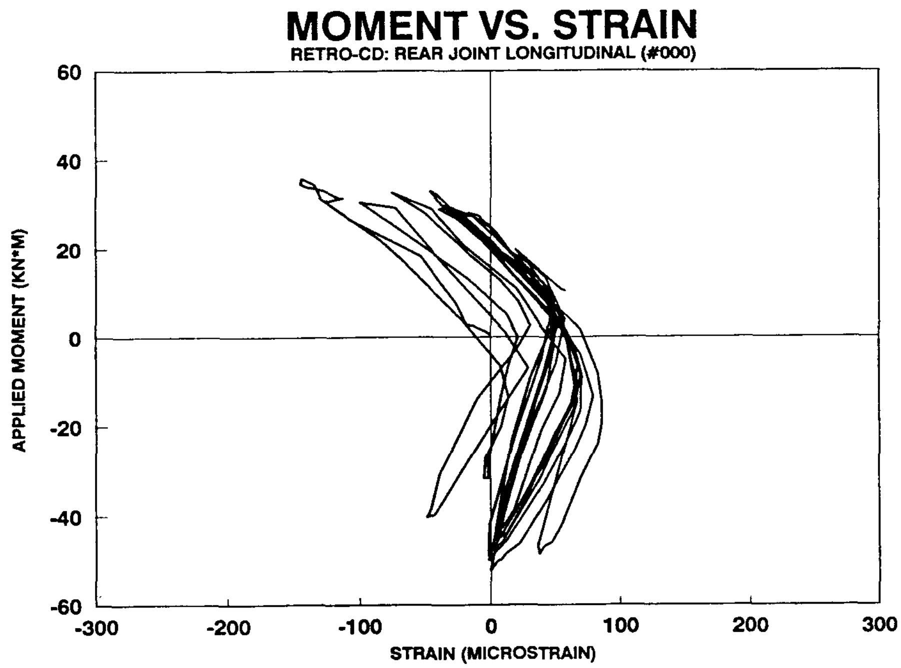


Figure D-65 RETRO-CD: Rear Joint Longitudinal (RJL)

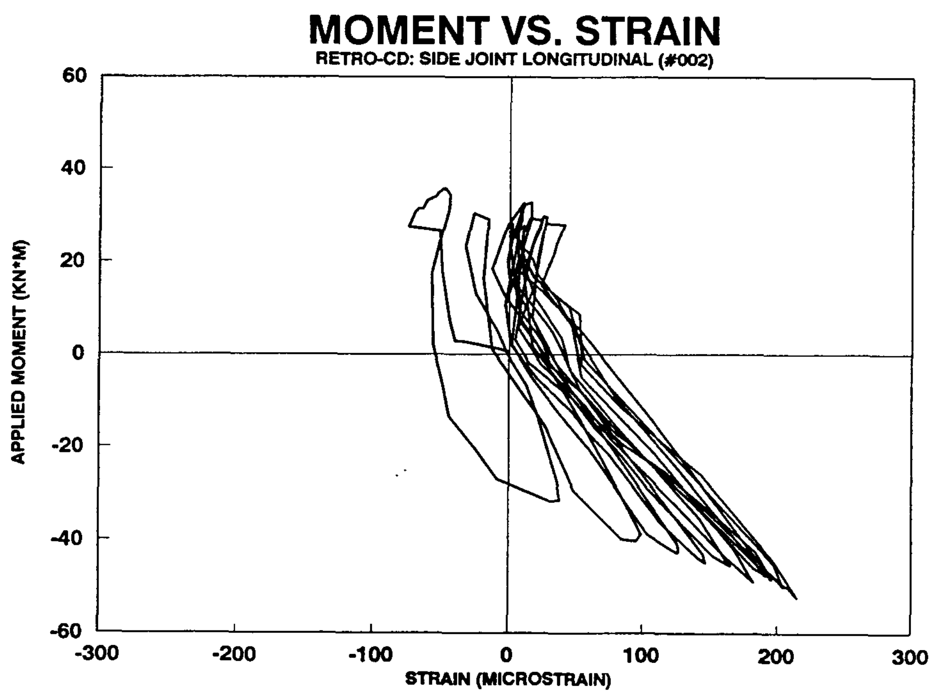


Figure D-66 RETRO-CD: Side Joint Longitudinal (SJL)

