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REPAIR AND STRENGTHENING OF BEAM-TO-COLUMN CONNECTIONS SUBJECTED TO EARTHQUAKE LOADING

by

Miguel Corazao[1] and Ahmad J. Durrani[2]

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1 Graduate Research Assistant, Department of Civil Engineering, Rice University
2 Associate Professor, Department of Civil Engineering, Rice University

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

PREFACE

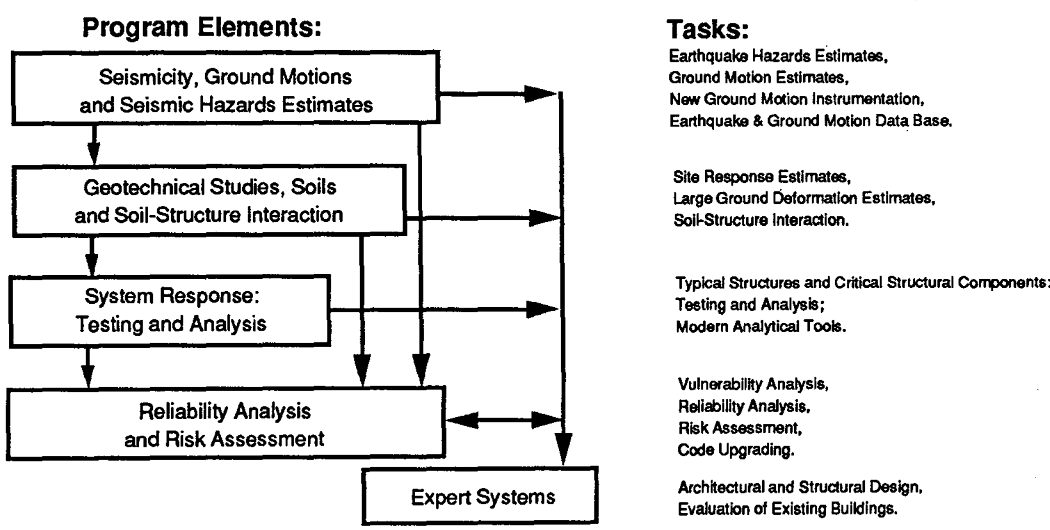
The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion and dissemination of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures and lifelines that are found in zones of moderate to high seismicity throughout the United States.

NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

- Existing and New Structures
- Secondary and Protective Systems
- Lifeline Systems
- Disaster Research and Planning

This technical report pertains to Program 1, Existing and New Structures, and more specifically to system response investigations.

The long term goal of research in Existing and New Structures is to develop seismic hazard mitigation procedures through rational probabilistic risk assessment for damage or collapse of structures, mainly existing buildings, in regions of moderate to high seismicity. The work relies on improved definitions of seismicity and site response, experimental and analytical evaluations of systems response, and more accurate assessment of risk factors. This technology will be incorporated in expert systems tools and improved code formats for existing and new structures. Methods of retrofit will also be developed. When this work is completed, it should be possible to characterize and quantify societal impact of seismic risk in various geographical regions and large municipalities. Toward this goal, the program has been divided into five components, as shown in the figure below:



System response investigations constitute one of the important areas of research in Existing and New Structures. Current research activities include the following:

1. Testing and analysis of lightly reinforced concrete structures, and other structural components common in the eastern United States such as semi-rigid connections and flexible diaphragms.
2. Development of modern, dynamic analysis tools.
3. Investigation of innovative computing techniques that include the use of interactive computer graphics, advanced engineering workstations and supercomputing.

The ultimate goal of projects in this area is to provide an estimate of the seismic hazard of existing buildings which were not designed for earthquakes and to provide information on typical weak structural systems, such as lightly reinforced concrete elements and steel frames with semi-rigid connections. An additional goal of these projects is the development of modern analytical tools for the nonlinear dynamic analysis of complex structures.

A group of projects is concerned with reinforced concrete structures that are typical in the east and midwest. Whereas the other projects concentrate on lightly reinforced concrete structures, the work summarized in this report was concerned with the strengthening or repairing of concrete structures, some with seismic details. The research is the continuation of prior work on this problem. The plans are to extend this research to lightly reinforced flat plate structures with weak connections.

ABSTRACT

An experimental investigation was performed with the object of evaluating and improving methods of repairing and strengthening of beam-to-column connections damaged by earthquake action. Commonly used repair procedures and strengthening details as well as modifications and improvements of those procedures were tested to compare the behavior of damaged and repaired connection subassemblies with that of undamaged subassemblies. These comparisons were made on the basis of restitution or improvement of strength, stiffness, and energy absorption capabilities.

Five subassemblies of single beam-to-column connections and six multiple connection subassemblies each consisting of two exterior and one interior connection, with some including a floor slab, were subjected to twelve cycles of a predefined cyclic displacement routine. The specimens were repaired using repair techniques such as replacement of damaged concrete, enlargement of section, addition of rolled steel elements, etc. The repaired specimens were then subjected to the same loading history as that used in the original test.

Epoxy injection and replacement of concrete restored the strength of specimens to their original level. However, the stiffness and energy absorption capacity of the specimens repaired with epoxy injection could not be restored, primarily due to difficulty in filling all the internal cracks. Specimens strengthened by reinforced concrete and by steel encasement showed increased strength, stiffness, and ductility and exhibited desirable failure mechanisms.

ACKNOWLEDGEMENT

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INTRODUCTION

1.1 Introduction

Cast-in-place reinforced concrete has a number of desirable characteristics that make it a versatile material in a wide variety of constructions. One of its disadvantages, however, is that, at least in comparison to steel, modifications and repairs of damage are difficult to accomplish. This deficiency creates an important problem in the case of buildings located in seismically active zones since the usual design philosophy is to accomplish dissipation of energy through controlled inelastic deformations. It is, therefore, inherent in the design process to expect even well designed structures to sustain some damage during strong earthquakes. Furthermore, as building codes are updated, some of the existing structures may not comply with current standards even though they may have been properly designed and constructed according to earlier building standards. Therefore, some repair and/or strengthening must be performed at some point in the useful lives of many structures located in seismically active zones either by requirements of building codes or by decision of the owner out of concern for safety.

In the case of earthquake resistant structures, the task of retrofitting an existing structure must ensure that the strength and stiffness of the overall structure is restored to the level of the original design requirements or to the level specified by new regulations. One way to accomplish this is to restore or strengthen each member and connection to the desired level and the other is to add new members or structural systems such as shear walls and diagonal bracing systems that are capable of resisting most of the lateral load. Most of the time, however, a combination of the two approaches will render the best solution.

In addition, it must be assured that, in the event of a subsequent earthquake, failures do not occur at undesirable locations as a result of the strengthening process and that the structure has adequate ductility. Furthermore, the modifications introduced must not interfere with the functions of the structure nor be in detriment of its appearance. The economic impact of projects of this type are often significant since the cost of repairs not properly designed or executed can be very high even though the cost of the repair itself may be small compared with that of building a new structure. As a result, it is essential that the solutions opted must be reliable.

Arriving at a solution that satisfies the above mentioned considerations can be a complex problem since the pattern of damage depends on a variety of variables such as geometric characteristics, quality of materials, construction details, etc. Furthermore, the extent of damage and its impact on the characteristics of the structure are difficult to quantify which further complicates the problem. Thus, in general only guidelines are available and the specific solution to a particular repair and strengthening problem is perhaps more an art learned from experience than a science. Testing, evaluating and improving the procedures and the materials used in repairing helps with the understanding of how these efforts affect structures, reduce the degree of uncertainty, and raise the degree of confidence in those procedures.

1.2 Background

Repairing, strengthening, and stiffening of reinforced concrete structures has been performed for many years and many times with satisfactory results. However, the adequacy of these efforts was generally assessed by the structural performance under service loads only. The response of repaired structures during strong earthquakes is of growing concern to researchers and practicing engineers.

Since the 1960's many tests have been performed on structural members repaired using epoxy injection. This type of repair has proven to be effective in bonding cracked portions of simple elements such as beams and columns where the patterns of damage are relatively simple [3]. In the case of beam-to-column connections, however, substantial slippage of reinforcing bars can occur without extensive surface cracking. Injection of epoxy in surface cracks may not restore the bond of reinforcing bars passing through the connection [28].

When damage is too extensive and injection of epoxy is not practical, recasting the damaged region and/or enlarging the cross sections of damaged members has been attempted successfully [2,15]. With this approach the damaged portion of concrete is removed and new concrete is cast or shotcreted to replace the removed portion and/or enlarge the section of the member. This type of repair is expected to perform well under service loads. However, the performance of interfaces between the new and old concrete and the behavior of reinforcement splices at large deformations is largely unknown. Restoring bond along reinforcing bars within the connection can still be a problem.

A large number of damaged structures are repaired by adding new structural elements, such as shear walls and bracing members, in order to correct deficiencies in strength, stiffness or dynamic characteristics. Several investigation [4,6,7,8,10,11,12,13,14,22] on this approach to repairing and strengthening of structures. Such repairs rely on new members or structural systems to resist at least partially the applied loads. In such repairs, it is important to insure that the connections between existing and new members function properly within the entire range of expected load levels. Some basic techniques of implementing these connections have been tested, starting with the design of dowels [16,17,18], to the shear capacity of interfaces between new and existing concrete [1].

Results of testing of repaired structural members, connections, and frames are reported in references 1, 9, 19, 20, 27 and 28. Properties of materials used in repairs are reported in references 15 and 17. With the increasing attention to the problem of upgrading concrete structures, several state-of-the-art reviews, guidelines and recommendations for research are available [5,11,23,24].

In most of the structures that have been repaired in the field and in many laboratory investigations, it appears that the attention has been mostly given to increasing the strength of damaged columns and, in some cases, to that of damaged beams. The restoration of moment resisting connections, however, has been generally neglected. While efforts have been made to restore bond by epoxy injection, though with relatively little success, alternate economical and practical means of transmitting moments from beams to columns have yet to be found. Furthermore, most of the tests reported in the literature are based only on a single type of repair and on very limited number of specimens. As a result, it is difficult to compare the performance of various repair techniques, so that a practicing engineer can select the repair procedure most suitable to a specific situation.

1.3 Objectives

This investigation was aimed at determining the effect of various repair procedures on the behavior of frame subassemblies subjected to earthquake-type loading. Within this scope, a special attention was given to the behavior of connections between beams and columns.

For this purpose, several moment resisting frame subassemblies were tested under a predefined loading routine and repaired using various repair procedures. The repaired specimens were then subjected to the same loading routine as used in the undamaged specimens. Thus, by comparing the response of the repaired specimens with those of the virgin specimens, the effectiveness of various repair techniques could be ascertained.

1.4 Scope

The investigation focused on the repair of moment-resisting frames damaged by earthquake action. The procedures evaluated were those aimed at restoring or improving the characteristics of members and connections rather than adding new structural elements such as shear walls or diagonal braces. The main parameters of interest were strength, stiffness, and energy dissipation capacity.

A total of eleven specimens were subjected to cyclic loading of characteristics similar to those expected from earthquake excitation. Strains, displacements, cracking patterns, and other response parameters were recorded at suitable intervals during each test. After each specimen was tested, it was repaired using procedures that are commonly used in practice. Modifications and improvements to those procedures were introduced where considered appropriate. The specimens were then tested under conditions that duplicated those during the original test and the same measurements recorded whenever possible. The observed behavior of the repaired specimens was then compared to that of the original specimens to evaluate the effectiveness of various repair techniques.

SECTION 2

TYPES OF DAMAGE AND REPAIR TECHNIQUES

2.1 Types of Damage.

Damage sustained by a structure can range from minor cracking or spalling of concrete to hinging of the members or total collapse of the structure. When the damage is severe it may be appropriate to modify the basic characteristics of the structural system in addition to repairs and strengthening of individual members. This study addresses the issue of damage sustained by individual members and their connections. Some of the commonly encountered damage sustained during an earthquake are described below:

2.1.1 Minor Cracks

Minor cracks can occur either in beams or columns, particularly near the connections. This type of damage is consistent with the usual design philosophy and the only repair necessary may be for cosmetic purposes unless the cracks are large.

2.1.2 Cracking and Spalling

Spalling occurs mainly near the connections due to stress concentration ans excessive compressive stress in concrete. Superficial spalling may require only localized replacement of concrete. However, when the core concrete is seriously damaged, a more extensive repair of the damaged portion may be necessary. In these cases, the strength of concrete must be restored by injecting bonding agents or by replacing the damaged concrete. In addition to these repairs, it may be necessary to strengthen the member or the connection to insure proper performance in future events.

2.1.3 Shear Cracking

Diagonal cracks usually follow flexural cracks and may indicate that the member has lost some of its shear capacity or that the shear capacity was insufficient to start with. The members that exhibit appreciable diagonal cracking should be strengthened to restore or increase its shear resistance.

2.1.4 Joint Shear Cracks

Properly designed structures are proportioned so that members develop their full strength before damage occurs in the connections. When connections exhibit considerable cracking, it may be necessary to replace the damaged concrete, increase confinement, provide for shifting of damage to connecting beams or some combination of these techniques.

2.1.5 Bond Failure and Slippage of Bars

When connections are subjected to large deformation reversals, a progressive deterioration of the bond between longitudinal bars and concrete occurs inside the connections. This process produces fragmentation of concrete surrounding the reinforcement passing thru the joint, which may not be detectable from the surface. This type of damage results both in loss of strength as well as deterioration of stiffness.

2.1.6 Buckling of Reinforcement

Spalling of concrete under compression is usually followed by the buckling of longitudinal bars which severely affects the capacity of the member. This type of damage occurs usually due to insufficient amount and large spacing of transverse reinforcement.

2.2 Repair Techniques

The repair technique best suited to a particular situation depends in general on the characteristics of the structure and the type and degree of damage. Therefore, only general guidelines are available instead of specific design procedures. Some of the more commonly used techniques are briefly outlined below.

2.2.1 Epoxy Injection

This is perhaps the simplest procedure to repair small cracks of widths less than 1/4 inch. Larger cracks and voids can be repaired with epoxies of high viscosity or with a mixture of epoxy and sand. The procedure starts with cleaning the cracks and placing injection ports at selected locations along the cracks and then sealing the surfaces of cracks. When the sealant has cured, an epoxy resin of low viscosity is injected under pressure. If the injection is done properly, the repaired concrete can be as strong as the original concrete. Ensuring that the epoxy fills all the cracks in the damaged zone can be difficult.

2.2.2 Replacement of Spalled Concrete

Spalling normally is a localized phenomenon and it reduces or eliminates the cover of reinforcement. Repairing this type of damage has mainly the purpose of restoring the protection and bond of reinforcing steel with concrete and the appearance of the structure. This can be accomplished by casting new concrete or by an epoxy based compound with similar characteristics. Many times formwork may not be necessary for this type of repair. The concrete mix must contain the minimum amount possible of water to eliminate shrinkage cracks at the interface with the old concrete. Alternatively, a bonding agent can be used.

2.2.3 Replacement of Core Concrete

When major damage is observed in a member, probably the strength of the member is no longer adequate and the damaged portion must essentially be rebuilt. This procedure must start with insuring the stability of the structure by temporary shoring or other adequate means. The damaged concrete is then completely removed and formwork built for casting the new concrete. Bond with old concrete can be accomplished as described above.

2.2.4 Addition and Replacement of Reinforcement

Reinforcement steel can be added readily when the section is being enlarged. In other situations providing bond with the existing concrete is in general more complicated. The existing reinforcing bars can be partially uncovered and the new bars splice-welded across the buckled length of the existing bars. The cover can then be restored by casting concrete or shotcreting. Replacing damaged bars can be also accomplished by fusion-compression welding or mechanical splicing to avoid problems of congestion created with lap splices.

2.2.5 Enlargement of Section

When considerable damage is detected in beams, columns or connections it may be necessary to strengthen the damaged elements. It is however important to perform an overall structural analysis to insure that the changes in stiffness introduced do not produce adverse distributions of internal forces. It is also advisable to insure that the zones of likely damage in future events are confined to portions of the beams outside the connections. This is commonly accomplished by encasing the member with a reinforced concrete jacket. The surface of the damaged member is first roughened and new reinforcement is placed around it; then the forms are erected and new concrete is poured. A major difficulty encountered here is the placement of transverse reinforcement which usually requires drilling thru old concrete and perhaps bending of bars in place. Furthermore, because of the small clearances it is difficult to insure that no cavities are left in the new concrete during the casting process and that proper bond is developed at the interface with the old concrete.

2.2.6 Addition of External Rolled Steel

The strengthening of members can also be accomplished by placing rolled steel elements external to members. These can be angles located at the corners and/or plates bonded to the member surfaces. The transfer of stresses between the old concrete and the reinforcement is accomplished by a bonding agent and/or dowels anchored in the concrete. Methods commonly used to protect steel structures can also be used for the added steel elements. A deficiency that has been noticed in buildings where this type of repair has been used is that the transfer of stresses at the joints was not properly addressed.

SECTION 3

EXPERIMENTAL PROGRAM

3.1 Original Specimens

The test specimens consisted of moment resisting frame subassemblies both in the form of beam-to-column connection subassemblies and as two bay frame subassemblies. Most of the specimens also included a floor slab and transverse beams. Out of a total of eleven specimens, five were single connections between beams and columns and the rest were subassemblies composed of three columns connected by a beam as shown in figure 3-1.

3.2 Design of Specimen Repairs

The type of repairs used in each specimen was chosen on the basis of the observed damage and the desirability of evaluating the procedures described earlier. In general, it was concluded that injecting epoxy in small cracks would be an economical way to restore the strength of the existing members at least partially. For this reason, epoxy injection was the first step in the repair procedures used for all specimens. Table 3-I presents a summary of characteristics of the specimens and repair procedures used in each case . The details of procedures used in each specimen are described next.

3.2.1 Specimens CR and CTBR

During the test before repairs, these two specimens suffered considerable damage in beam segments adjacent to the joints and, in the case of specimen CR, the joints and part of the columns were also damaged as shown in figure 3-2. The deterioration of bond between the longitudinal reinforcement and the concrete inside the joint can be seen in figure 3-3 where the loose concrete has been carefully removed. The damage inside the joint in specimen CTBR could not be observed due to the presence of transverse beams although flexural cracks were visible in the columns as shown in figure 3-4. The method chosen to repair such extensive damage was replacement of damaged concrete. In the case of specimen CR, the concrete within the joints and adjacent portions of beams and columns was removed as shown in figure 3-5a, while in the case of specimen CTBR only the damaged portions of beams were removed as shown in figure 3-5b. The purpose of removing concrete from the joint and its vicinities in specimen CR was to use this somewhat ideal type of repair as a basis for comparison with a more realistic case such as that of specimen CTBR. In both cases forms were built and new concrete having the same proportions as the one used to build the specimen originally was poured. The bond between the old and the new concrete was ensured by applying epoxy based bonding agent to the old concrete surface.

3.2.2 Specimen CS1R

Specimen CS1R exhibited a large number of cracks of width less than 1/8 in. wide in beams, columns, and the slab as seen in figure 3-6. In addition, some spalled concrete less than 1 in. deep was also removed at the intersections of beam and column surfaces. The epoxy injection method was chosen as the main repair procedure for this specimen and the spalled concrete was replaced with a mixture of high viscosity epoxy and sand. The aim here was to compare this often used technique with others from the point of view of the behavior of the repaired specimens.

3.2.3 Specimens CS2R and CS4R

These two specimens exhibited damage in the form of flexural and diagonal cracks in beams and columns near the connections. In particular, the center column in specimen CS2R was damaged to the point (figure 3-7) that its contribution to lateral load resistance was considerably reduced. For these reasons, it was considered appropriate to strengthen the columns by encasing with a reinforced concrete jacket.

The thickness of jacket was based on clearance requirements to properly place new concrete and to provide adequate cover. The details of the jacket, shown in figure 3-8 were determined such that both the bending and shear capacities of the jacket alone were greater than those of the original section. Thus, the repaired columns would be at least as strong as the original columns even if the contribution of the damaged portions was totally lost.

The transverse reinforcement in specimen CS2R consisted of cross ties placed in holes drilled in the beams with each end bent in place to form a 135 degree hook. Similar cross ties were also placed in the columns. Due to difficulties described later, this procedure was modified by using dowels with a standard hook at one end in specimen CS4R. These dowels penetrated only half of the beam width and their adequacy in providing bond between the jacket and the column core was based on the fact that the enlarged column section was expected to have a relatively low level of compressive stress. Figure 3-9 shows specimens CS2R and CS4R with the added reinforcement in place. A close-up view of the reinforcement detail is shown in figure 3-10.

The formwork was designed to allow casting of new concrete in two phases. In the first phase, the concrete was cast around the columns up to an elevation six inches below the beams. In the second phase, the space below the slab was filled from the top through holes drilled in the slab to pass the column longitudinal reinforcement (figure 3-11). Prior to placing the form work, an epoxy based fresh concrete bonder was applied to the column surface. The concrete was designed to have a slump of 5 to 6 inches to facilitate flow of concrete in narrow clearances.

3.2.4 Specimen CS3R

This specimen was originally designed to move the plastic hinges a certain distance along the beams away from the joint. During testing of the virgin specimen, the desired behavior did occur and considerable damage could be observed at the predefined hinge locations especially near the external joints. The damage was in the form of severe flexural and diagonal cracks accompanied by the spalling of cover at the bottom of beams and buckling of longitudinal reinforcement. In addition, the top cover of transverse beams at the external joints was lost due to inplane compression forces in the slab.

In repairing this specimen, concrete was removed and the buckled portions of reinforcement were spliced with new bars. Anchor dowels were added at the top of transverse beams to resist the compression forces from the slab, and the damaged concrete replaced with new concrete. The details of this repair are shown in figure 3-12.

3.2.5 Specimen ER

Specimen ER was used to test in more detail the behavior of a joint where only the column was strengthened with a reinforced concrete jacket. The details are similar to those used in specimen CS4R except that this specimen did not have the transverse beams and the slab.

3.2.6 Specimens IR and ES1R

These two specimens were used to extend the method of encasing members with reinforced concrete jackets to the cases where bond damage was severe along the longitudinal bars inside the joints. The repair procedure consisted of encasing the column as well as a portion of the beam adjacent to the column. New reinforcement, shown in figure 3-13, was added to the beams to help reduce the bond stress requirements in the existing bars and to increase the flexural and shear capacity of the beam adjacent to the columns.

3.2.7 Specimens ES2R and ISR

The alternative of using external steel elements instead of a reinforced concrete jacket was tested in these two specimens. The details of reinforcement are shown in figure 3-14. Plates bonded to the surfaces of the columns and anchored with dowels were used to transmit forces to the joint by means of angles placed at the corners. These in turn transferred the forces to the beams through an enlarged section around the joint. In the case of specimen IS1R, a plate was also bonded to the bottoms of the beams to increase the bending capacity of beam segments adjacent to the joint. Figures 3-15 and 3-16 show the damage in the beam before the repairs and some of the details of reinforcement around the joint for these two specimens.

3.3 Test Setup

The subassemblies were tested in a steel reaction frame as shown in figure 3-17. Columns were pin-connected at the top to a beam capable of horizontal movement. A single actuator was used to impose predefined cyclic displacement routine at the top ends of the columns. The bottom ends of the columns were connected to hinges that rested on hydraulic jacks for the application of axial load to the columns. A stationary link beam connected to the bottom hinges provided lateral restraint to all columns. This type of testing arrangement takes into account the P-Δ effect experienced in actual buildings. Figure 3-18 shows the actual test set up during testing of a multiple-connection subassembly. The single-connection assemblies were tested in a similar manner except that the beam ends were supported on rollers to simulate the point of inflection at mid-span as shown in figure 3-19.

3.4 Loading Routine and Instrumentation

The specimens were subjected to a predefined displacement routine that consisted of 13 cycles of increasing amplitude. As shown in figure 3-20, certain cycles were repeated to measure the degradation of strength. Small amplitude cycles were also introduced into the loading routine to measure the lateral load stiffness of the specimen near zero displacements. Each specimen was externally instrumented to record various response parameters. The measurements and miscellaneous data recorded during each test consisted of load-displacement curves, axial forces in the columns, the distribution of lateral load among the columns, strain measurements in beam, slab, and column reinforcement, rotation of the beam flexural hinging region, and the distribution of cracks in beams, columns, and the slab where present.