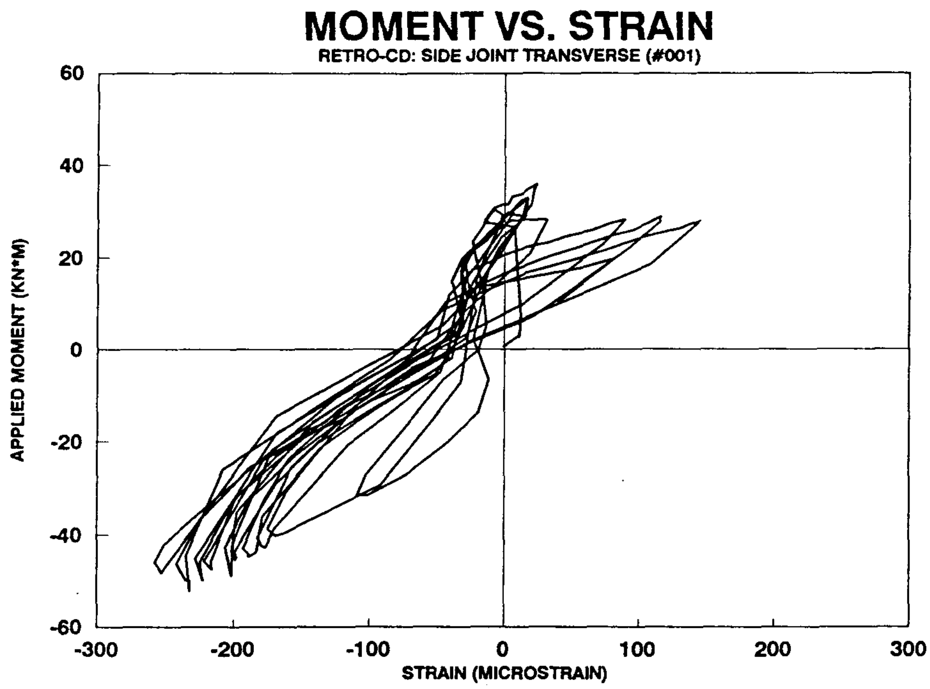


Figure D-67 RETRO-CD: Side Joint Transverse (SJT)

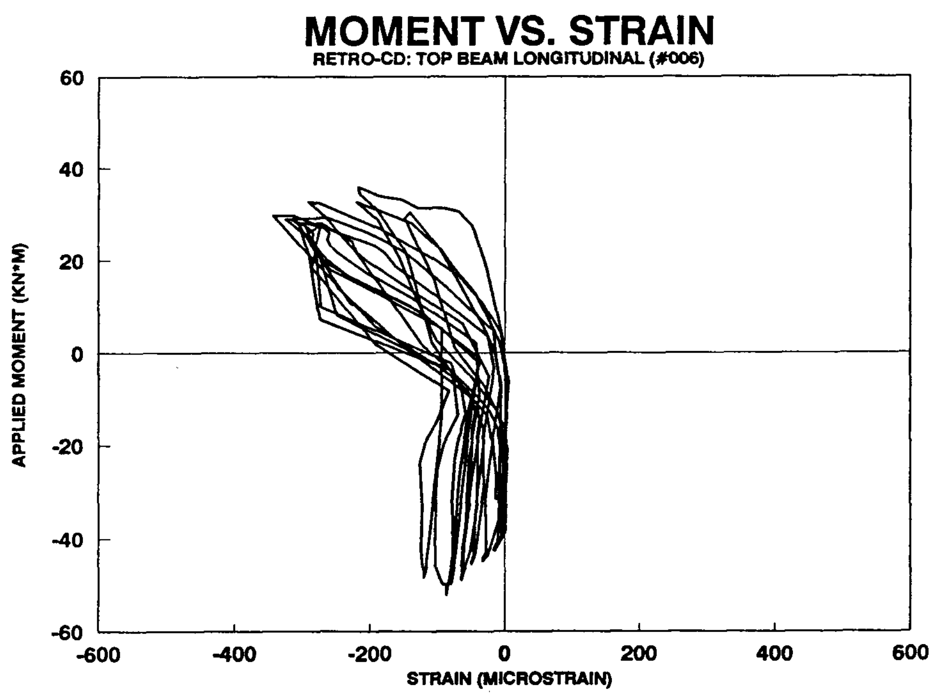


Figure D-68 RETRO-CD: Top Beam Longitudinal (TBL)