TABLE 3-I Characteristics of Specimens and Repairs

Specimen	Description	Repairs
CR	Three Columns connected by beams	Replace damaged concrete; beams and columns
CTBR	Transverse beams added to CR	Replace damaged concrete; beams only
CS1R	Floor slab added to CTBR	Injection of epoxy only
CS2R	Similar to CS1R	Jacket columns with reinforced concrete
CS4R	Similar to CS1R	Similar to CS2R
CS3R	Similar to CS1R; Plastic hinge moved away from joint	Splice buckled bars, replace damaged concrete
ER	Single exterior joint, no slab	Jacket column with reinforced concrete
IR	Single Interior Joint, no slab	Jacket column and part of beam with reinforced concrete
ES1R	Single exterior joint with a slab	Jacket column and a part of beam with reinforced concrete
ES2R	Single exterior joint with a slab	Add external steel to column only
IS1R	Single interior joint with a slab	Add external steel to both column and beams

Note: All specimens were injected with epoxy to seal small cracks (less than 1/4 in.) before the above mentioned repairs were performed.

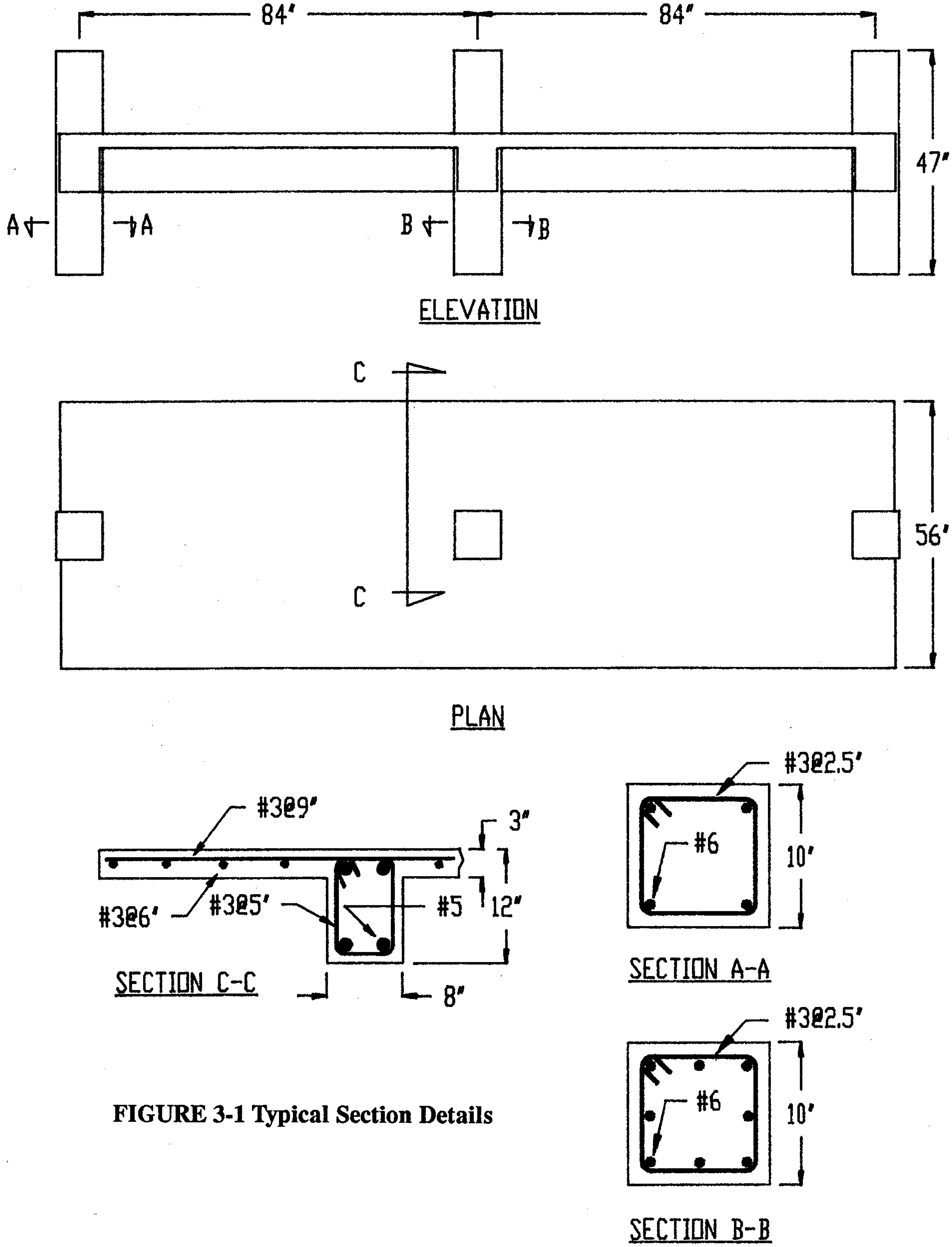
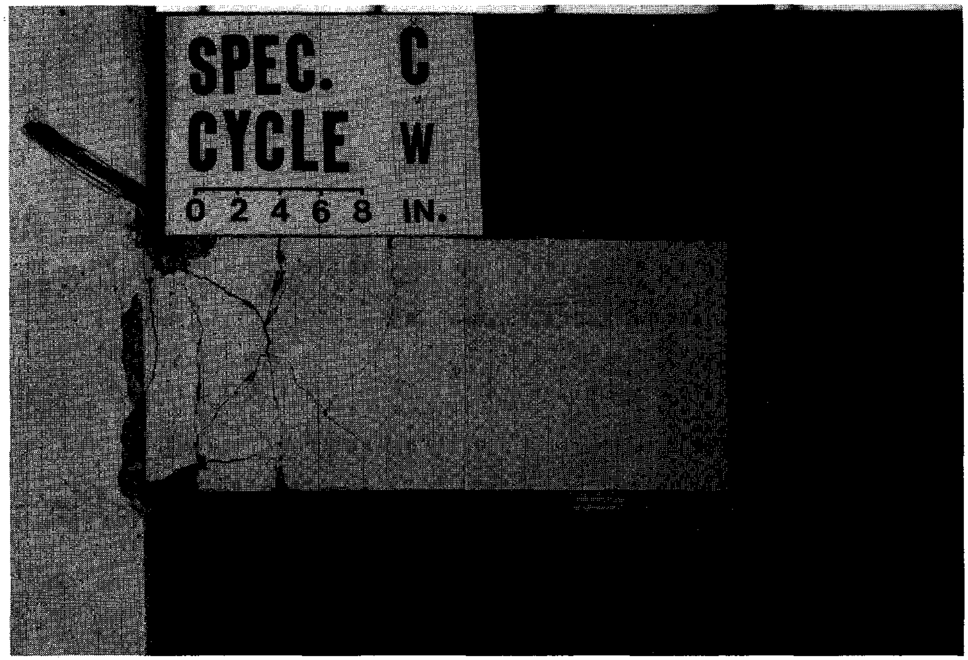
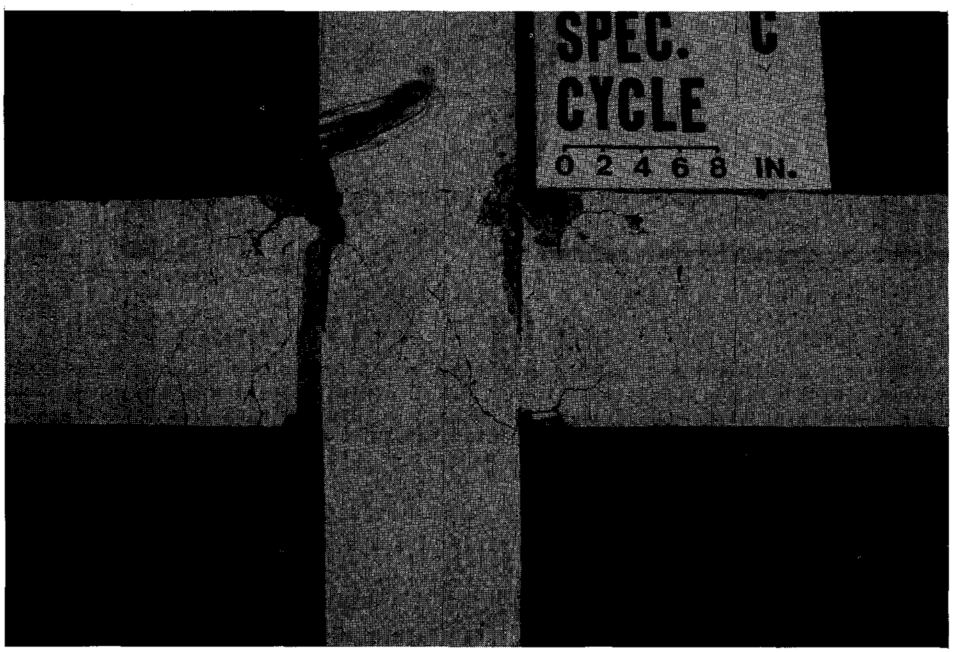


FIGURE 3-1 Typical Section Details

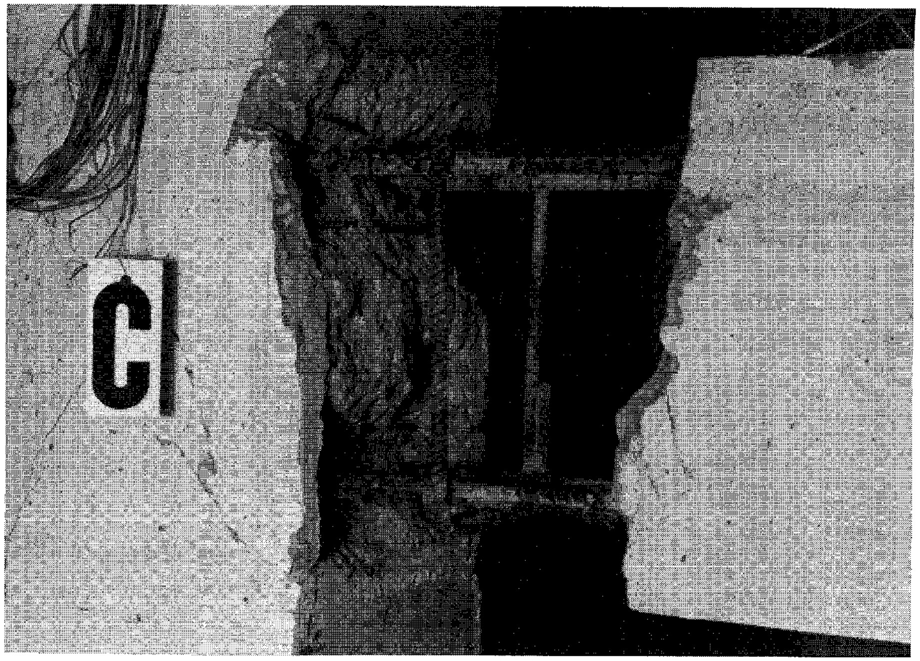


a) Exterior joint

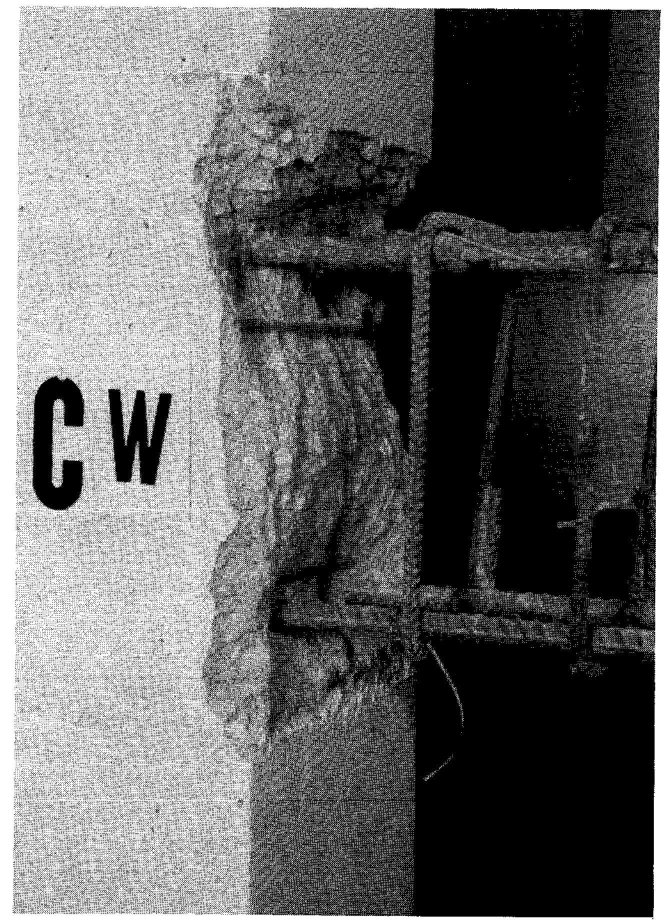


b) Interior joint

FIGURE 3-2 Damage in Specimen CR Before Repairs

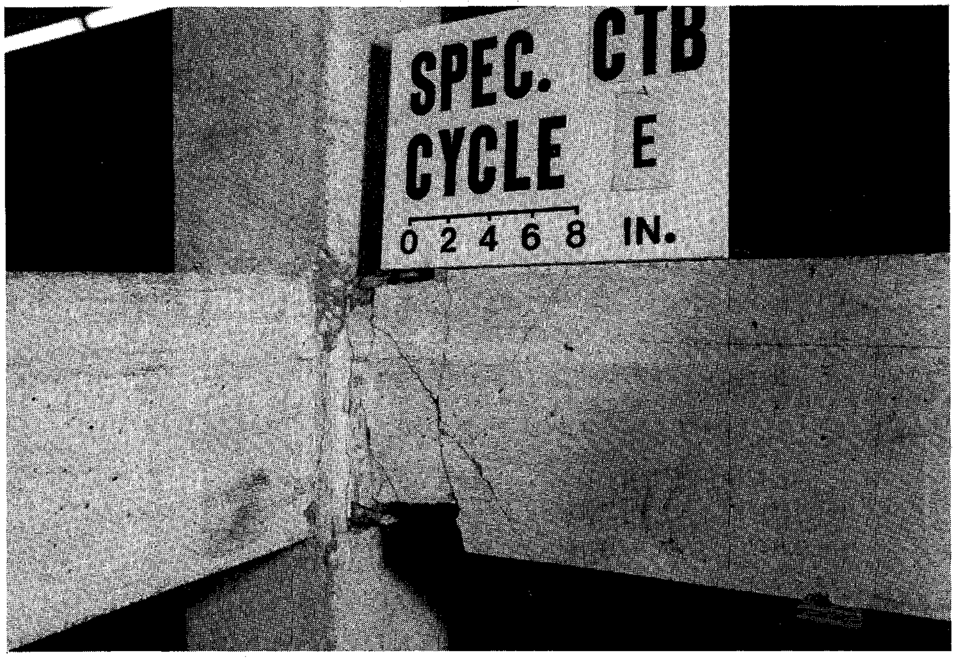


b) Interior joint

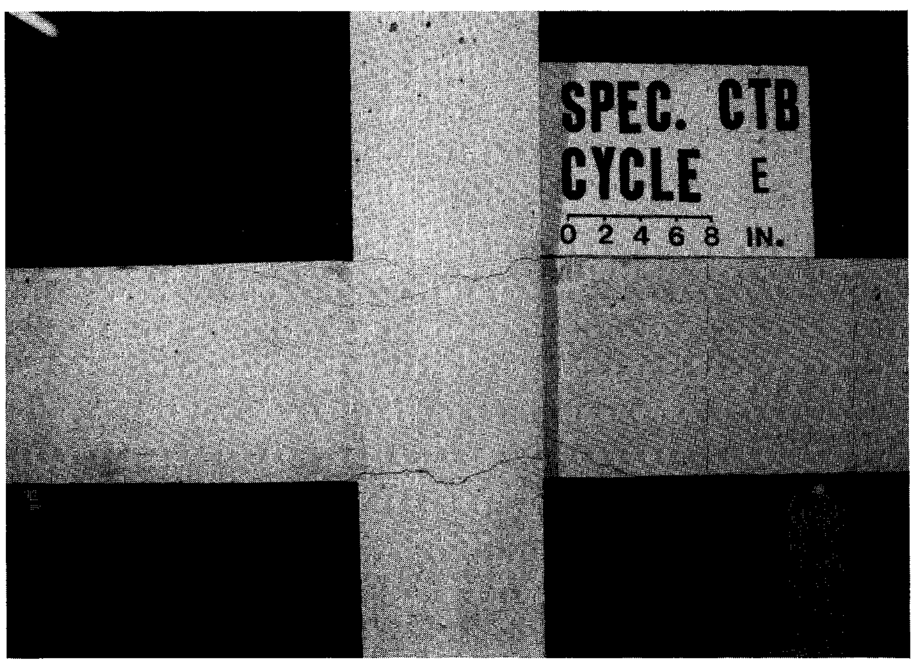


a) Exterior joint

FIGURE 3-3 Specimen CR after Removing Damaged Concrete



a) Beam damage in exterior joint



b) Column damage in exterior joint

FIGURE 3-4 Damage in Specimen CTBR Before Repairs

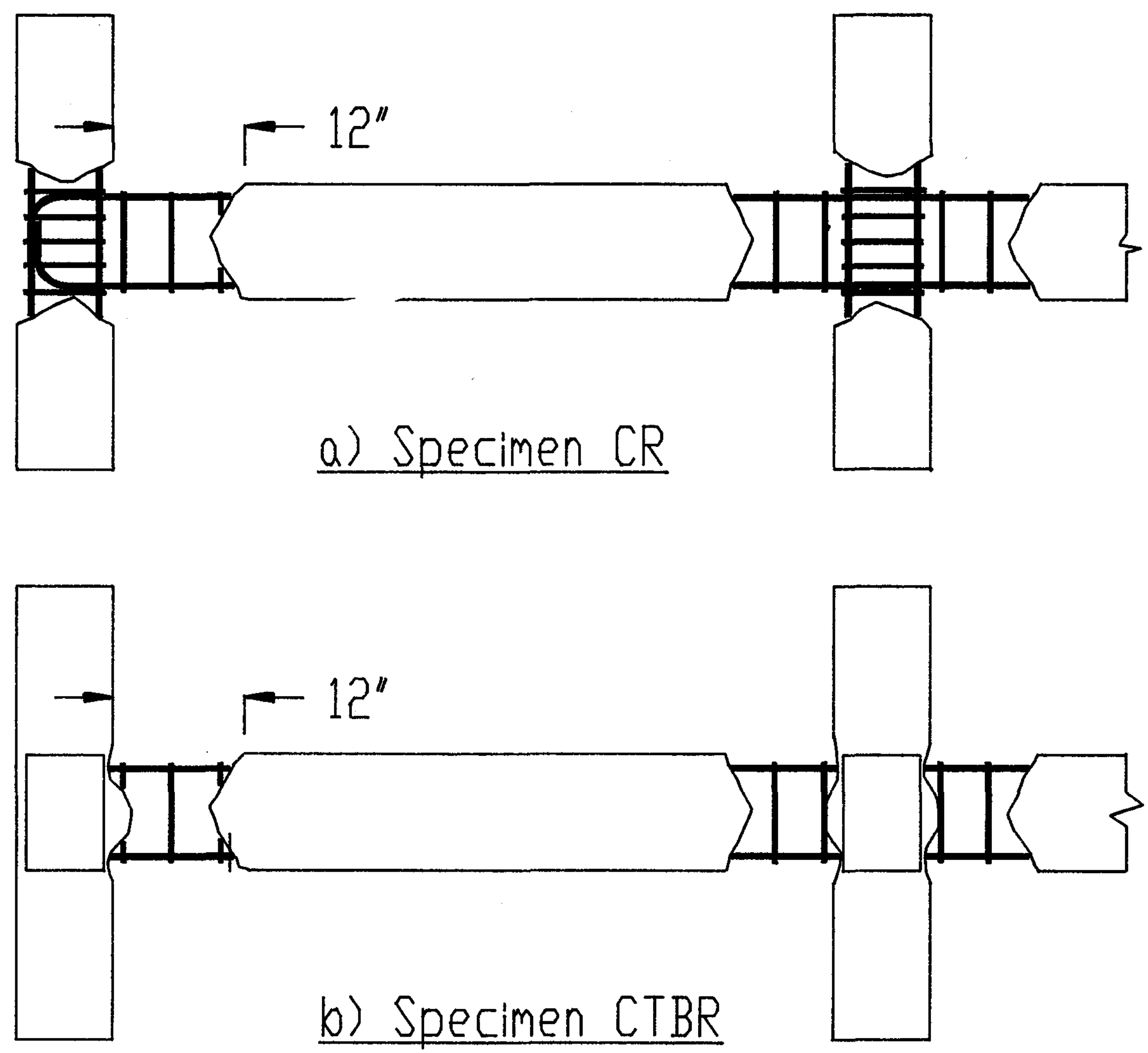


FIGURE 3-5 Damaged Concrete Removed in Specimens CR and CTBR

FIGURE 3-6 Specimen CS1R Before Repairs

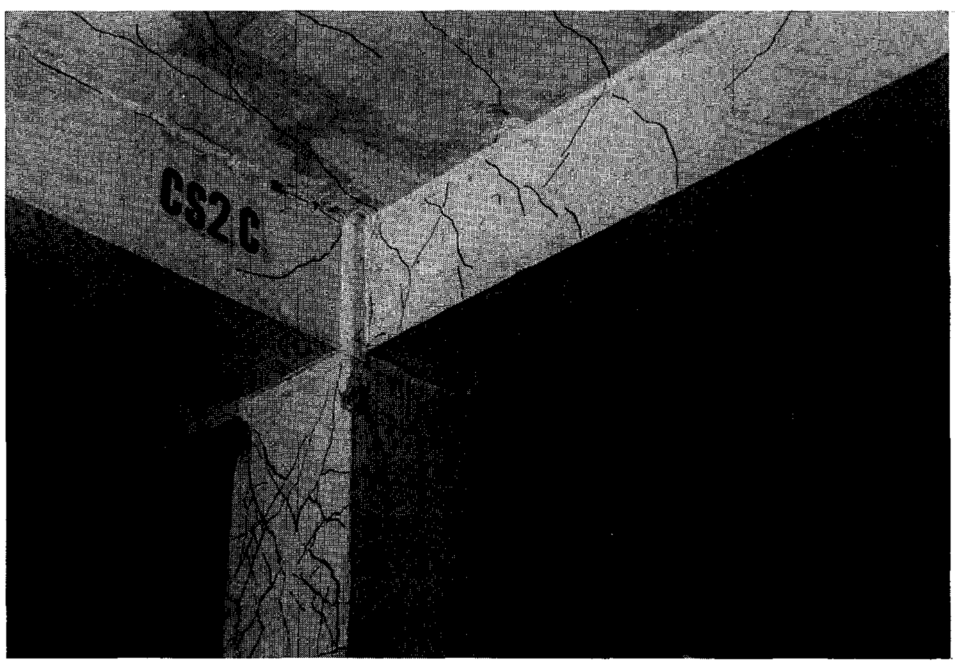


FIGURE 3-7 Specimen CS2R Before Repairs

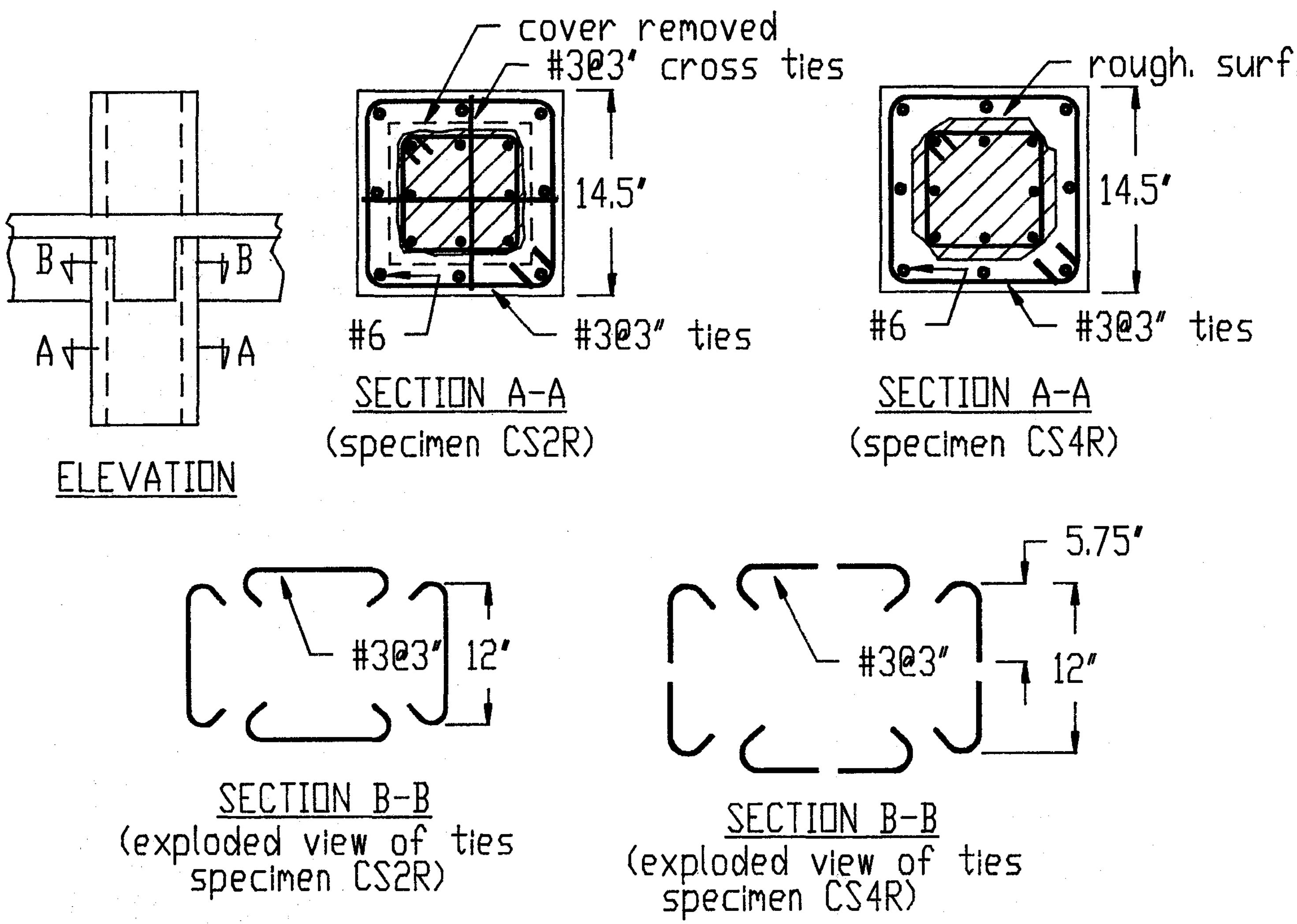
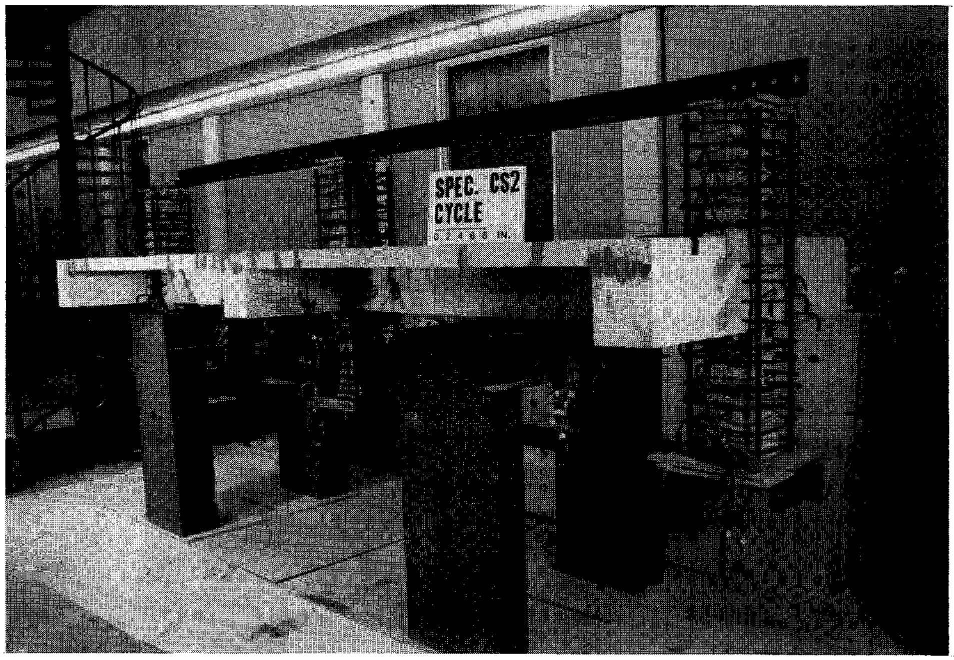


FIGURE 3-8 Details of Reinforced Concrete Jackets



a) Specimen CS2R

b) Specimen CS4R

FIGURE 3-9 Steel Reinforcement of Concrete Jackets

FIGURE 3-10 Detail of Jacket Reinforcement Under the Slab

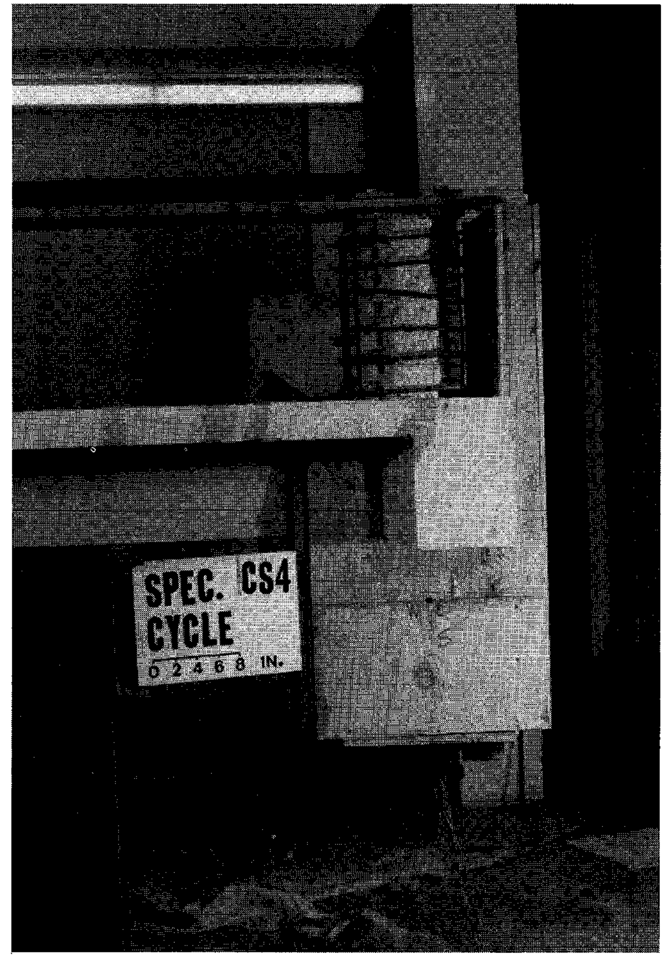


FIGURE 3-11 Formwork for Casting Concrete Under the Slab

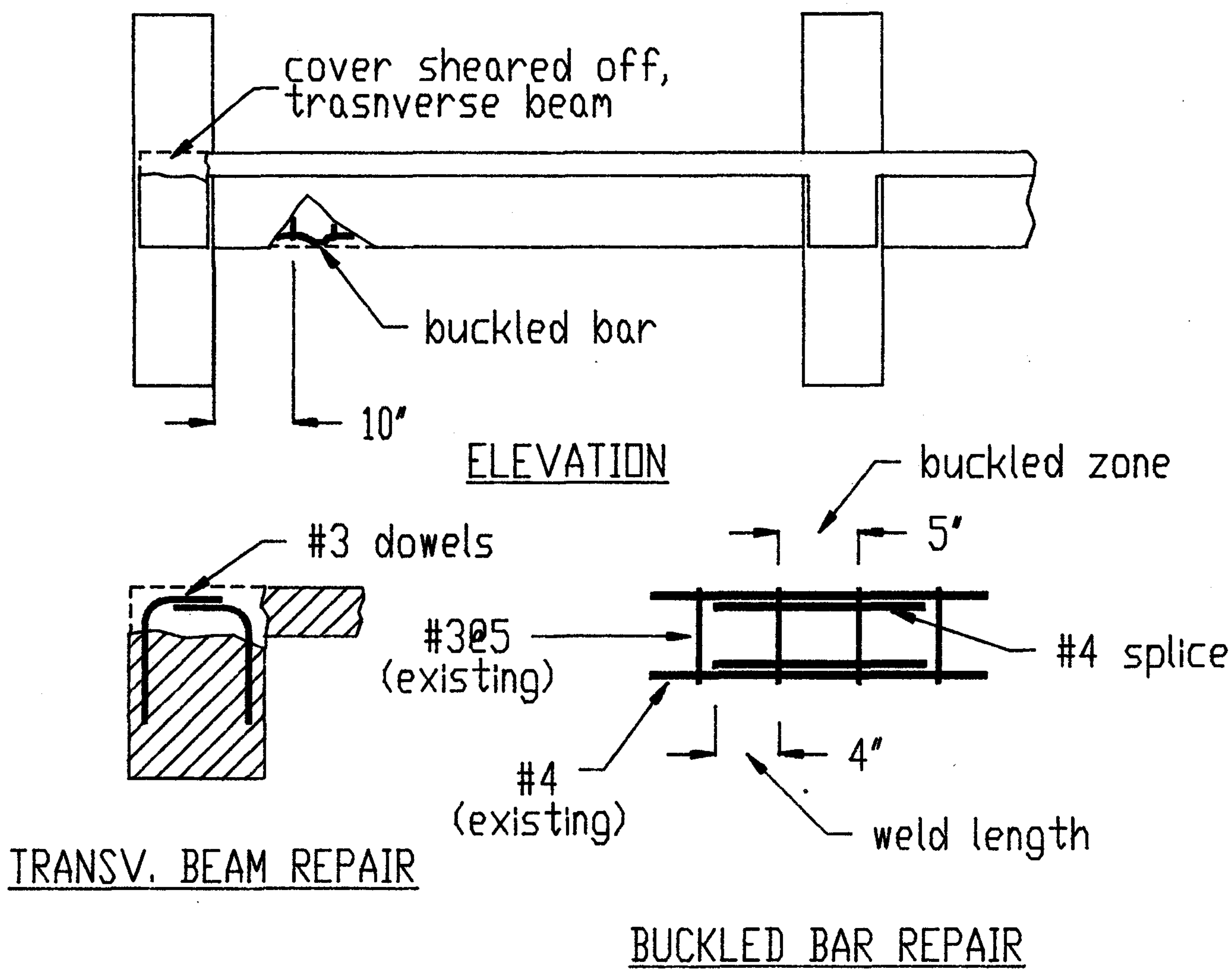


FIGURE 3-12 Details of Repairs on Specimen CS3R

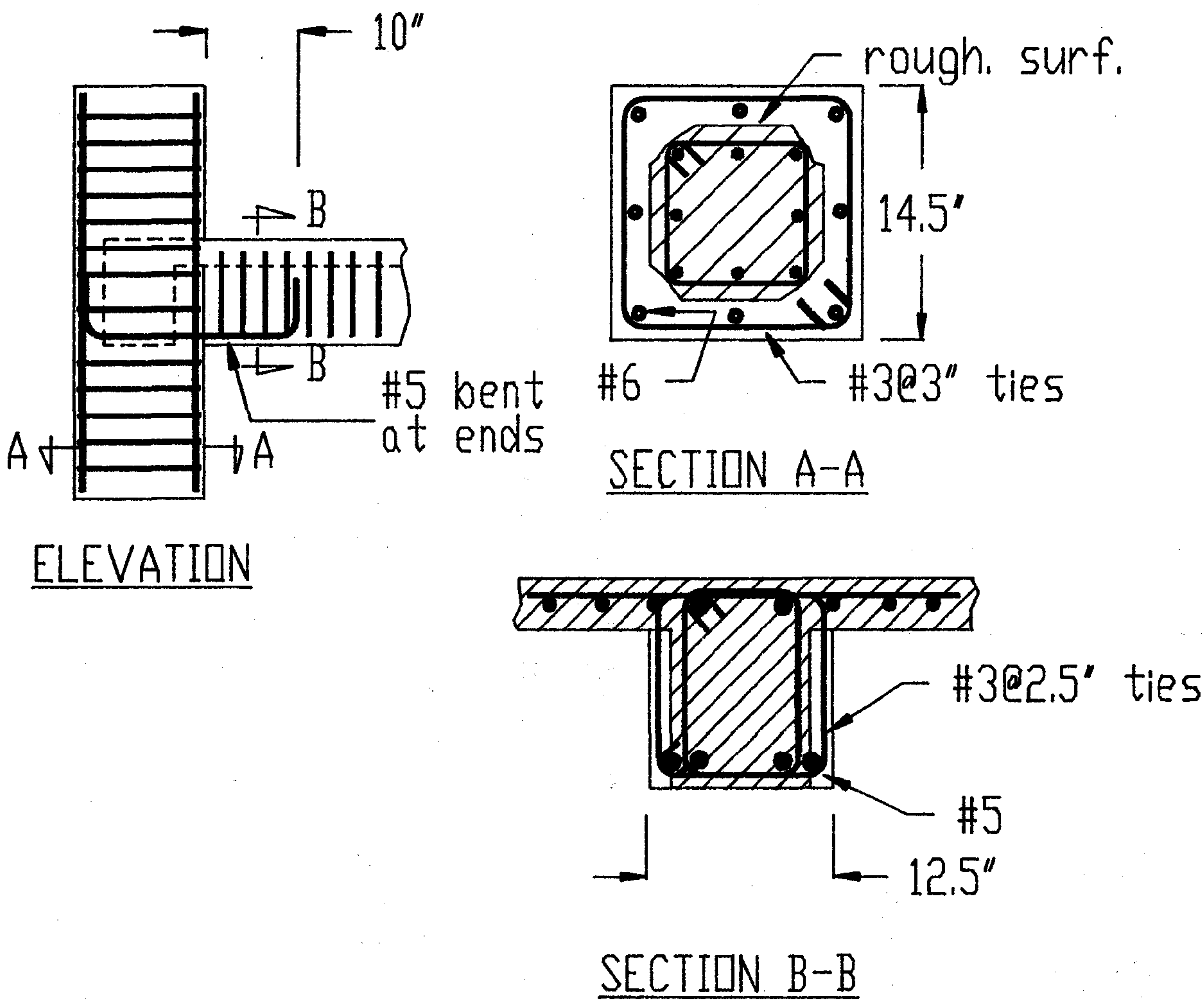


FIGURE 3-13 Jacketing Detail for Column and Beam Segments

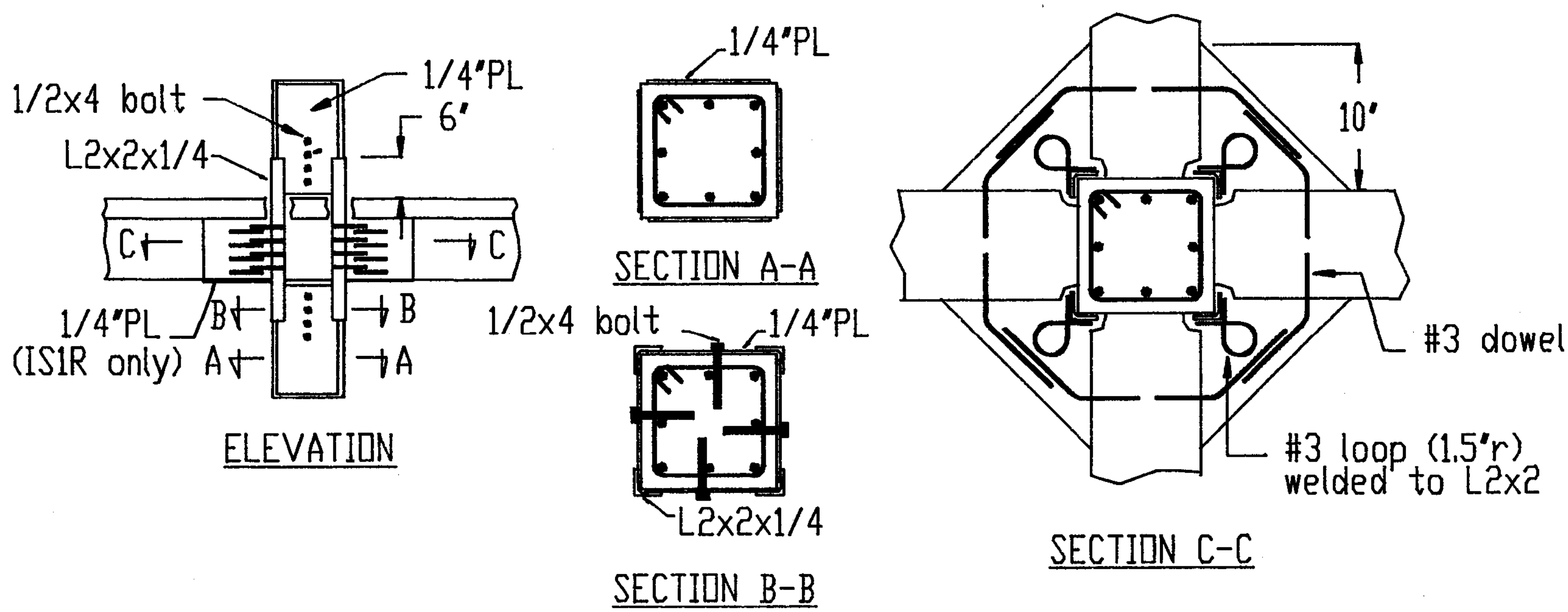
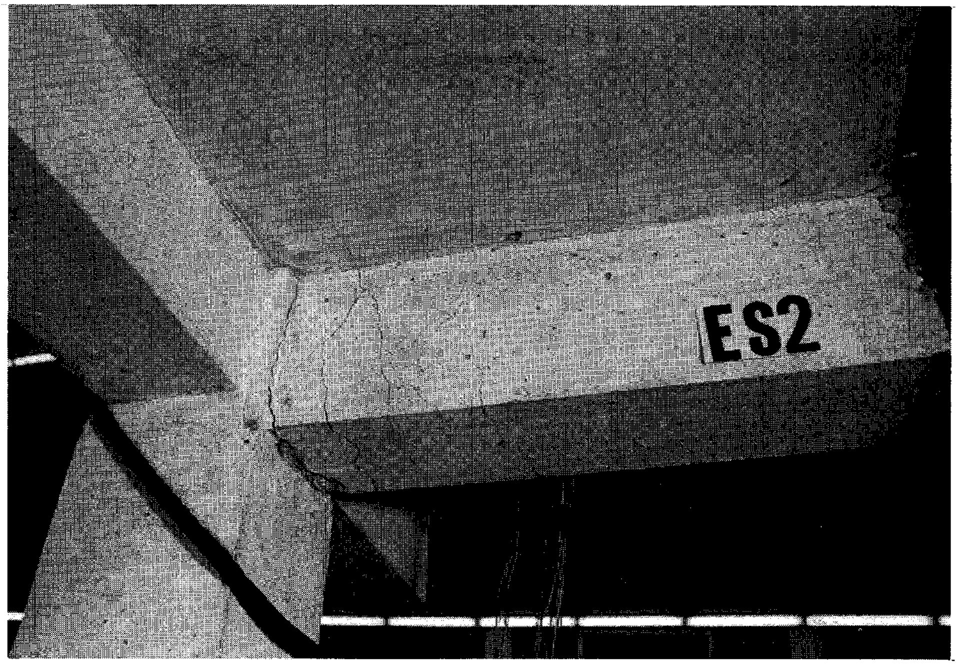
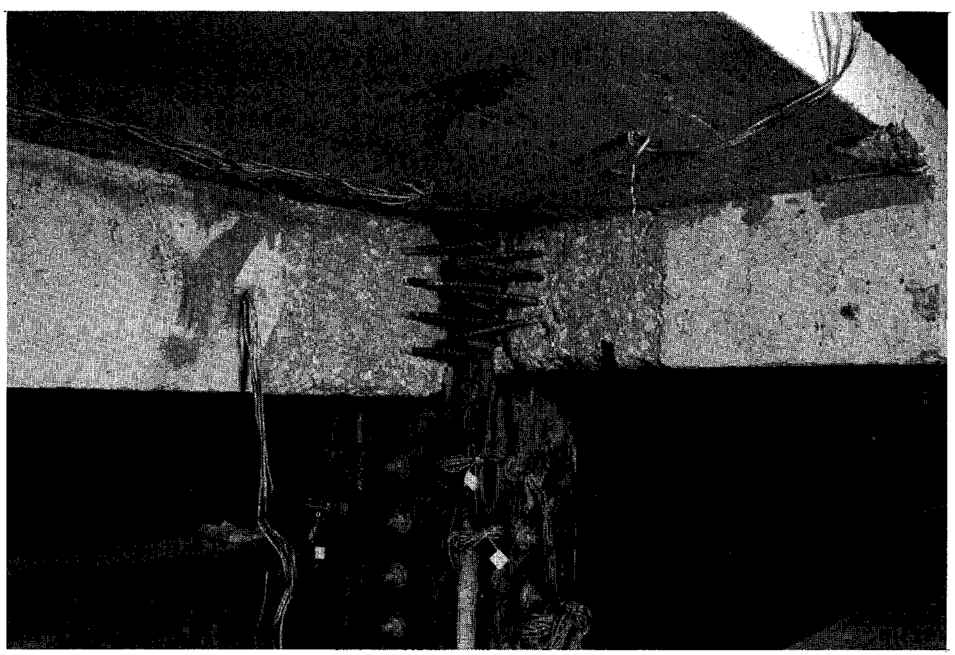