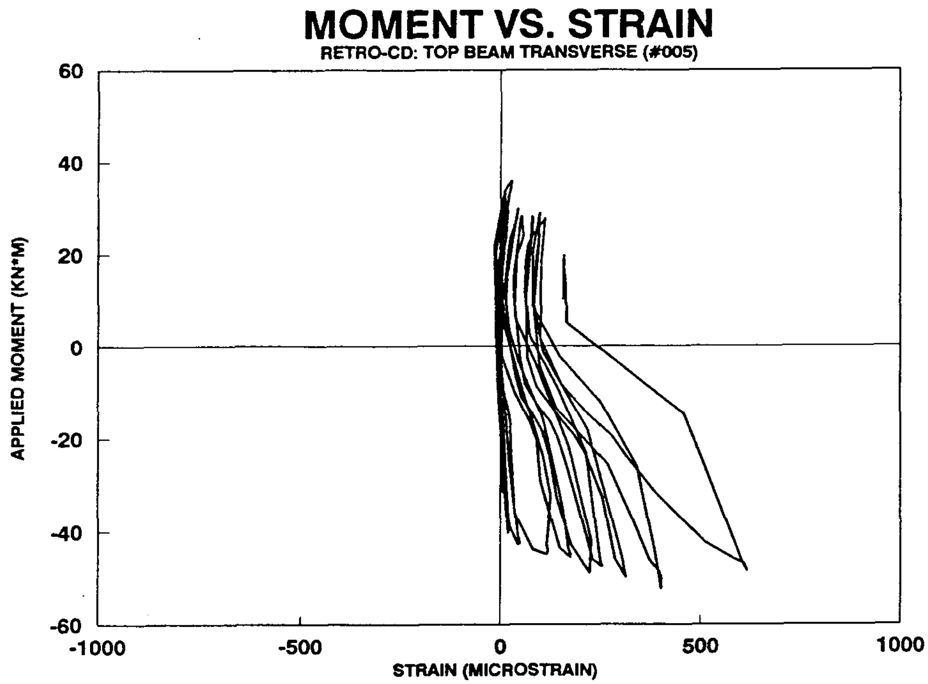


Figure D-69 RETRO-CD: Top Beam Transverse (TBT)

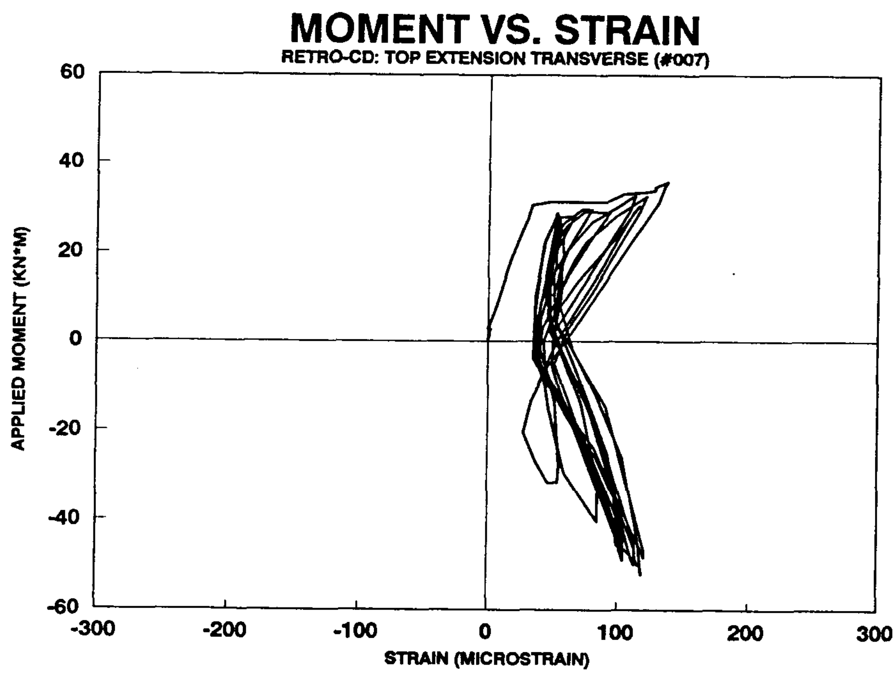


Figure D-70 RETRO-CD: Top Extension Transverse (TET)