FIGURE 3-14 Details of External Steel Reinforcement



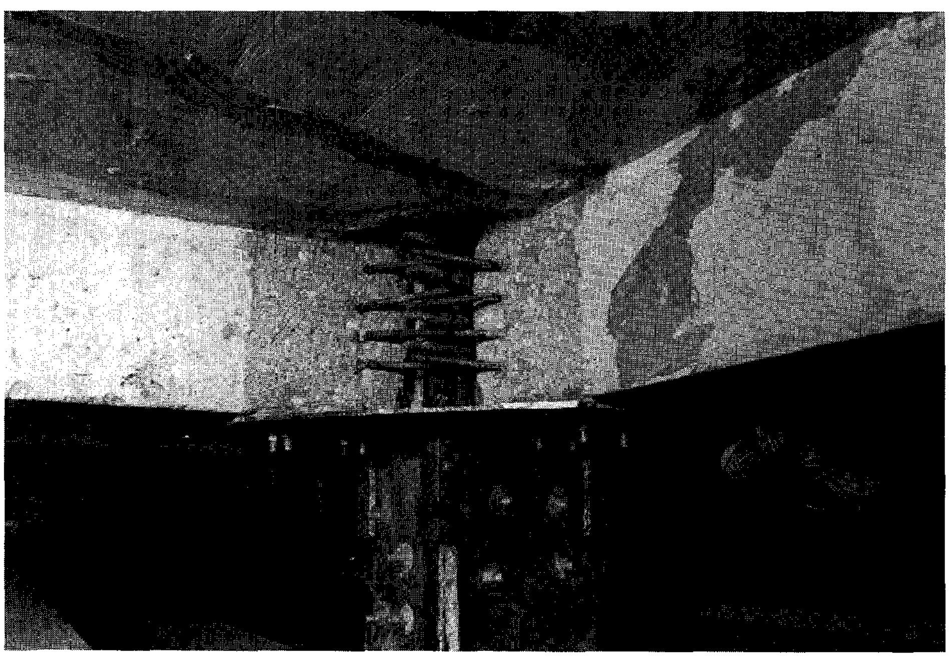
a) Damage before repairs



b) Column and joint reinforcement details

FIGURE 3-15 Repair of Specimen ES2R

a) Damage before repairs



b) Column and joint reinforcement details

FIGURE 3-16 Repair of Specimen IS1R

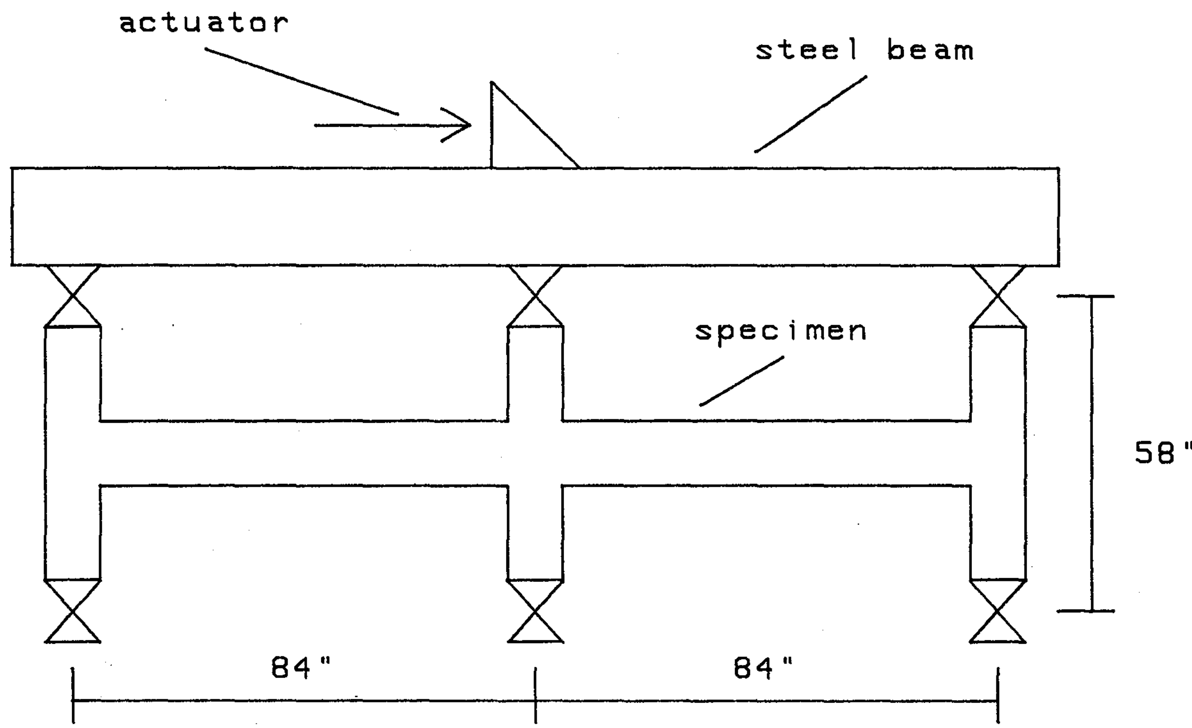


FIGURE 3-17 Test Setup Detail

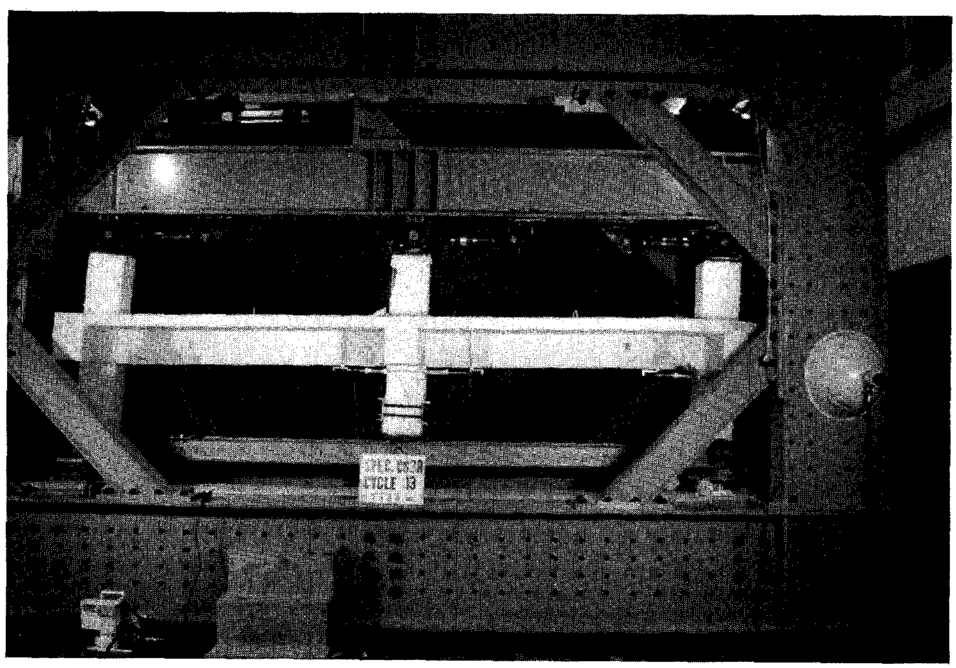
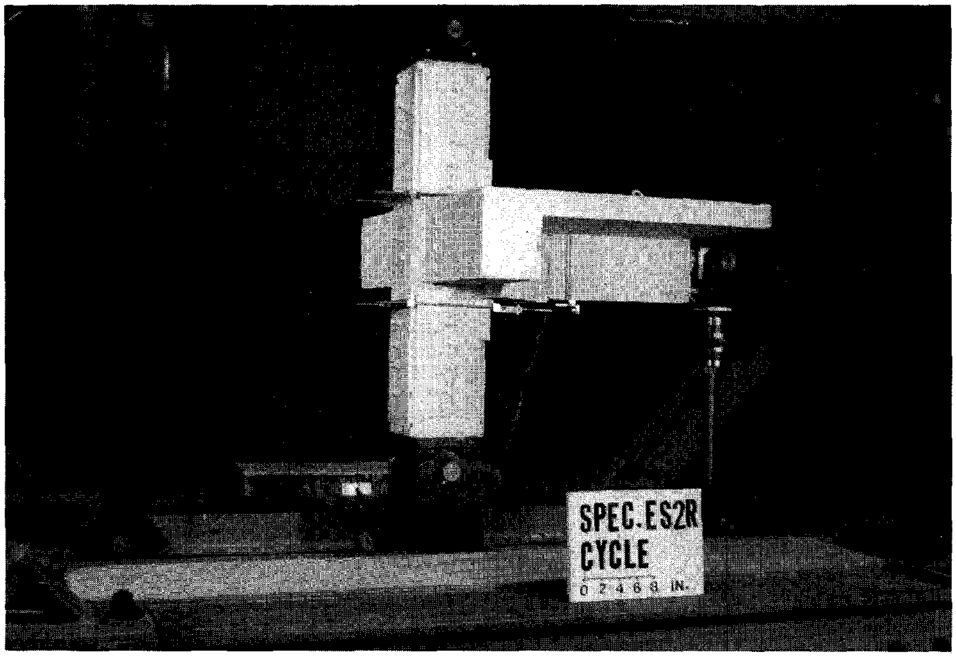
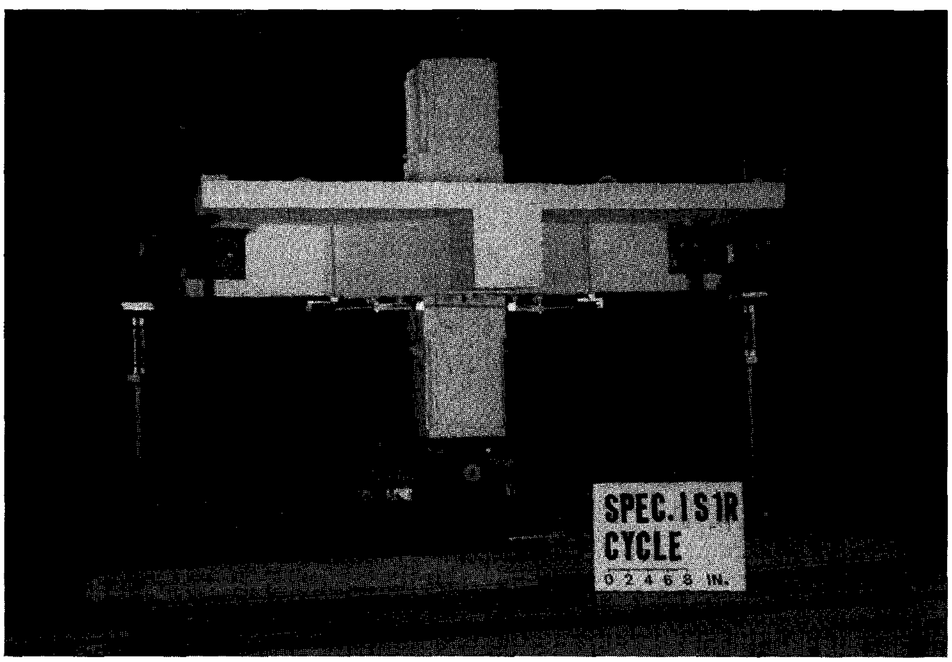


FIGURE 3-18 Multiple-Connection Subassembly in the Testing Frame



a) Exterior joints



b) Interior joints

FIGURE 3-19 Test Setup for Single-Connection Specimens

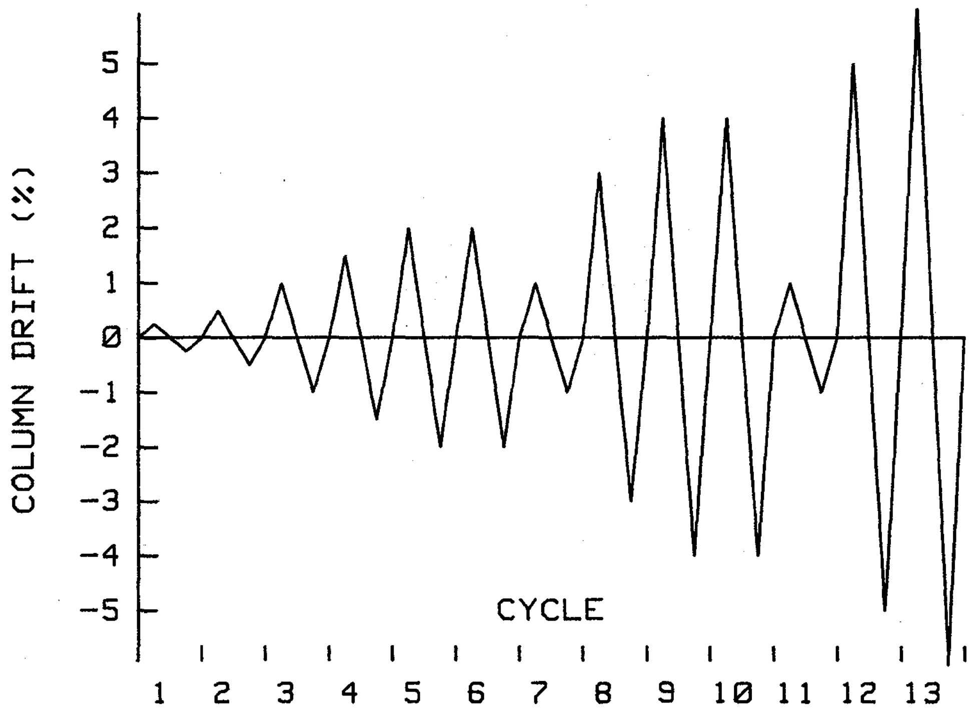


FIGURE 3-20 Displacement Routine

SECTION 4

RESULTS

4.1 Parameters Of Response

When evaluating the structural response of a specimen, whether it is repaired or not, the parameters of most interest are the load carrying capacity or strength, the stiffness, and the energy dissipation capacity. In addition, since the loading applied to the specimens studied was of cyclic nature, the variation of these parameters in each cycle of the response helps understand the behavior of the specimens. Plots of the type shown in figure 4-4c are used in this report to present a comparison of the total lateral load vs. lateral inter-story displacement loops, also referred to as hysteresis loops, before and after the repairs of specimens. In these plots, the load and displacement are normalized with respect to the peak load and peak displacement of the corresponding cycle in the test before repairs.

Figure 4-1 shows the salient characteristics of a typical load vs. displacement loop of response during a cycle. The parameters defined in this figure are the displacement and load amplitudes, the energy dissipated during the cycle, and a total of four stiffness quantities. Three more stiffness values could be defined on the negative side of the loading cycle but these are not used here for the sake of brevity. These parameters are in general interdependent but they are useful in studying various aspects of the response. The remainder of this section presents in general terms the significance of these parameters.

4.1.1 Load Amplitude

Since the tests performed were of the displacement-controlled type, the load amplitude depends mainly on the stiffness of the specimen during the smaller displacement amplitude cycles. This is due to the basically linear behavior at this stage. As the displacement amplitudes increase, the load amplitude is limited by the yelding of steel, crushing of concrete, and the slippage of bars through the joint. The occurrence of damage during a cycle is manifested by reductions of slope in the load vs. displacement curve and in the shape of future load-displacement loops.

A comparison of the variation in load amplitude during the tests before and after repairs is presented for each specimen in plots of the type shown in figure 4-5a. The peak displacement in terms of percentage of column height is shown by a dotted line.

4.1.2 Stiffness

Stiffness is generally defined as the ratio between a change in load and the corresponding change in displacement. This definition, which is unique in linear structures, becomes much less clear in the case of concrete structures due to the highly nonlinear nature of their response at loading levels expected during strong earthquakes. For the purpose of comparing the behavior of the specimens, the definitions shown in figure 4-1 are used in this study. These stiffness parameters are useful in describing quantitatively the shape of the hysteresis loops and in investigating various aspects of the specimen response.

4.1.2.1 Secant Stiffness

This parameter, defined as the slope of the line that joins the points of extreme displacements in the positive and negative loading directions, is useful to represent the overall response of the specimen during a given cycle.

4.1.2.2 Zero-Displacement Stiffness

The slope of the hysteresis loop at points of zero displacement, when compared with reloading or unloading stiffnesses, provides a measure of the "pinching" or narrowing of the hysteresis loop at small displacements. The pinching of the hysteresis loop can be due to the presence of cracks and slippage of bars due to loss of bond which prevent, at least partially, the concrete and the steel from being stressed when the displacements are small. For this reason the zero-displacement stiffness can be a measure of the damage sustained by the structure during previous cycles. The zero-displacement stiffness also gives a measure of the response of the specimen to loads that produce small displacements.

4.1.2.3 Reloading Stiffness

When the displacements increase beyond the pinched region of the loops but before they reach levels that produce new damage, the remaining undamaged components of the specimen begin to resist load and temporarily exhibit essentially an elastic behavior. This is manifested in the straight portion of the hysteresis loop immediately following the pinched region. The slope of this segment is called the reloading stiffness. This stiffness, when compared with the initial stiffness, provides another measure of damage suffered during the preceding cycles.

4.1.2.4 Unloading Stiffness

After reaching the peak displacement, whether positive or negative, the test subassembly undergoes the unloading process. The first part of this process is generally non-linear due firstly to the fact that the strain in the newly damaged concrete under compression is not completley recovered when the load is reduced and secondly to the unloading of the yielded reinforcement. The unloading process after this initial portion and before the pinched zone is reached, is characterized by another quasi-elastic branch that represents the behavior portions that are left undamaged up to the last peak. The slope of this approximately linear portion of the hysteresis loop is defined as the unloading stiffness.

The variation of these parameters on a cycle-by-cycle basis is presented in plots of the type shown in figure 4-6. The stiffness values represented in these plots are normalized by the initial stiffness of the specimen before the repairs. Note that the numerical value of a given stiffness parameter depends on the point that is chosen to measure the slope of the tangent and on the proximity of other data points. For this reason the plots of stiffness parameters present some irregularities which should not be interpreted in their strictest sense. In general, these plots are useful to study tendencies and make comparisons between tests.

4.1.3 Energy Dissipation

The area enclosed by the hysteresis loop represents the energy dissipated by the subassembly during a given load cycle. This parameter is important since the ability to dissipate energy has a strong influence on he response of the structure to dynamic loading. Different procedures of various levels of complexity have been proposed to quantify damage based on a number of parameters that often include energy dissipation.

When relating energy dissipation to damage of concrete structures, one should consider that steel and concrete are damaged at very different rates. Although a reinforcing bar will eventually fracture when subjected to cycles of tensile and compressive yielding, the number of cycles necessary to reach this type of failure is in general much higher than the number of cycles necessary to produce serious damage in a concrete structure. The reinforcing bars may, however, buckle in compression which could be avoided with adequate detailing.

Total energy dissipation by a structure consists of (1) energy dissipated by steel reinforcement, (2) energy dissipated by friction along existing cracks in concrete which occurs mostly when the displacements are relatively small and is attributed to opening and closing of cracks, sliding along shear cracks, and slippage of reinforcing bars, and (3) energy dissipated during the formation of new cracks including the crushing of concrete which occurs when the displacements are sufficiently large to cause new damage.

Separate measurement of these components of energy dissipated during a test is not a simple task. The energy dissipation as the only criterion for damage may, therefore, not be suitable for use in an experimental study. The assessment of damage based on stiffness is simple to measure and can be easily related to the response of the structure. The dissipated energy is, therefore, used together with different stiffness parameters for evaluation of damage in specimens tested during this study. The total energy dissipated by the specimen in each cycle is presented in plots of the type shown in figure 4-5(b) where the energy is normalized by the product of displacement amplitude and load amplitude. Similarly, plots of the type shown in figure 4-5(c) show the ratio between cumulative values of dissipated energy and products of displacement amplitudes and corresponding load amplitudes.

4.2 Effectiveness of Repair Techniques

The effectiveness of repairs can be evaluated in many ways. The use of a single parameter as a criterion for the effectiveness of repairs may not be adequate in many cases. For example, if one considers the maximum load capacity alone, one might conclude that any repair, or none at all, is satisfactory since a properly designed structure is usually capable of resisting load levels near its original capacity when subjected to large enough displacements.

An appropriate way to evaluate the performance of a repaired specimen is to consider a combination of response parameters observed during the tests before and after the repairs. In doing so, one might recall that the zero-displacement reloading and unloading stiffnesses can be viewed as a measure of the damage suffered by the structure or the reserve available to sustain additional damage. The effectiveness of repair on a given specimen can be observed by noting the difference in its behavior between the last cycle of loading before the repair and the first cycle of loading after the repair.

4.2.1 Injection of Epoxy Resin (Specimens CS1R, CS3R)

4.2.1.1 Repair Procedure

Even though this is a commonly used procedure, some reasons for concern were found during the repair process and are summarized as follows:

1. In the flexural hinging region near the column face, the cracking pattern is in general more complex. The cracks are not only closely spaced but they may also be accompanied by spalling of concrete. As a result, insuring that every crack is properly filled with resin is difficult at best.

2. Some of the cracks that were quite visible during the test had closed and were almost undetectable when the loading was removed. Thus, the injection process was slow and the depth of penetration of the resin was some times in question.

3. Leakage of resin was some times a problem. This happened mainly due to the presence of small cracks that could not be detected and sealed properly during the preparation phase. Although every effort was made to stop such leakage when it was observed during the injection process, evidence of leakage was sometimes found only after the resin had hardened. This happened mainly while repairing the slabs where the resin was injected from the top surface.

After testing specimen CS1R, the examination of the damage revealed that some cracks had indeed not been filled with resin and, in some cases, the resin had leaked out of cracks which had been properly filled.

In view of these problems, the process of epoxy injection in specimen CS3R was repeated. The crack surface sealant was removed from the specimen and all cracks that had not been fully penetrated by epoxy were identified. The epoxy was then injected through injection ports instead carefully repeating the whole process. As shown below, this additional effort to insure the quality of the repair work improved the performance of the specimen considerably.

4.2.1.2 Overall Performance

The cracking patterns in specimen CS1R during the tests before and after repairs are shown in figure 4-2. The patterns are similar except that the crack in the beam at the face of the column is wider in the test after repairs indicating that slippage of reinforcing bars inside the joint had increased. Figure 4-3 shows the cracks in the top and bottom portions of the center column. Many of these cracks developed along the cracks formed during the test before repairs. In fact, the largest diagonal crack in the bottom part of this column was formed during cycle 12 along a crack formed before the repairs and the load capacity of the specimen decreased from this point on.

The cracking patterns in specimen CS3R before and after repairs are shown in figures 4-7 and 4-8, respectively. In this specimen, the beam reinforcement was detailed to move the flexural hinge away from the column face. Thus, most of the damage before repairs was concentrated at the predetermined location outside the joint. Spalling of concrete at the bottom face of the beam was caused by buckling of the reinforcing bars and the large crack at the end marked W was the result of shear failure at the top of the transverse beam produced by compression in the slab.

The repair of specimen CS3R included splicing of buckled bars, anchoring dowels at the top of the transverse beam and replacing damaged concrete with epoxy mortar. This repair procedure proved quite satisfactory since the specimen exhibited much less damage after repairs. Most of the cracking in the beam was closer to the joint due again to buckling of the bottom reinforcing bars at this new location. The epoxy mortar at the previous hinge location did not exhibit significant diagonal cracks and the top of the transverse beam did not fail in shear. After completing the test, the longitudinal bars at the damaged ends of the beams were uncovered and it was confirmed that these bars had indeed buckled and that the splice across the old buckle had survived the new test without signs of damage. The buckling of longitudinal bars in this specimen occurred due to the excessive spacing of stirrups in the plastic hinging region.

4.2.1.3 Load vs. Displacement loops

The load vs. displacement plots for specimens CS1R and CS3R are shown in figures 4-4, 4-5a, 4-9 and 4-10a. Some similarities are apparent between these figures if one looks at the peak loads in each cycle. In both cases the load amplitude in the test after repairs starts at about 60% of the corresponding value in the test before repairs and increases steadily until the maximum load capacity is reached in both cases. Recalling that the damaged concrete was replaced with epoxy mortar, which is a material with lower modulus of elasticity, the lower stiffness of specimen CS3R is to be expected.

There are also some important differences between the response of the two specimens. First, the shape of the loops after repairs are quite different. In the case of specimen CS1R, there is considerable pinching even in the early cycles, while the shape of the loops of specimen CS3R are basically the same as those before repairs. Second, specimen CS1R reached its maximum capacity during the twelfth cycle and did not reach its capacity before repairs. On the other hand, specimen CS3R exceeded its capacity before repairs and had not reached its peak by the end of the test. Based on these observations, it is apparent that the repair was more effective in restoring the performance of specimen CS3R than that of CS1R.

4.2.1.4 Dissipated Energy

Figures 4-5b, c and 4-10b, c show the plots of energy dissipated during the tests of these two specimens. It is to be noted that specimen CS1R dissipated more energy during small displacement cycles in the test after repairs while the opposite happened during cycles of larger amplitude. The increase in dissipated energy during small amplitude cycles is an indication of increased resistance along old cracks during the test after repairs and the decrease in dissipated energy during larger amplitude cycles suggests that fewer new cracks developed due to the loss of stiffness. Specimen CS3R, on the other hand, dissipated consistently at least as much energy during the test after repairs as it did during the test before repairs.

4.2.1.5 Stiffness Parameters

The stiffness parameters in each load cycle corresponding to these two specimens are shown in figures 4-6 and 4-11. In the case of specimen CS1R the stiffness values in these plots are ratios relative to the secant stiffness of the first cycle of the test before repairs. In the case of specimen CS3R, however, the normalizing factor used in each curve is the secant stiffness of the first cycle of the corresponding test. This difference in treatment of data is due to the inherent difference in stiffness due to the use of epoxy mortar in the case of specimen CS3R. The plots in the later case then can be used only to compare the rate of damage during both tests.

The secant stiffness and the zero load stiffness plots show significant changes in the performance of CS1R during the test after repairs. The stiffness of the repaired specimen in the early cycles was only about 50% of the initial stiffness before repairs. In addition, the initial stiffness of the repaired specimen was only 20 percent greater than the residual stiffness of the damaged specimen at the end of the test before repairs. This modest increase in stiffness reflects the effect of the repair procedure.

The reloading and unloading stiffnesses of CS1R show a very small change after repairs indicating that outside the pinched zone, where the crack openings and bar slippage reduce the stiffness of the specimen, there was no significant change in the behavior of the specimen.

In the case of specimen CS3R, there are only small differences in the rates of decay of stiffness parameters over the duration of the test. The most noticeable difference is in the unloading stiffness, where the repaired specimen shows an improved performance. This is attributed to the better performance of the epoxy mortar due to its higher tensile strength.

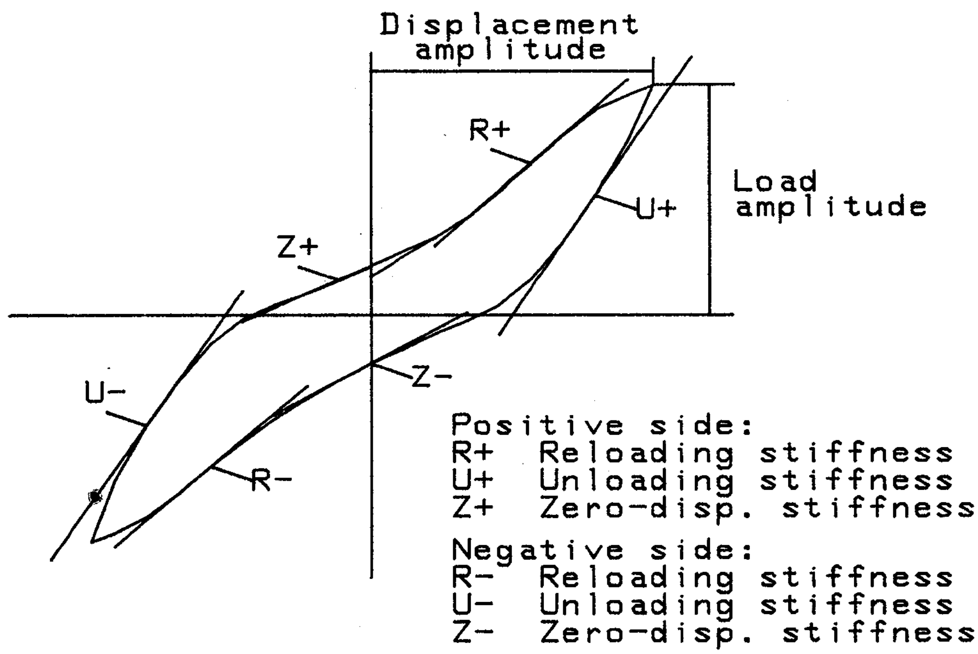
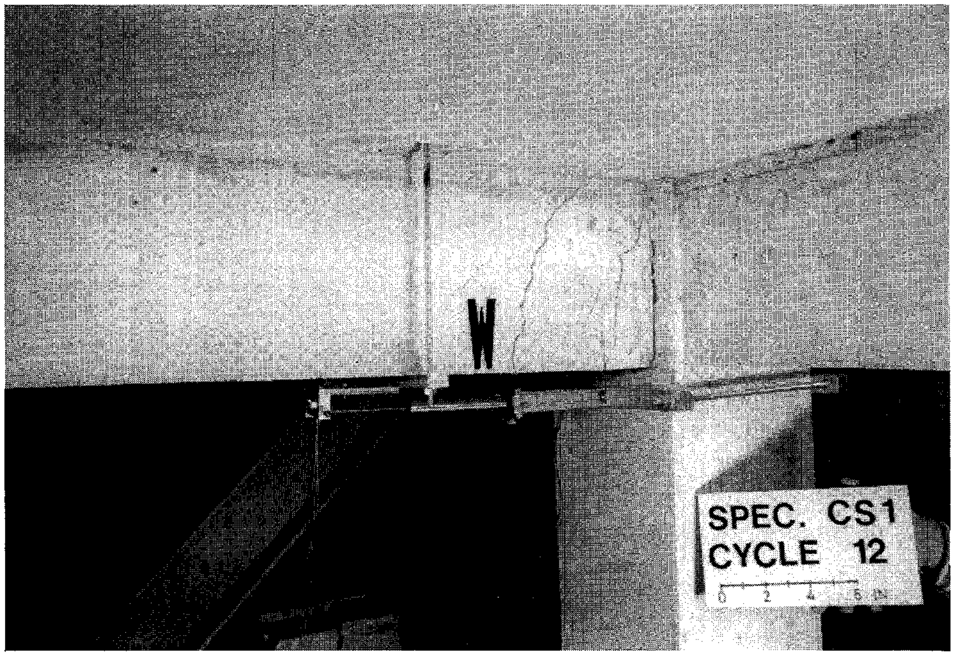


FIGURE 4-1 Characteristics of a Hysteresis Loop



a) Test before repairs

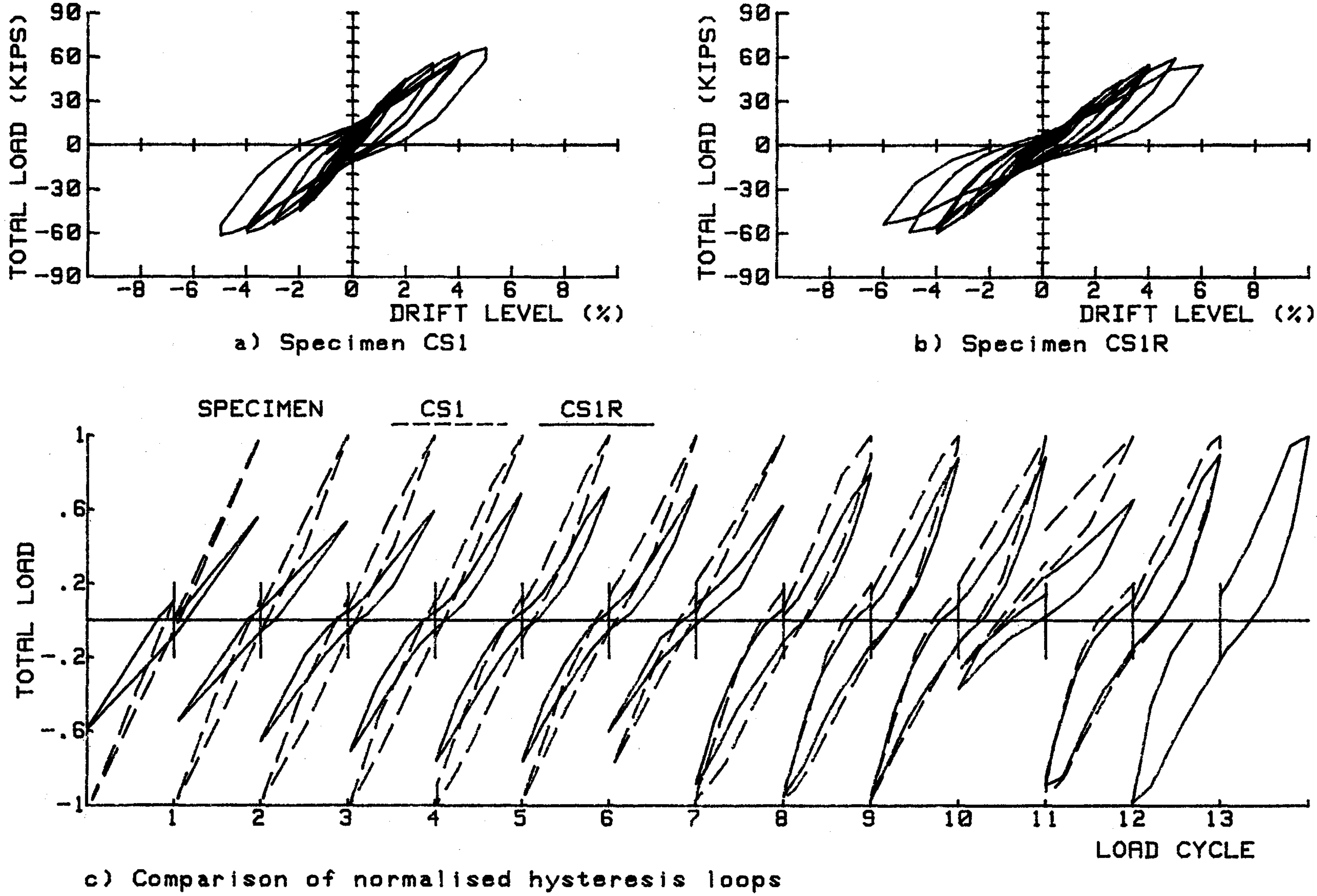
b) Test after repairs

FIGURE 4-2 Cracking Pattern of CS1R Before and After Repairs

a) Column cracking above the slab

b) Column cracking below the slab

FIGURE 4-3 Cracking in Center of Column of Specimen CS1R



a) Specimen CS1

b) Specimen CS1R

c) Comparison of normalised hysteresis loops

FIGURE 4-4 Load vs. Displacement Plots of Specimen CS1R

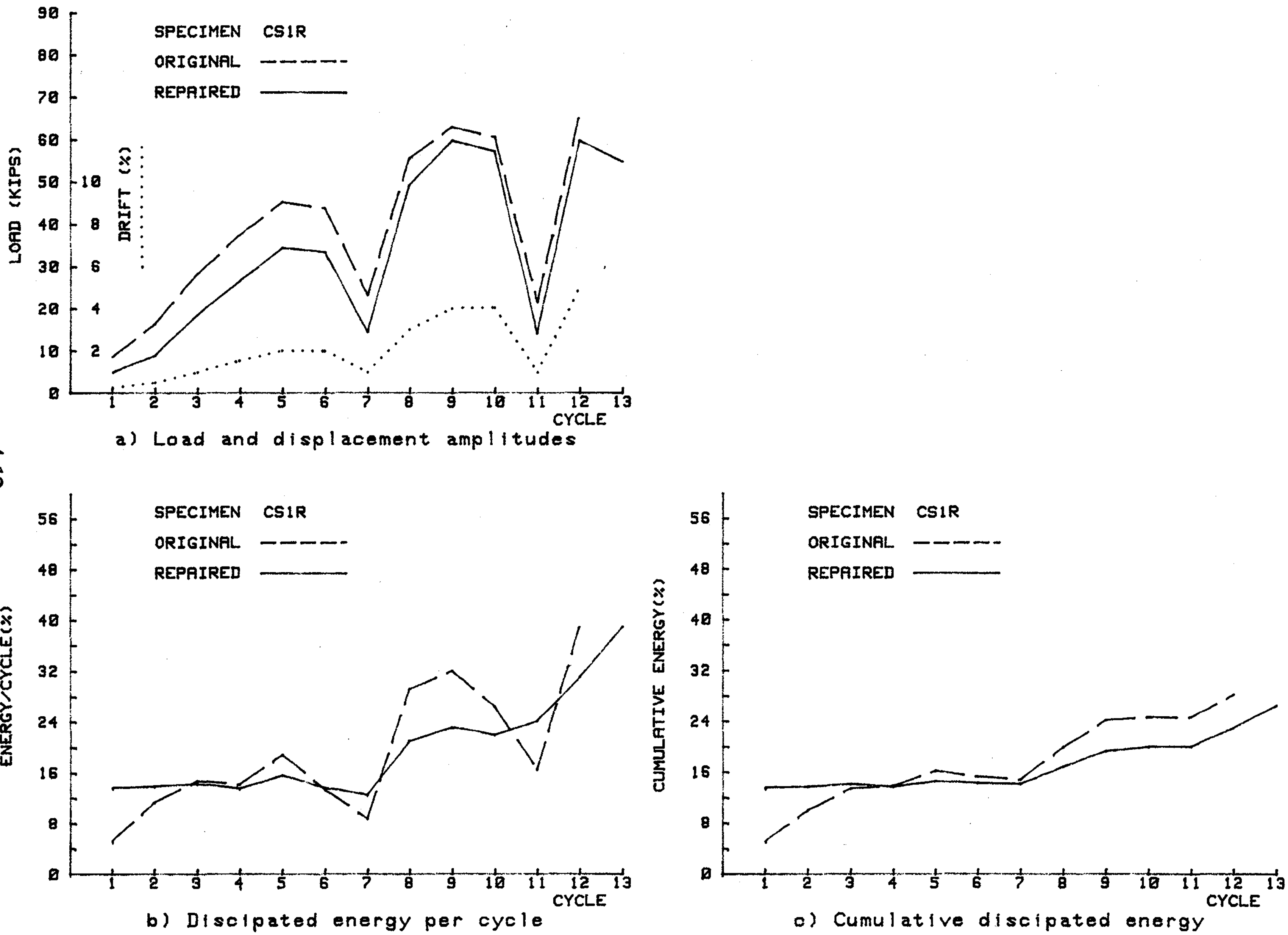


FIGURE 4-5 Strength and Energy Dissipation of Specimen CS1R

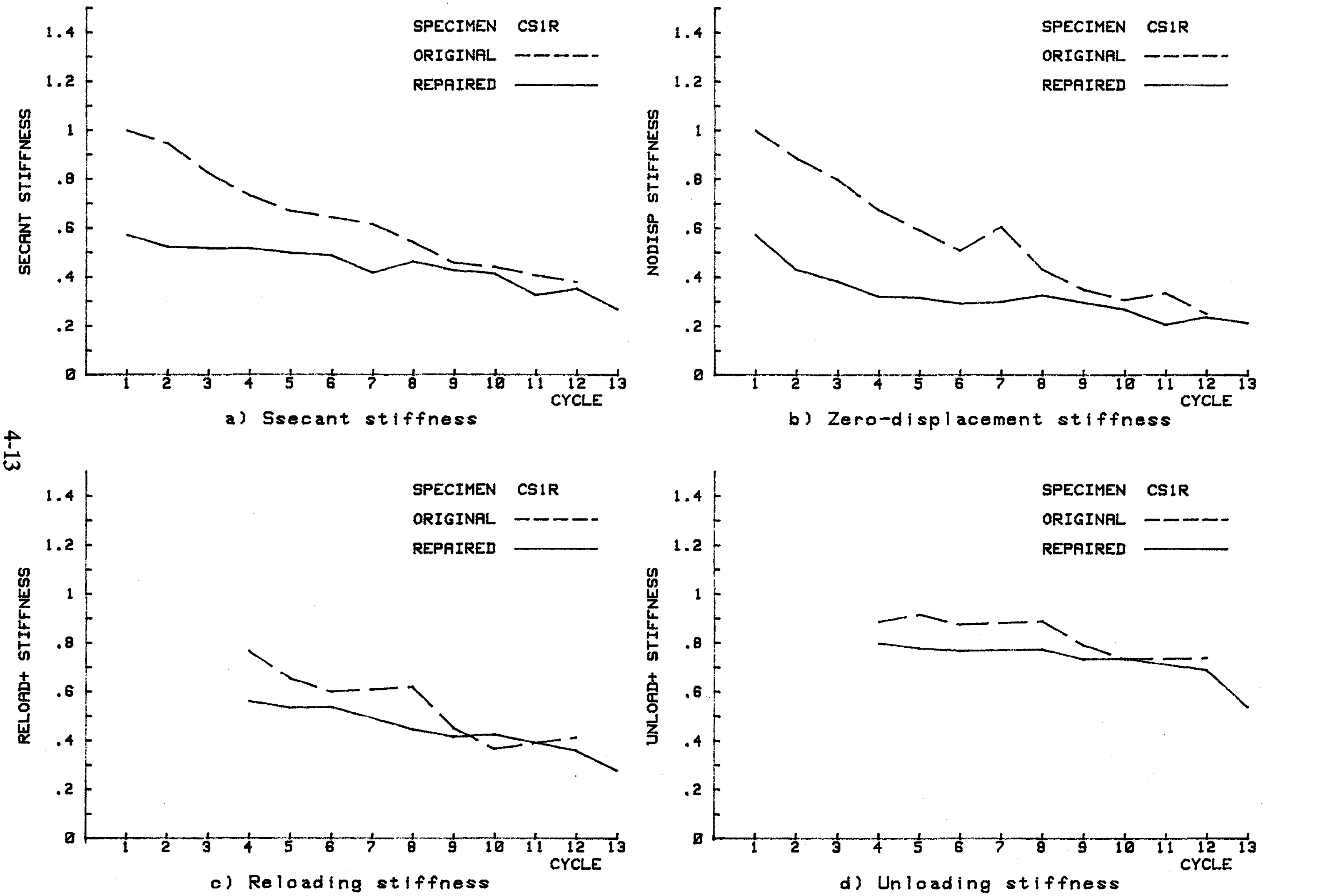
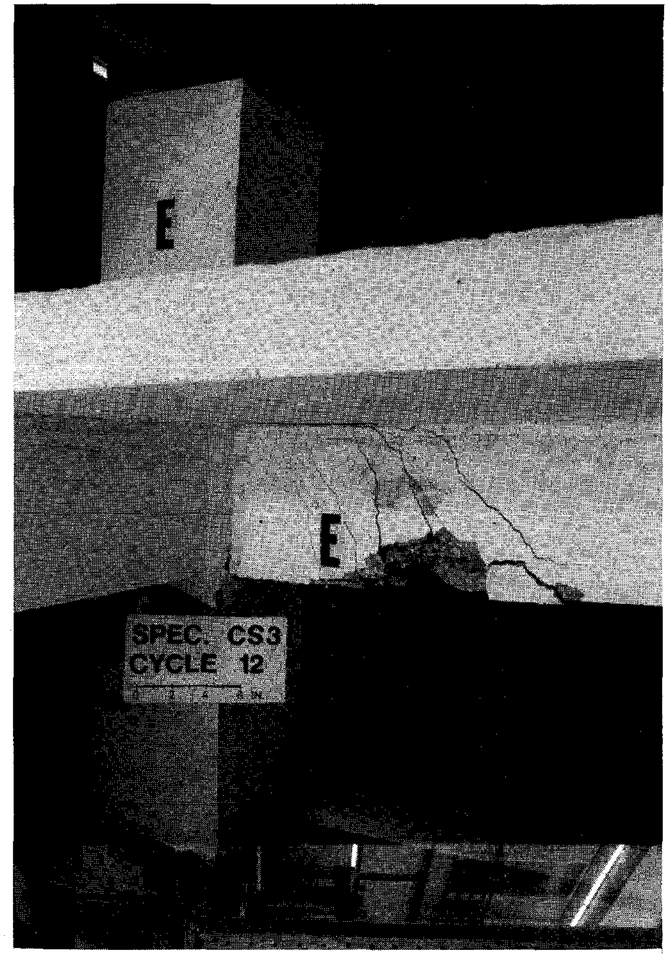


FIGURE 4-6 Stiffness Parameters for Specimen CS1R

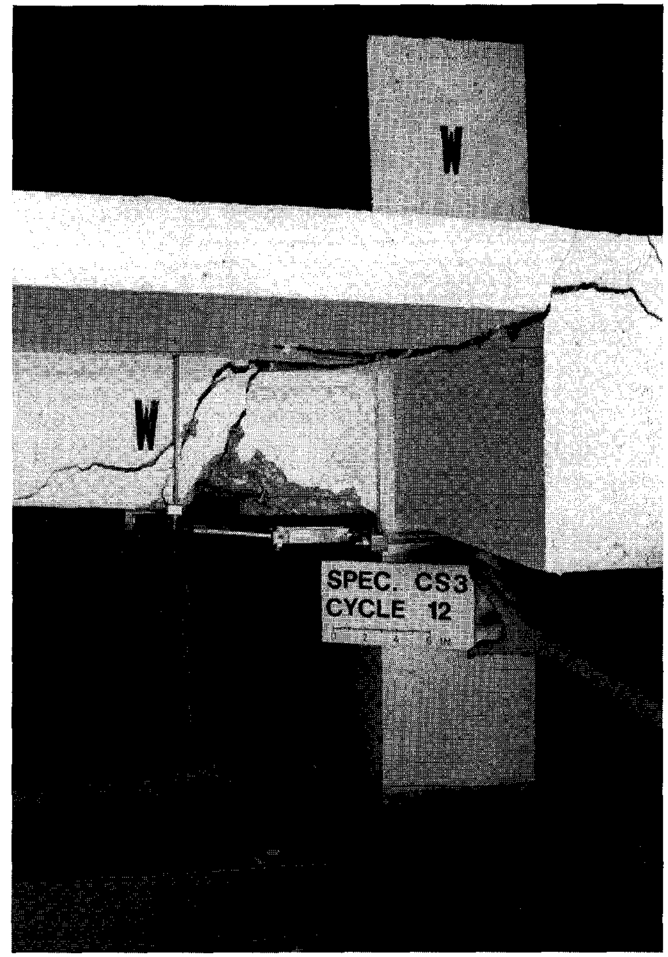
b) Test after repairs



a) Test before repairs

FIGURE 4-7 Damage in East Joint of Specimen CS3R

b) Test after repairs



a) Test before repairs

FIGURE 4-8 Damage in West Joint of Specimen CS3R

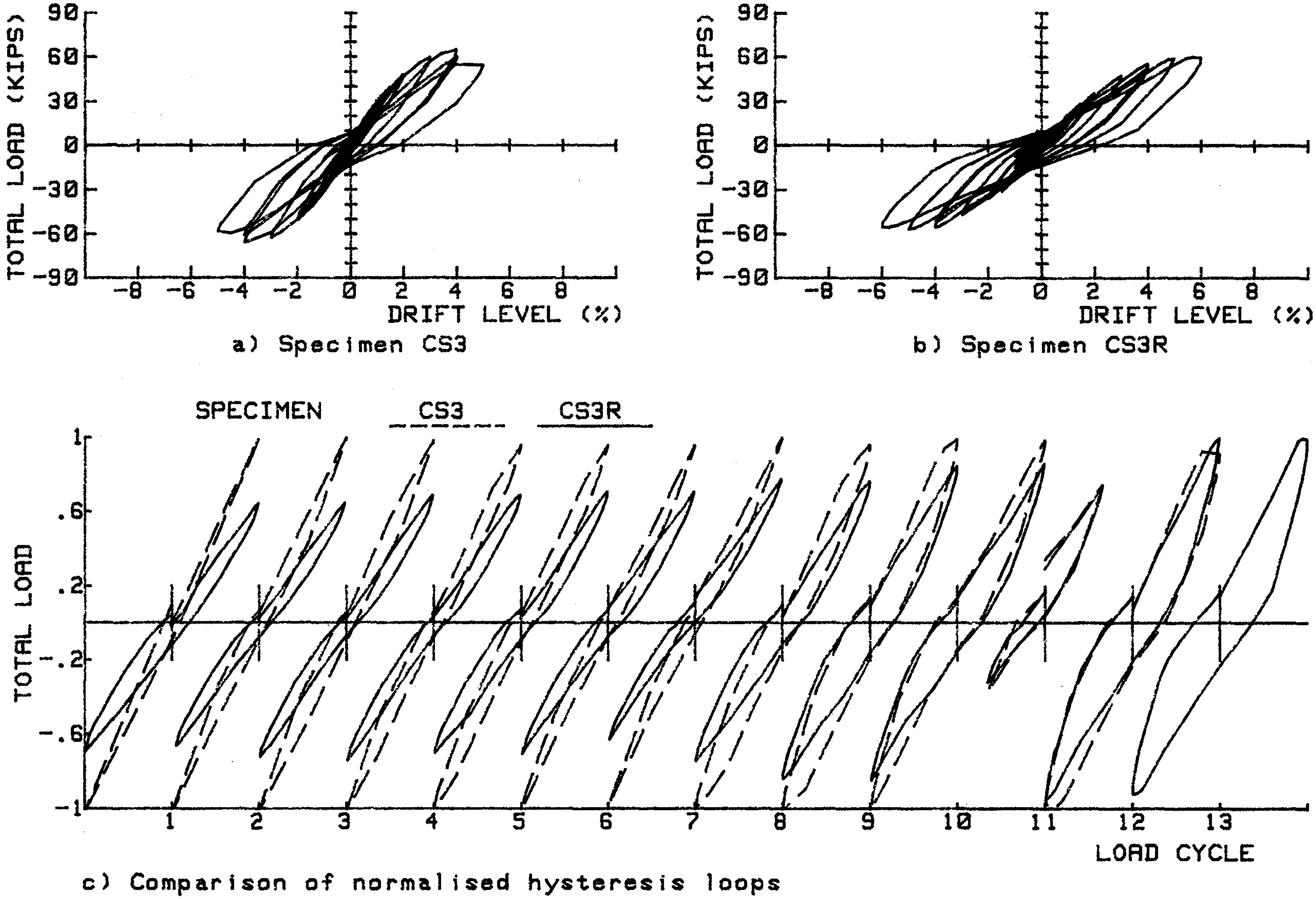


FIGURE 4-9 Load vs. Displacement Plots of Specimen CS3R

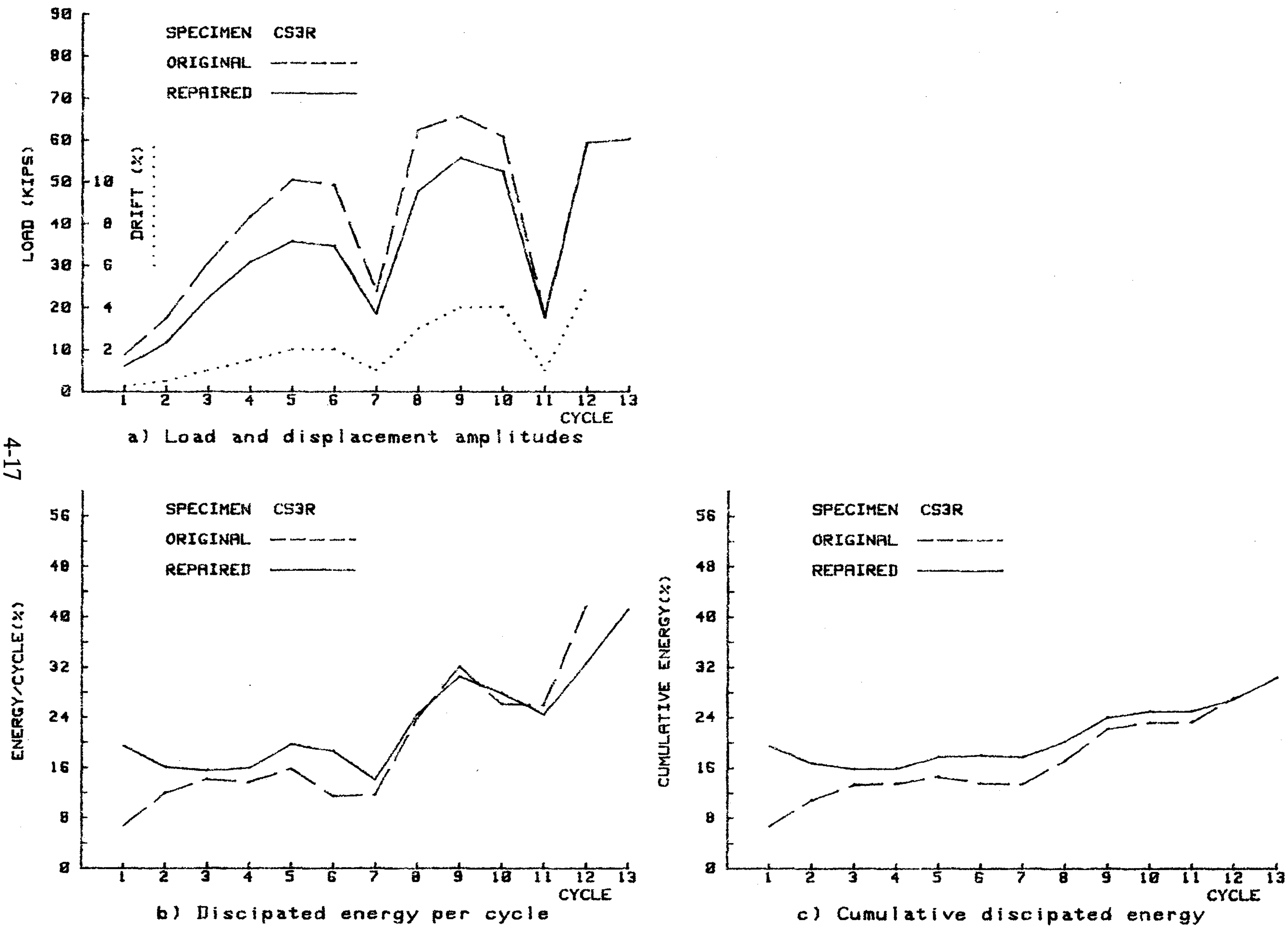
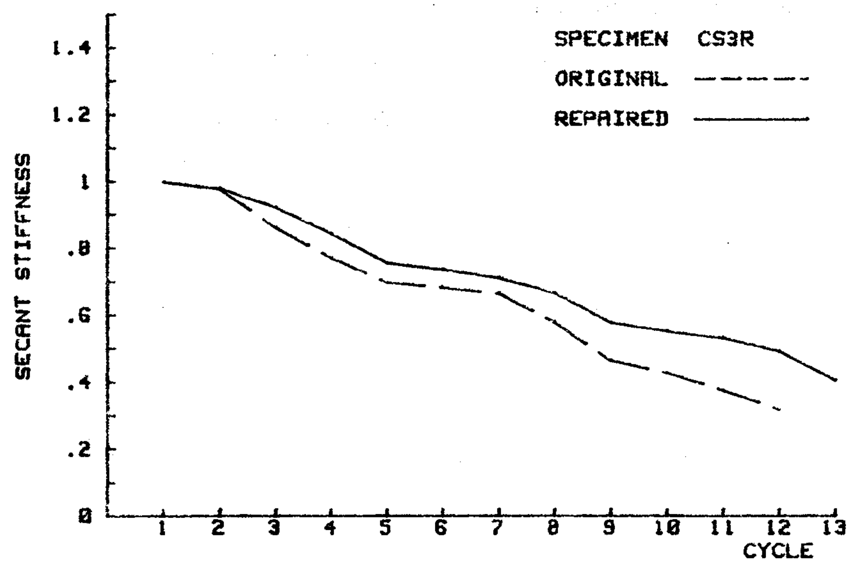
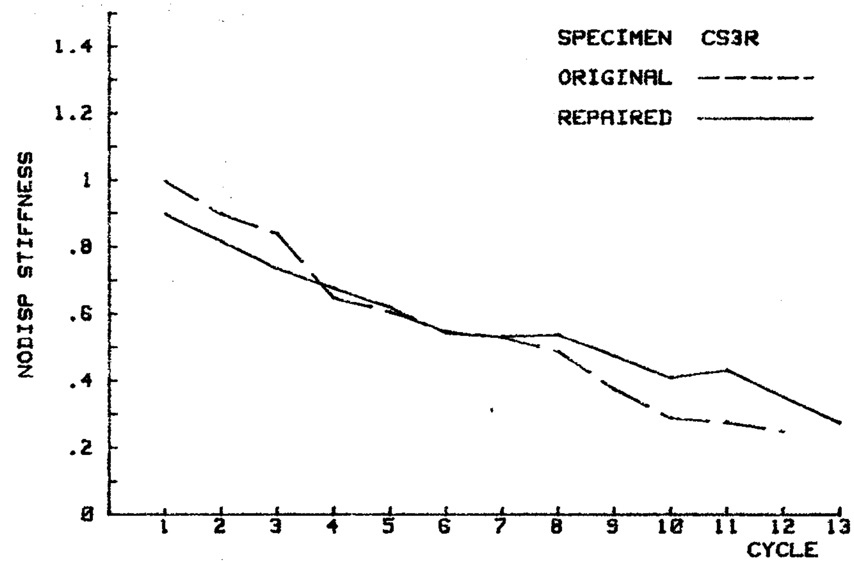


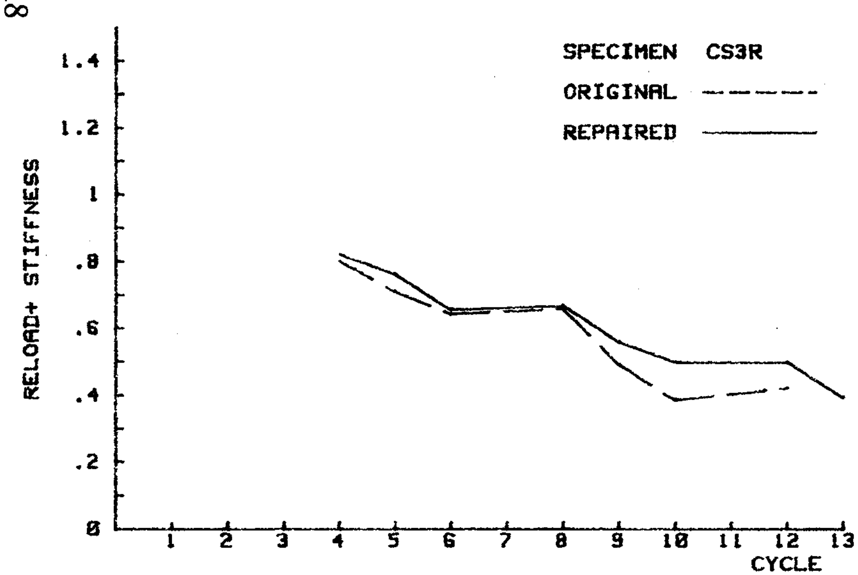
FIGURE 4-10 Strength and Energy Dissipation of Specimen CS3R



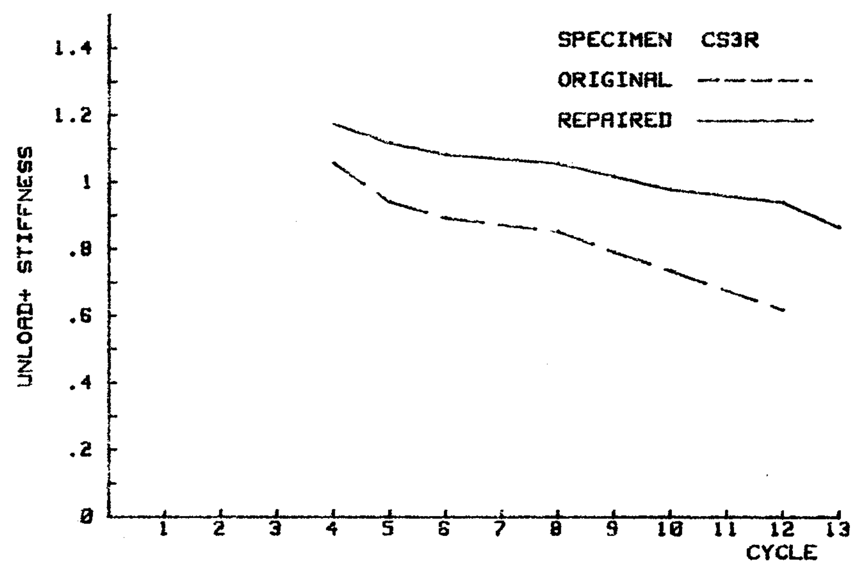
a) Ssecant stiffness



b) Zero-displacement stiffness



c) Reloading stiffness



d) Unloading stiffness

FIGURE 4-11 Stiffness Parameters for Speciment CS3R

4.2.2 Replacement of Damaged Concrete (Specimens CR, CTBR)

4.2.2.1 Repair Procedure

Replacement of damaged concrete requires shoring of the structure, removal of the damaged concrete, and replacement with new concrete. During this investigation, the first of these tasks was not a problem. The removal of old concrete was rather easy where heavy damage was evident, the less damaged concrete, however, required the use of a percussion drill. Building forms and casting new concrete will normally require some ingenuity but even if some voids are created during the casting process they can easily be remedied without major consequences from the structural stand point.

4.2.2.2 Overall Performance

The cracking patterns for specimen CTBR before and after repairs are shown in figure 4-15. The cracks before repairs are generally similar to those after repairs both in location and size.

4.2.2.3 Load vs. Displacement Loops

The load vs. displacement loops for these two specimens are shown in figures 4-12 and 4-16, respectively. The shape of the loops of specimen CR before and after repairs are quite similar in each cycle; the main difference is that the hysteresis loops are narrower during early cycles in the test after repairs. This would indicate a reduced rate of damage due to stronger concrete used for the repairs. In the case of specimen CTBR, however, significant pinching of the loops is observed in the test after repairs even during the first few cycles. This is not obvious during the first two cycles because of fewer data points.

The peak loads obtained during the test after repairs of specimen CR, shown in figure 4-13a are consistently at least equal to those obtained before the repairs. The peak load in cycle 9 was ten percent larger for the repaired specimen. On the other hand, in the case of specimen CTBR, the peak load shown in figure 4-17a obtained during the test after repairs was initially only 70 percent of that before the repairs in the early cycles. At larger displacements the peak load reached a value which was five percent greater than the load before repairs. The pinched shape of the hysteresis loops and the reduced load amplitudes in specimen CTBR indicate that the repair had not effectively restored the stiffness although the repaired specimen was at least as strong as the original specimen. Both specimens reached their maximum load capacity during the ninth cycle in both tests before and after the repairs and both lost strength at about the same rate thereafter.

4.2.2.4 Dissipated Energy

The cyclic and cumulative energy dissipation by specimens CR and CTBR are shown in figures 4-13 and 4-17, respectively. Specimen CR dissipated less energy after repairs due to the reduced level of damage as demonstrated by the narrow shape of the hysteresis loops. On the other hand, specimen CTBR dissipated more energy during early cycles of the test after repairs as a result of friction in the damaged connections and less energy in the later cycles due to loss of bond of reinforcing bars passing through the joints.

4.2.2.5 Stiffness Parameters

The stiffness parameters corresponding to each load cycle for these two specimens are shown in figures 4-14 and 4-18. The comparison of stiffness parameters shows that specimen CR had at least as much stiffness after the repairs as it did when it was undamaged. In the case of specimen CTBR, however, the secant and zero-displacement stiffness plots indicate considerable reduction of stiffness particularly in the early cycles. This loss of stiffness does not appear in the reloading and unloading stiffness plots since the effect of pinching is eliminated in these parameters.

The stiffness parameters complement the trend observed from other data and show that the repair was quite effective in restoring the strength as well as stiffness of specimen CR. In the case of specimen CTBR, the repair was effective mostly in restoring the strength of the specimen. The stiffness and energy dissipation, however, were not affected by the repairs. This deficiency can be explained by recalling that the repair of this specimen consisted of removing the damaged concrete in the beams, injecting epoxy in all remaining cracks, and casting new concrete. Unlike the specimen CR, the damaged concrete in joint core could not be replaced due to the presence of the transverse beams. This process thus restored the flexural and shear capacity of the beams but it did not prevent the slippage of beam bars through the joint despite the efforts to restore bond by injecting epoxy.

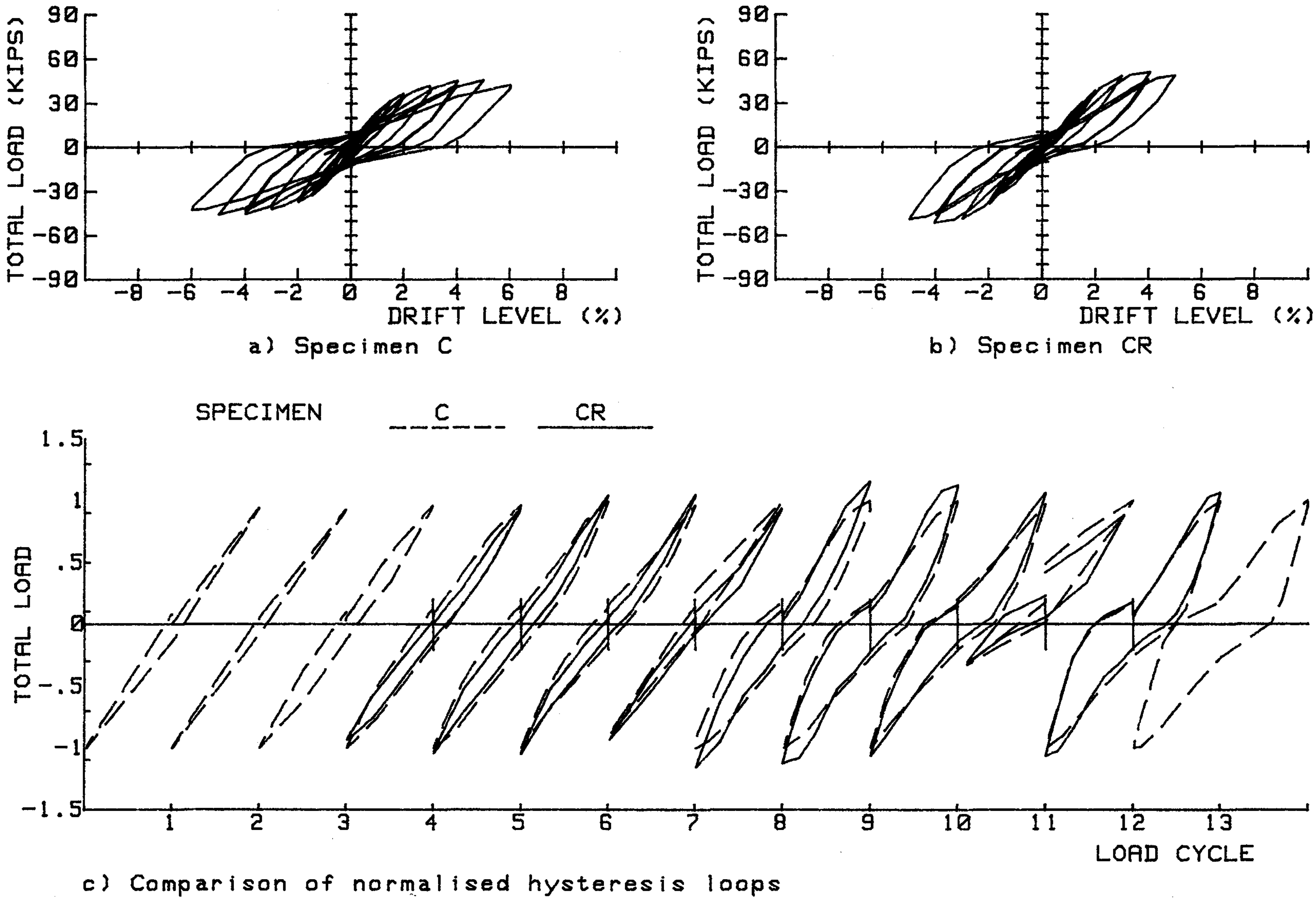
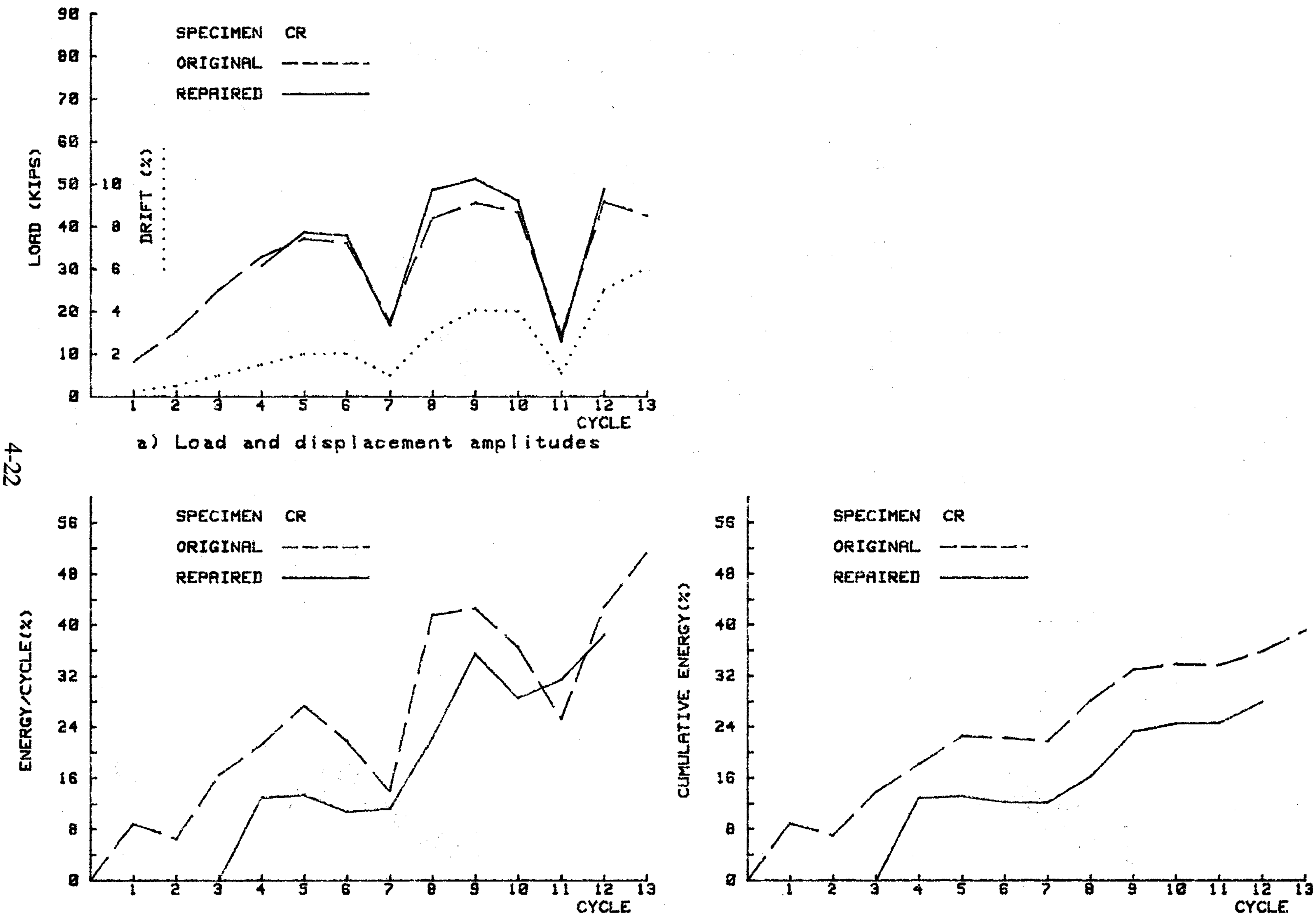


FIGURE 4-12 Load vs. Displacement Curves for Specimen CR



a) Load and displacement amplitudes

b) Discipated energy per cycle

c) Cumulative discipated energy

FIGURE 4-13 Strength and Energy Dissipation of Specimen CR

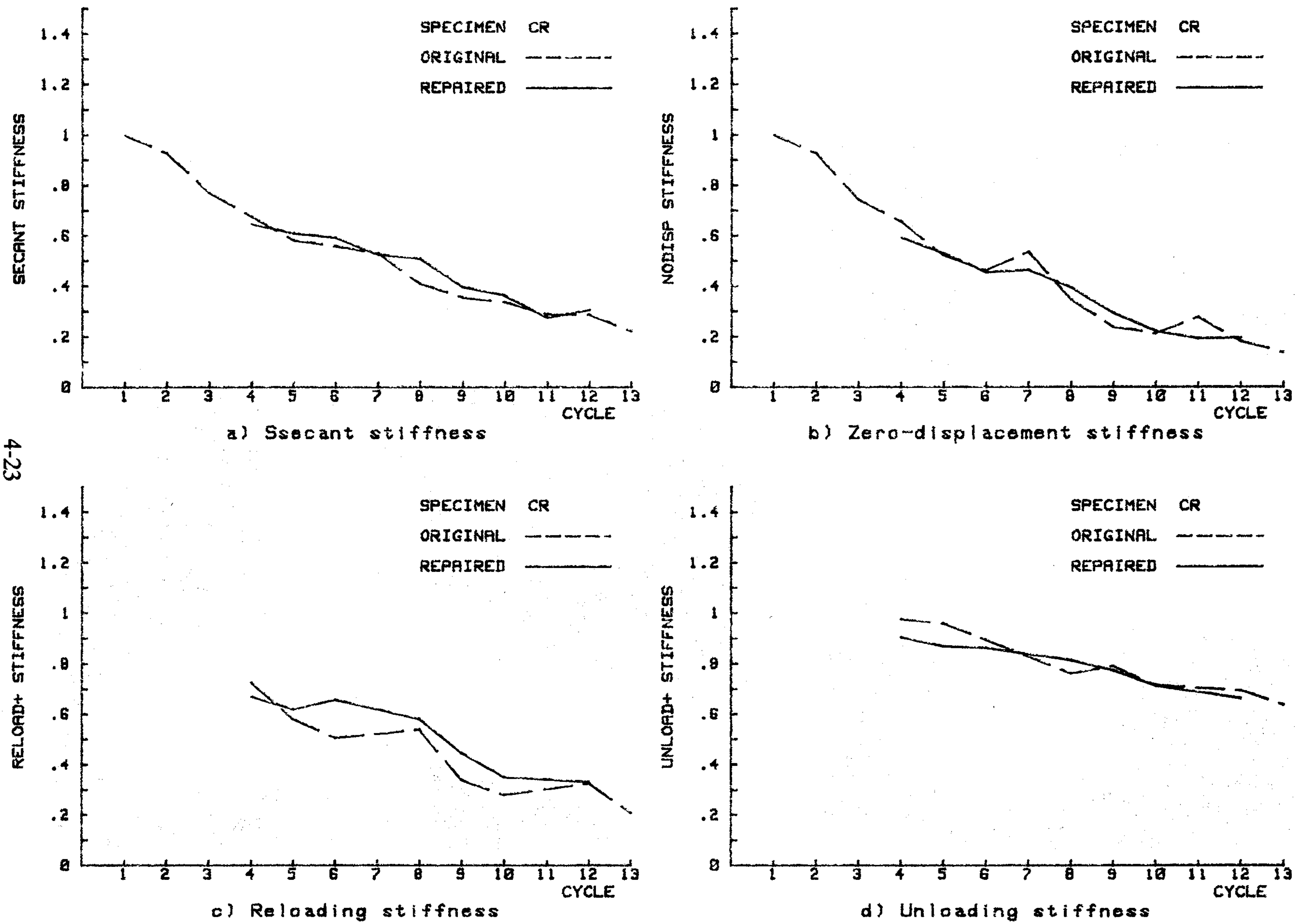
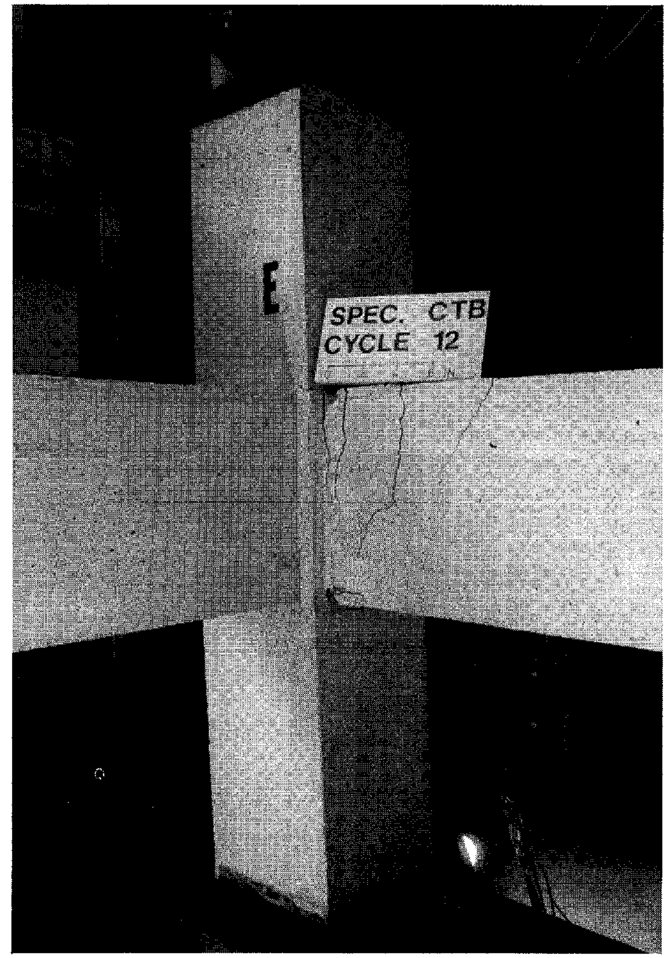


FIGURE 4-14 Stiffness Parameters of Specimen CR

b) Test after repairs



a) Test before repairs

FIGURE 4-15 Cracking Pattern of Specimen CTBR

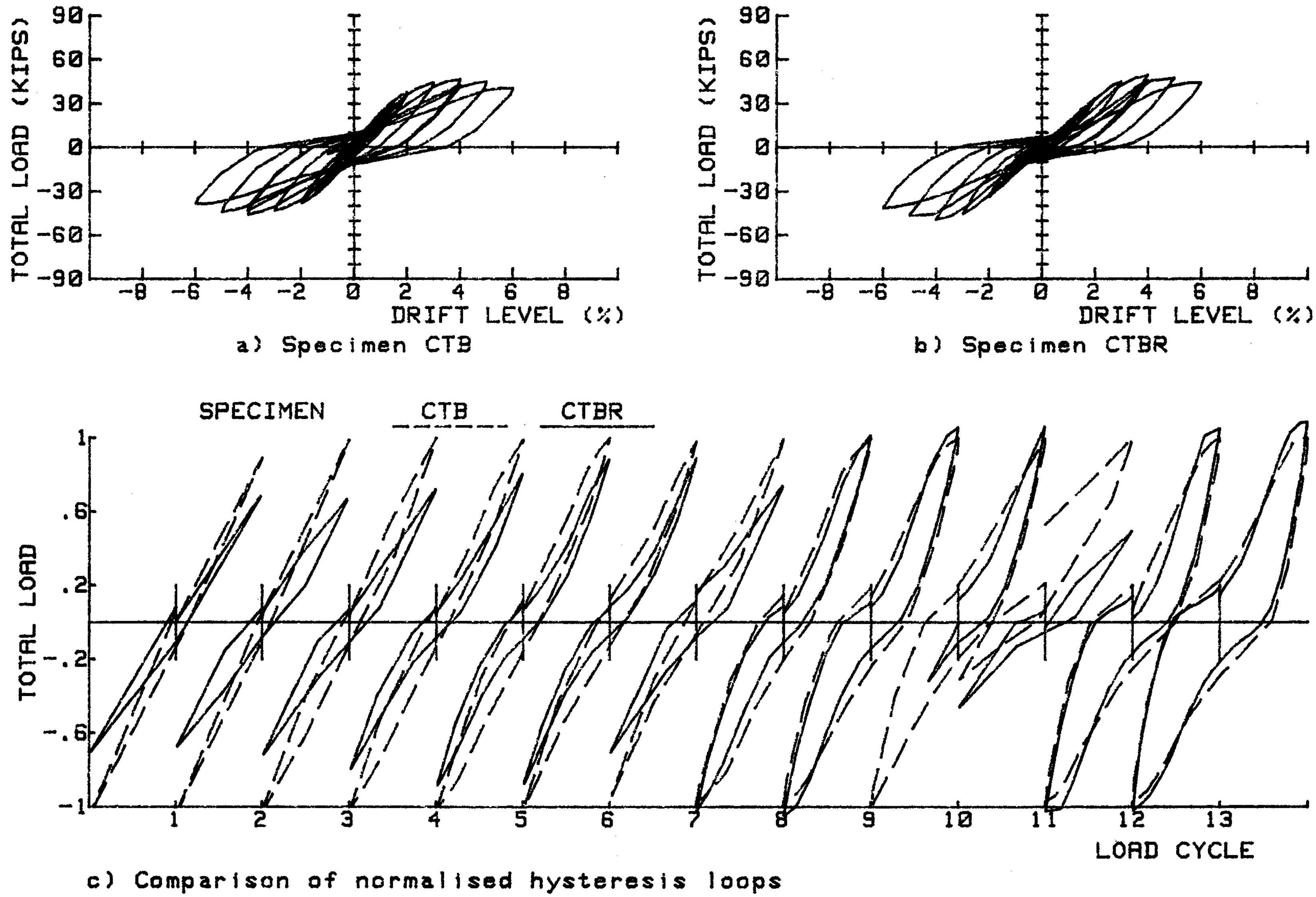


FIGURE 4-16 Load vs. Displacement Curves for Specimen CTBR

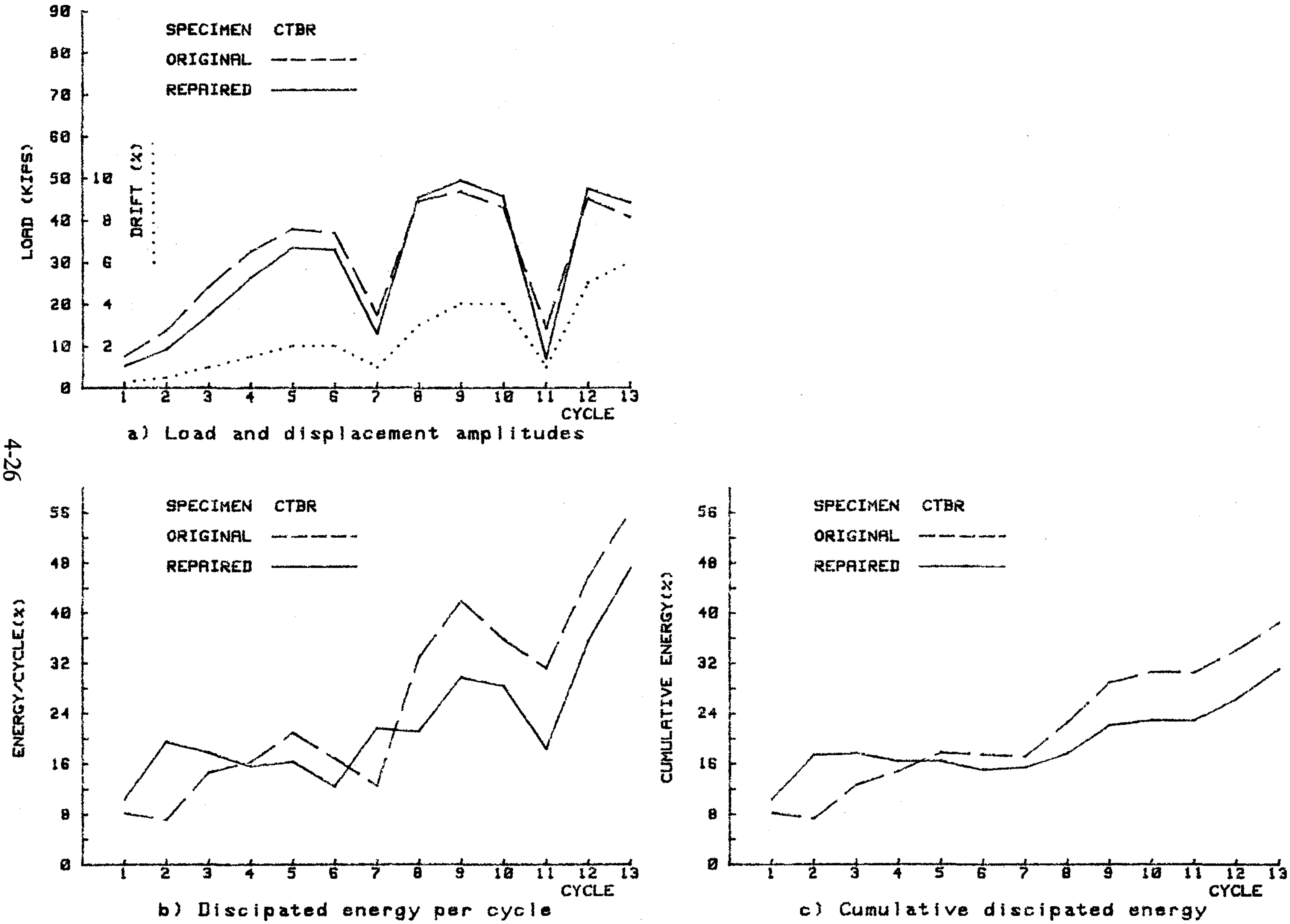


FIGURE 4-17 Strength and Energy Dissipation of Specimen CTBR

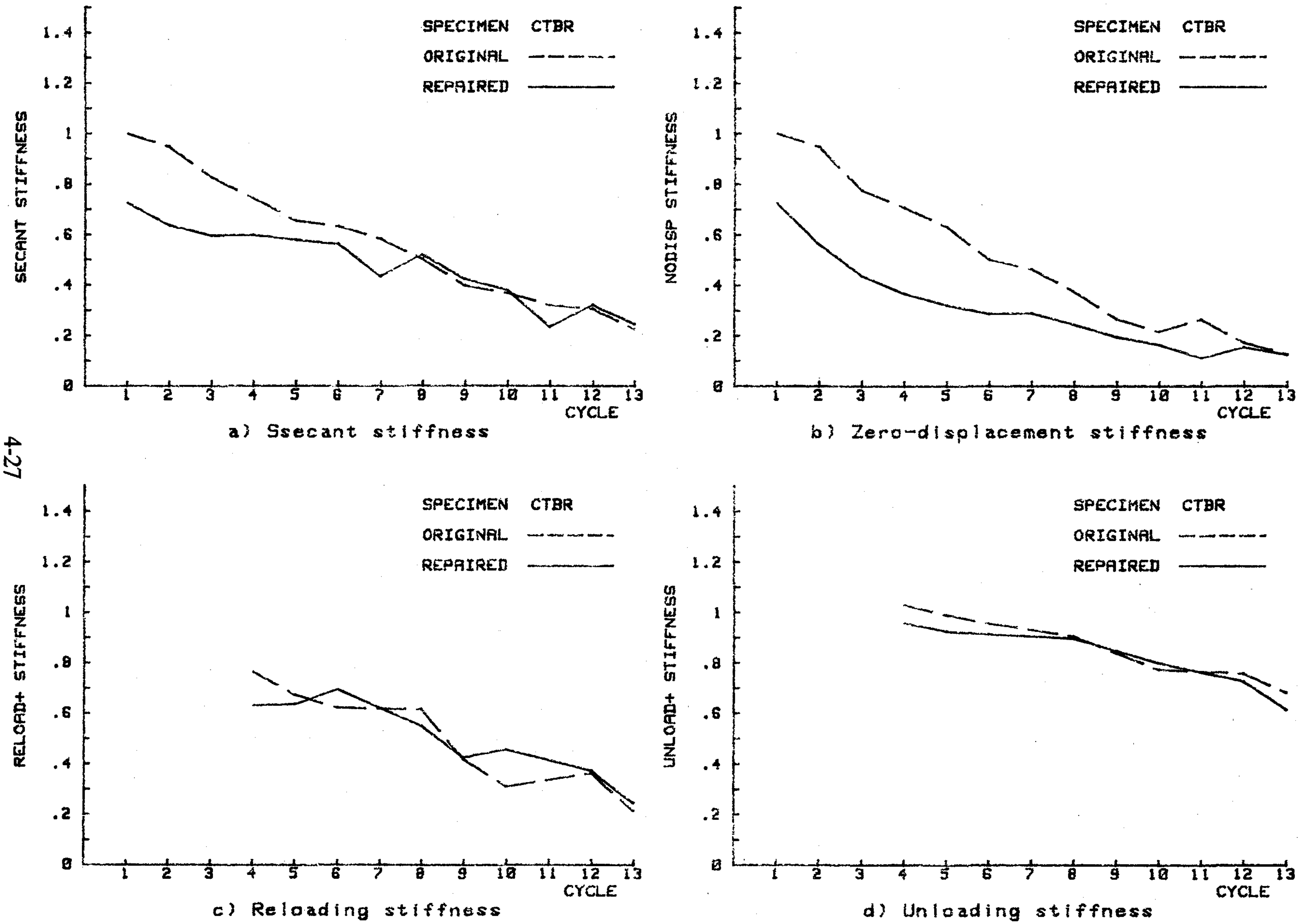


FIGURE 4-18 Stiffness Parameters of Specimen CTBR

4.2.3 Jacketing with Reinforced Concrete (Specimens ER, IR, ES1R, CS2R and CS4R)

4.2.3.1 Repair Procedure

This strengthening procedure presents some challenges that must be carefully addressed during the design phase. The difficulties are mainly related to the placement of transverse reinforcement in the joint region. The presence of beams and slabs requires that joint reinforcement be composed of hoop segments placed in holes drilled through the obstructions. On one hand the drilling process is difficult due to the presence of existing steel in the beams and on the other at least some of the tie segments must be bent in place, which is quite difficult to accomplish in the restricted space between the main and transverse beams under the slab.

For these reasons, dowels with a hook were used instead of attempting to provide continuity in the ties around the joint. An alternate solution would consist of enlarging the joint region with appropriate reinforement as done in specimens ES2R and IS1R. This alternative simplified the task of placing the transverse reinforcement and the casting of concrete around the joint.

4.2.3.2 Overall Performance

The cracking patterns in the vicinity of the joints of these specimens before and after repairs are shown in figures 4-19, 4-23, 4-26, 4-30 and 4-34. The columns in general exhibited only minimal signs of damage during the tests after repairs. Small flexural crsckd were, however, observed in the corners of the columns in the joint region. The length and width of these cracks remained approximately constant at each peak displacement during the test.

The beam cracking patterns during the tests after repairs were similar in location as well as in size to the corresponding patterns before repairs for specimens ER, CS2R, and CS4R. Since the columns in these three specimens were reinforced with a concrete jacket but the beams were injected with epoxy only, this similarity in behavior is understandable. In the cases of specimens IR and ES1R, however, most of the damage in the beams during the tests after repairs was confined around the section where the added longitudinal reinforcement was terminated, indicating that the plastic hinges formed at the intended locations. The beam regions near the joint suffered almost no damage in specimen IR with the exception of some compressive spalling that extended from the plastic hinges during the last few load cycles. In the case of specimen ES1R, some cracks developed at the top portion of the beam near the face of the column; this was possibly due to bond slippage inside the joint.

After cycle 8, specimen ES1R developed severe damage at the beam loading end due to the increased strength of beam after repairs. This resulted in loss of anchorage of the longitudinal bars and caused opening of the crack as shown in figure 4-26b. The strength of the specimen during the last few cycles was therefore reduced by this failure.

4.2.3.3 Load vs. Displacement Loops

The load vs. displacement plots for specimen ER are shown in figure 4-20. It is apparent from these plots that the specimen was stronger and stiffer after the repairs. The loops are less pinched during the test after repairs indicating a slower rate of damage. This favorable behavior is attributed to the fact that during the test before repairs the joint did not exhibit signs of damage, thus bar slippage through the joint was minimal. Also, the epoxy injection was probably effective in repairing the damage in the beam.

Specimen IR was prematurely damaged by an unforeseen movement of the actuator in the positive loading direction which resulted in the cracks shown in figure 4-23(a). The test was, hoever, continued and the resulting load vs. displacement plots for this specimen are shown in figure 4-24. When displacements are positive, the premature cracks tend to open further and the half loops show that the specimen was stronger and stiffer after repairs. The spiked shape of these half loops also suggests that the rate of damage was smaller after repairs. When the displacements are negative, the cracks tend to close but the displacements are not large enough to completely close the cracks and cause significant reloading of the specimen. The load capacity in each cycle, shown in figure 4-25a, remains basically constant starting at cycle 5 indicating again that the rate of damage inflicted by the test was small. Despite the early damage to the specimen resulting from uncontrolled movement of the actuator, the specimen performed quite satisfactorily during the test after repairs.

The load vs. displacement plots for specimen ES1R are shown in figure 4-27. The data shows that the specimen was stronger and stiffer after the repairs. The pinching of hysteresis loops was delayed until after the eighth cycle in the test after repairs compared to pinching of the loops observed during the fourth cycle in the original test. The irregularity in the load vs. displacement plot near the positive peak of the eighth loading cycle is attributed to a loacalised failure at the beam loading end. This failure, which did not occur at this early stage in the test before repairs, was due to the increased strength of the specimen. After this cycle, the hysteresis loops show similar irregularities at positive peaks and the load capacity of the specimen was limited by the strength of the loading connection.

The load vs. displacement plots of specimens CS2R and CS4R are shown in figures 4-31 and 4-35, respectively. These plots show similarities in the behaviors of these two specimens after repairs. In both cases the stiffness after repairs is very much similar to that before repairs during the first few cycles indicating that the increase in stiffness resulting from the larger columns was offset by the reduced stiffness of the beams. The reduction in beam stiffness is attributed to the inadequacy of the epoxy injection process in repairing the damage. This deficiency is further indicated by pinching of the loops during the early cycles. The effect of the increased column size is manifested in an increase in stiffness during the later loading cycles.

The maximum load resisted during each cycle by specimens CS2R and CS4R is shown in figures 4-32 and 4-36, respectively. These plots indicate that both specimens had basically identical behavior during the test after repairs even though their behavior before repairs was quite different. The center columns of specimen CS2R had suffered considerable damage during the test before repairs and thus limiting its load capacity. Specimen CS4R did not experience a similar damage during the first test. The behavior observed during the tests after repairs indicates that the repair was equally effective in both cases, which is confirmed by the absence of substantial cracking in the columns as mentioned earlier. It was also observed that the differences in details of transverse reinforcement in the columns did not influence the behavior of the specimens.

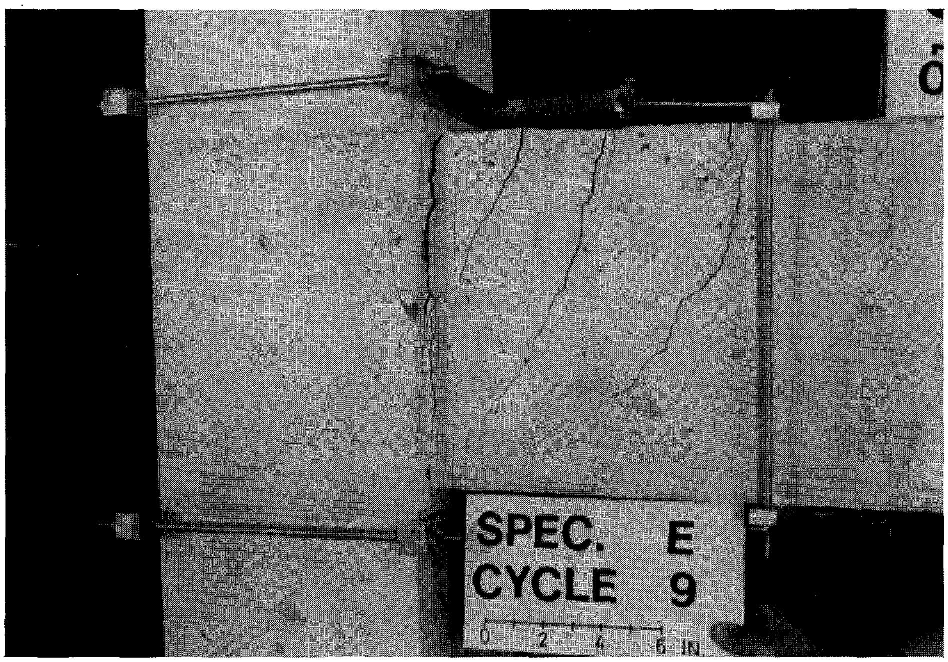
4.2.3.4 Energy Dissipation

The energy dissipation plots for these specimens are shown in figures 4-21, 4-25, 4-28, 4-32 and 4-36. In general, these plots confirm the observations made earlier. Specimen ER dissipated less energy after repairs during the early cycles indicating a slower rate of damage compared to the test before repairs. Specimen ES1R dissipated as much or more energy after repairs than it did before repairs. The lateral load capacity was higher during the second test and the type of damage was such that the loops did not develop as much pinching. Specimens CS2R and CS4R dissipated about the same amount of energy during both tests.

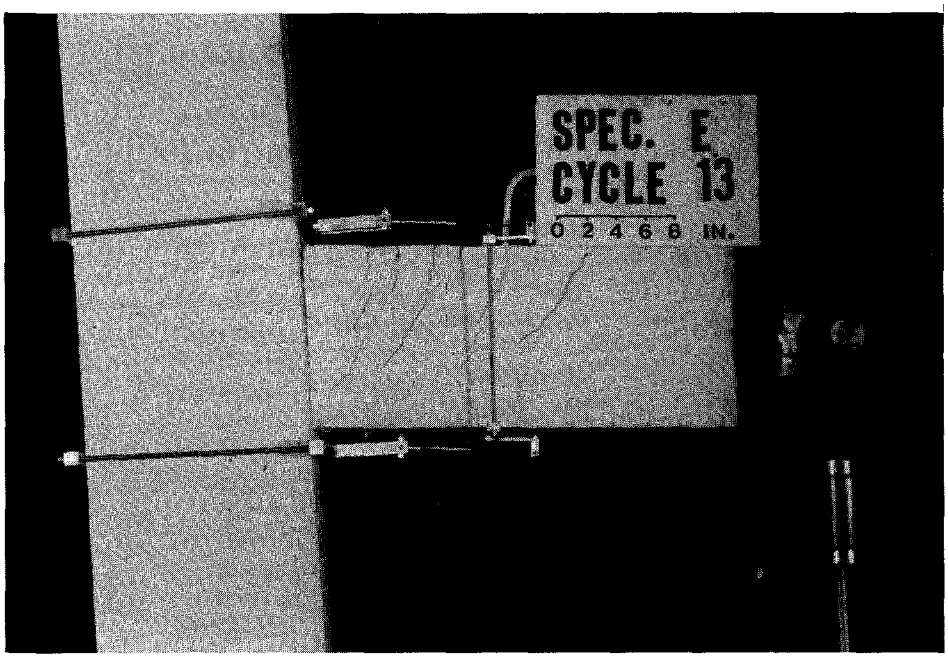
4.2.3.5 Stiffness Parameters

The plots of stiffness parameters for these specimens are shown in figures 4-22, 4-29, 4-33 and 4-37. All the stiffness parameters of specimens ER and ES1R showed increased stiffness during the test after repairs confirming the improved behavior noted earlier.

In the cases of specimens CS2R and CS4R, however, the reasons for the change in stiffness are not so obvious. Recalling that CS2R experienced more damage than CS4R during the test before repairs, the stiffness parameters after repairs of CS2R compare more favorably than those of specimen CS4R. It is important to note that, in spite of the column jacketing, the zero displacement stiffness is actually smaller after repairs in these two specimens indicating increased pinching. The secant stiffness for specimen CS4R was also smaller during the early cycles of the test after repairs and it marginally increased for specimen CS2R. Similar comments apply to the reloading stiffness. Only the unloading stiffness showed a definite increase in both specimens. These observations confirm that the repairs did not improve the behavior of these two specimens in a measure proportionate to the increase in size and strength of the columns. The results indicate that jacketing of columns alone was not adequate in restoring the performance without addressing the problem of load transfer between beams and columns.



a) Test before repairs



b) Test after repairs

FIGURE 4-19 Damage in Specimen ER Before and After Repairs

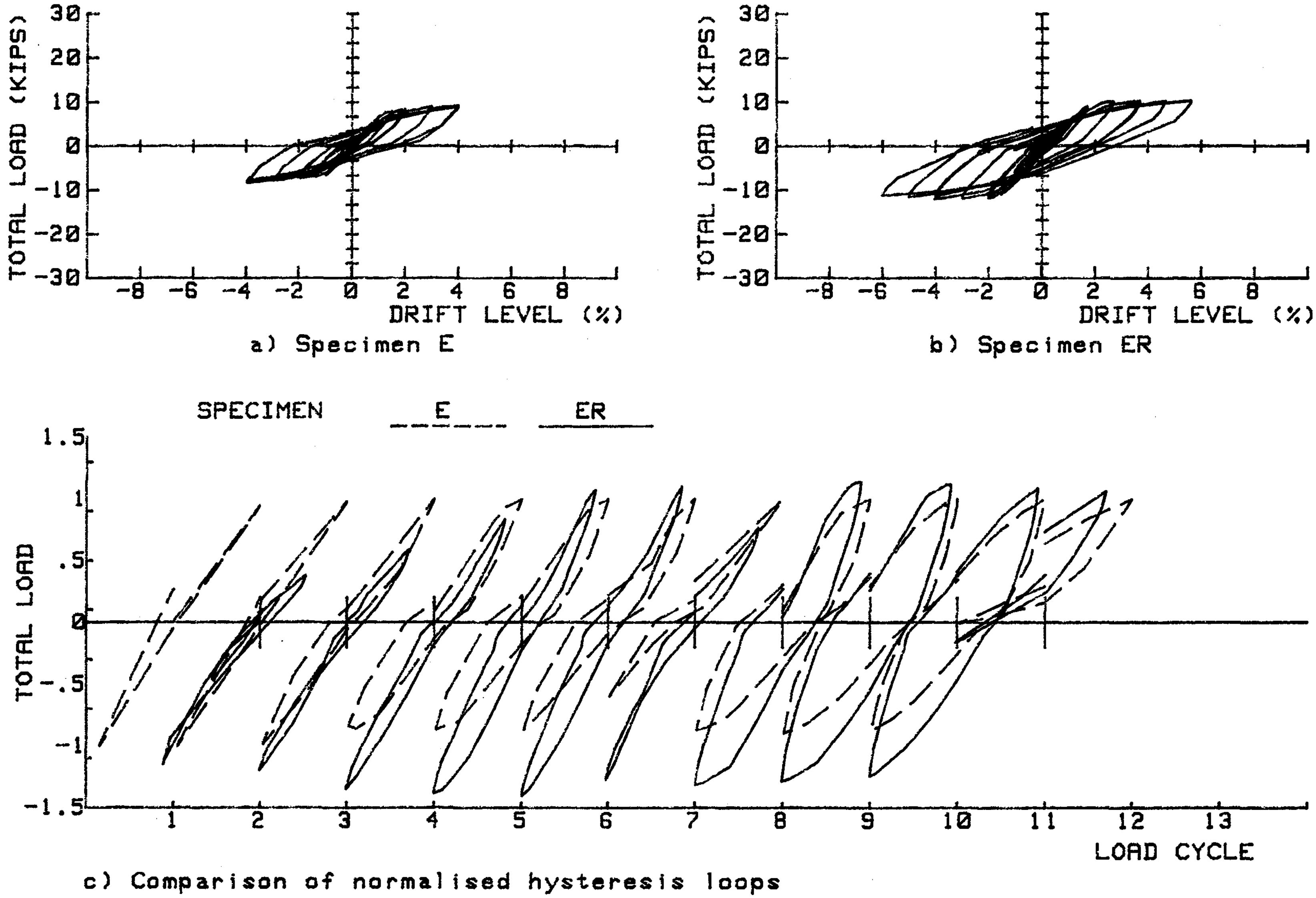


FIGURE 4-20 Load vs. Displacement Plots of Specimen ER

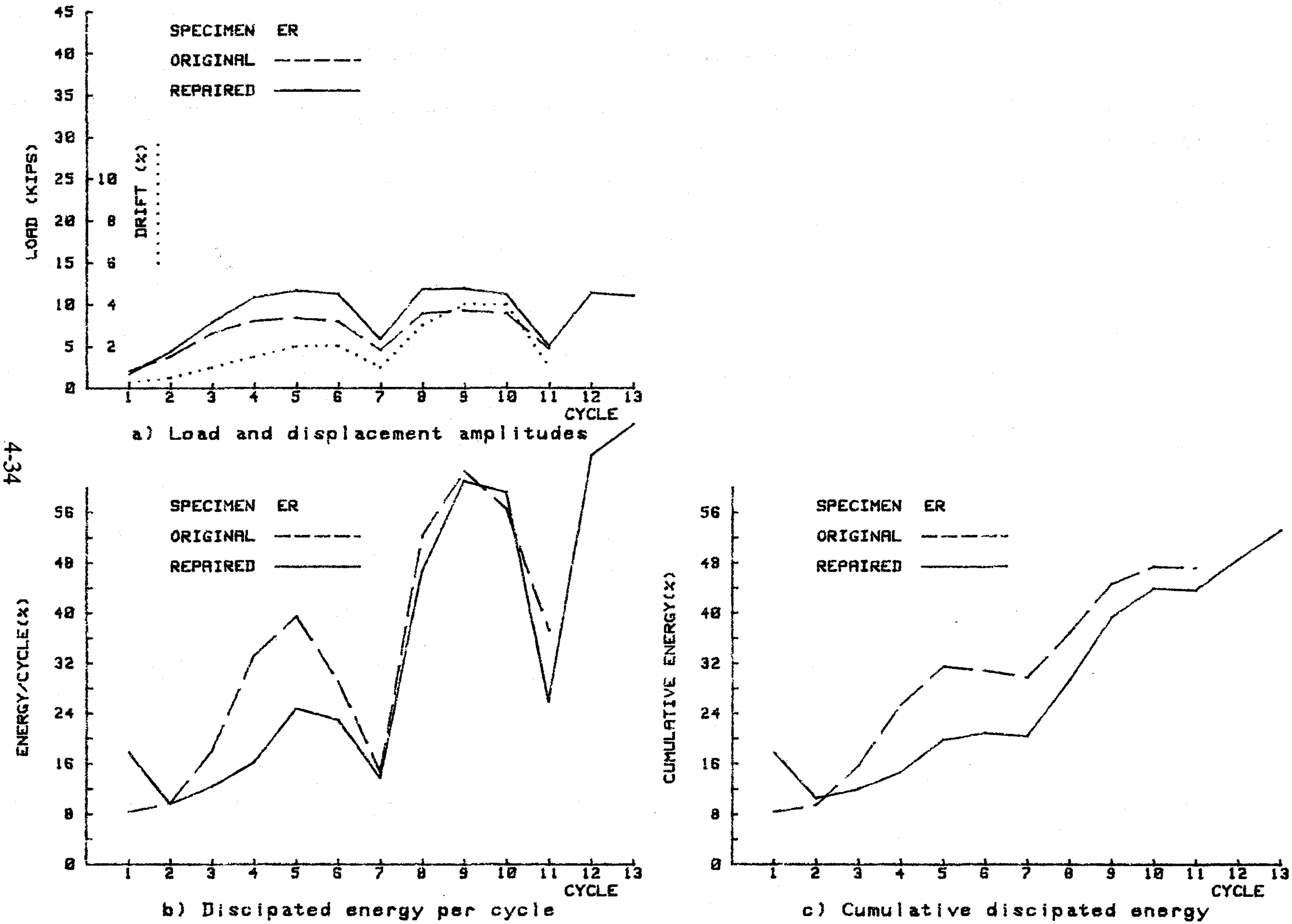


FIGURE 4-21 Strength and Energy Dissipation of Specimen ER

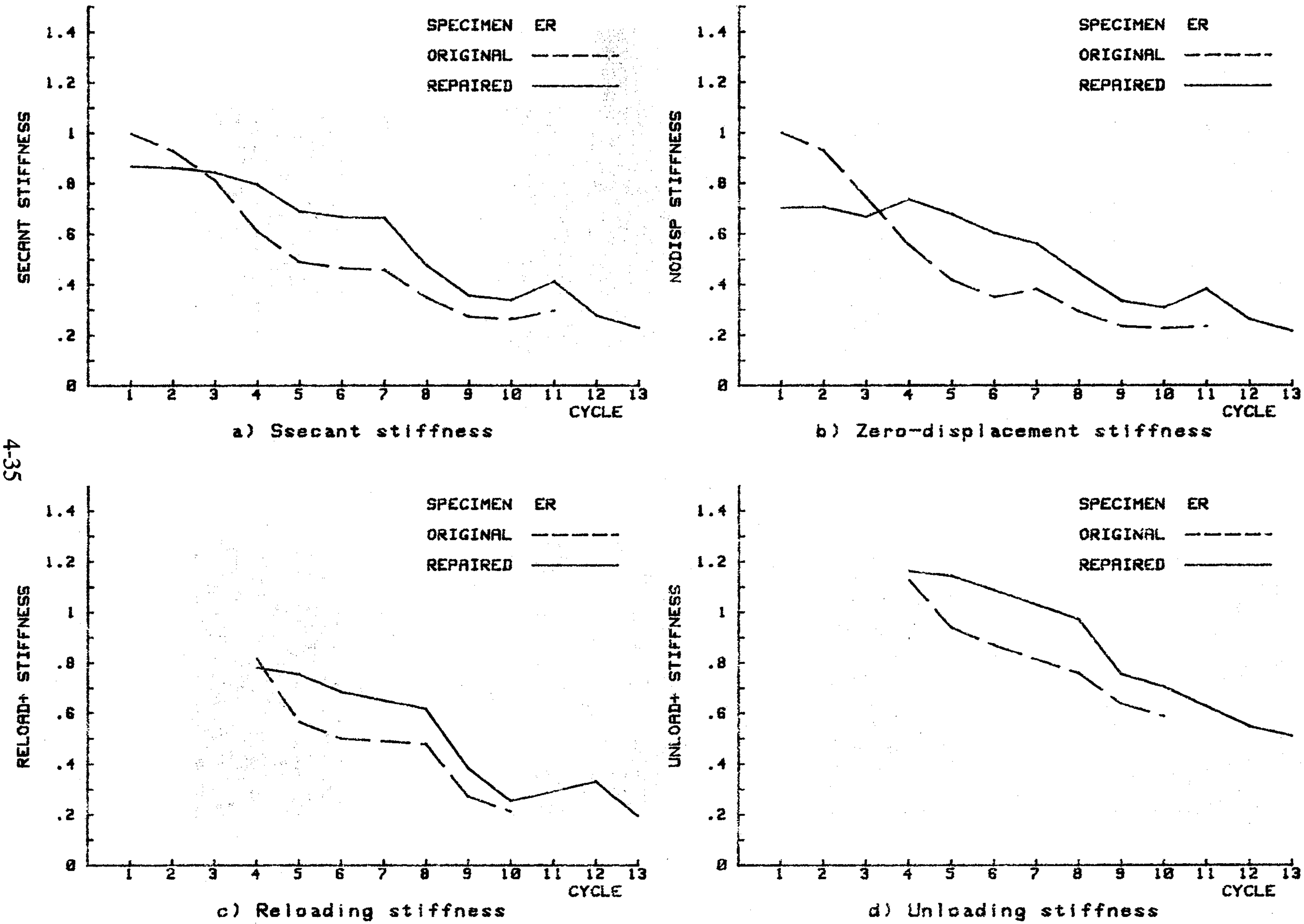
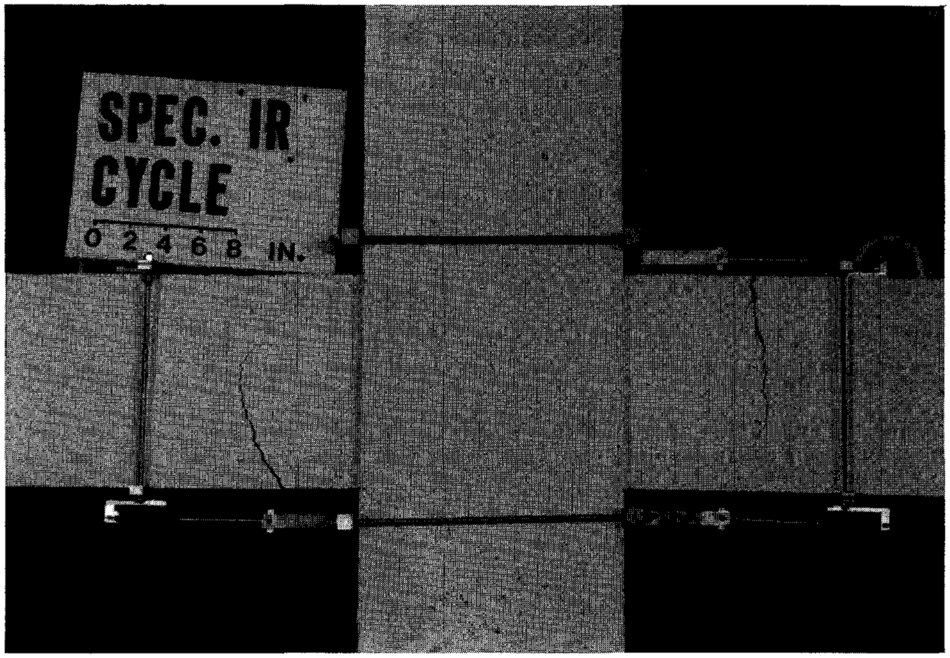
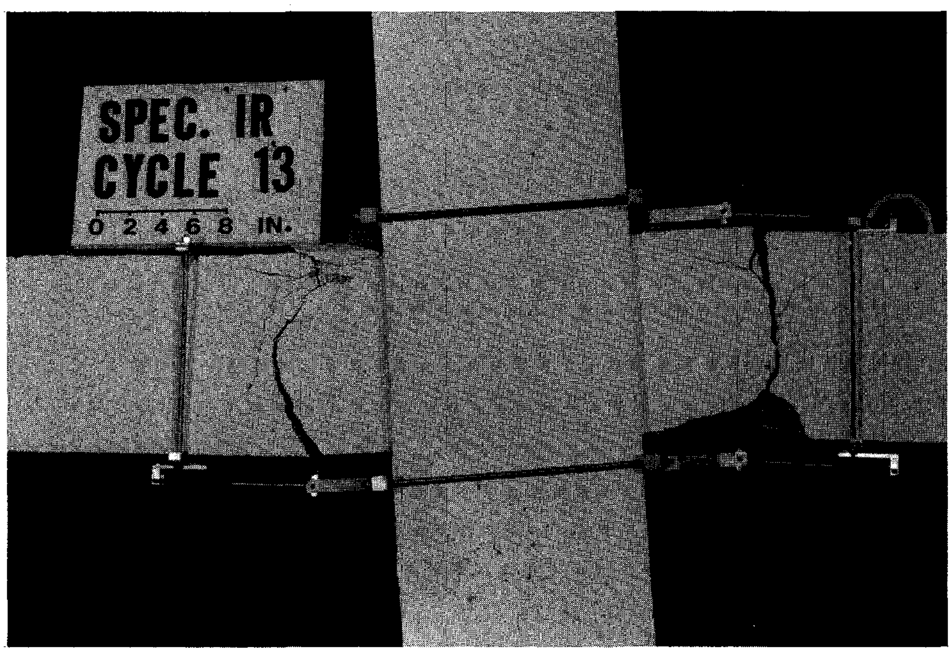


FIGURE 4-22 Stiffness Parameters of Specimen ER



a) Damage at the beginning of the test



b) Damage during load cycle 13

FIGURE 4-23 Damage in Specimen IR Before and After Repairs

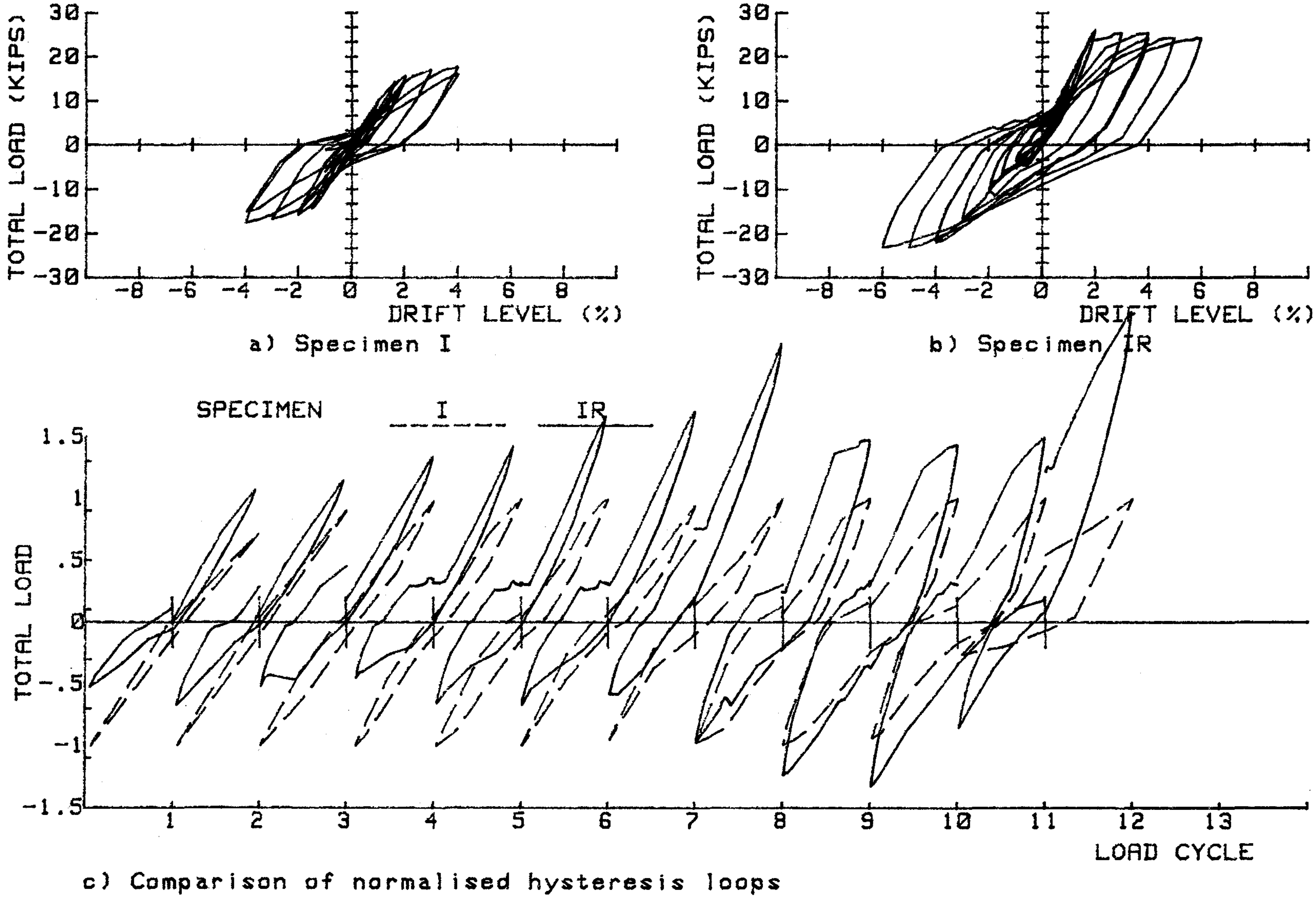
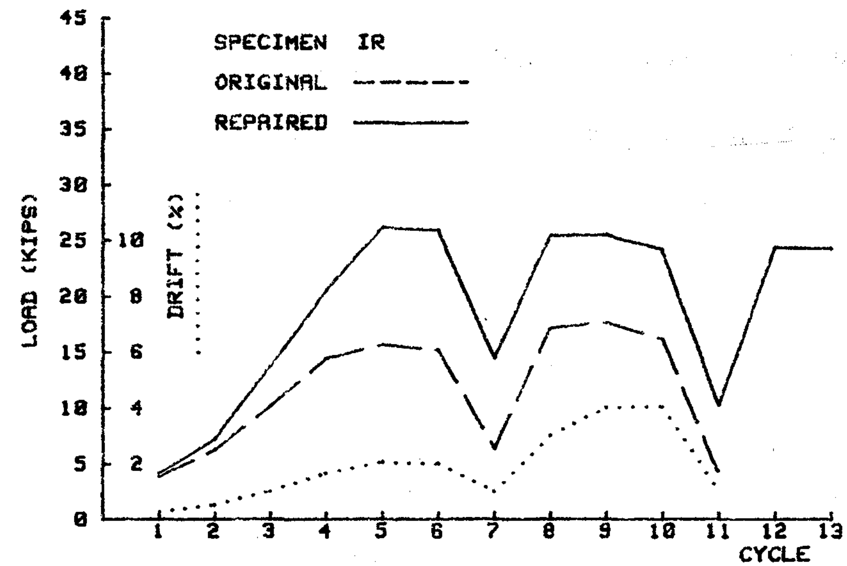
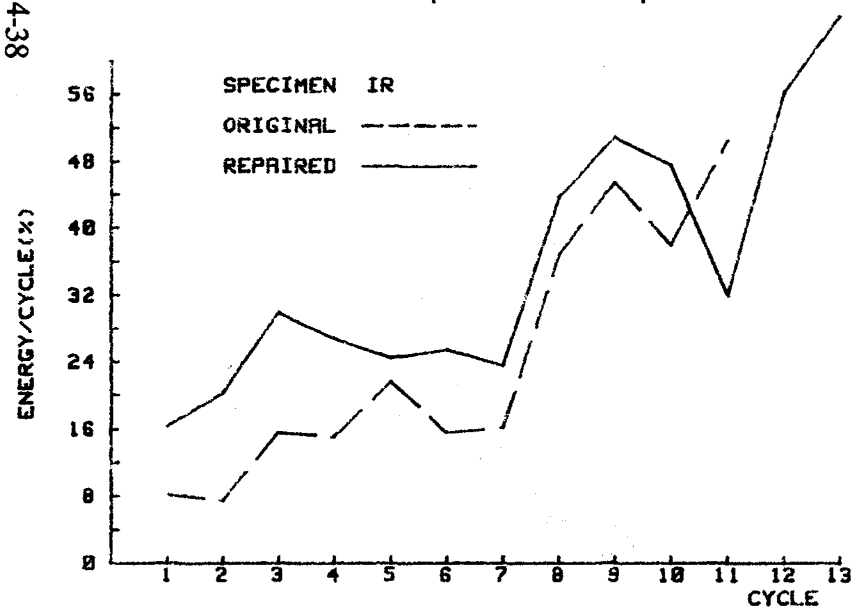


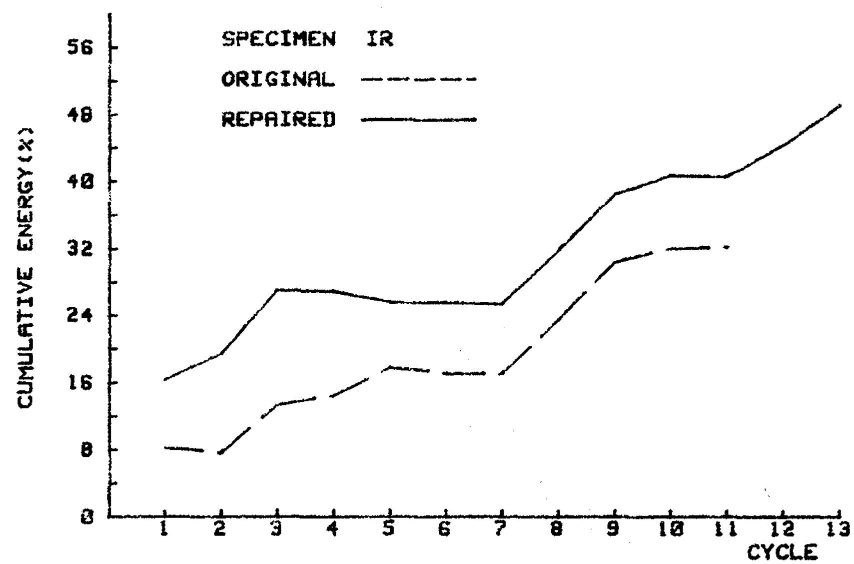
FIGURE 4-24 Load vs. Displacement Plots of Specimen IR



a) Load and displacement amplitudes

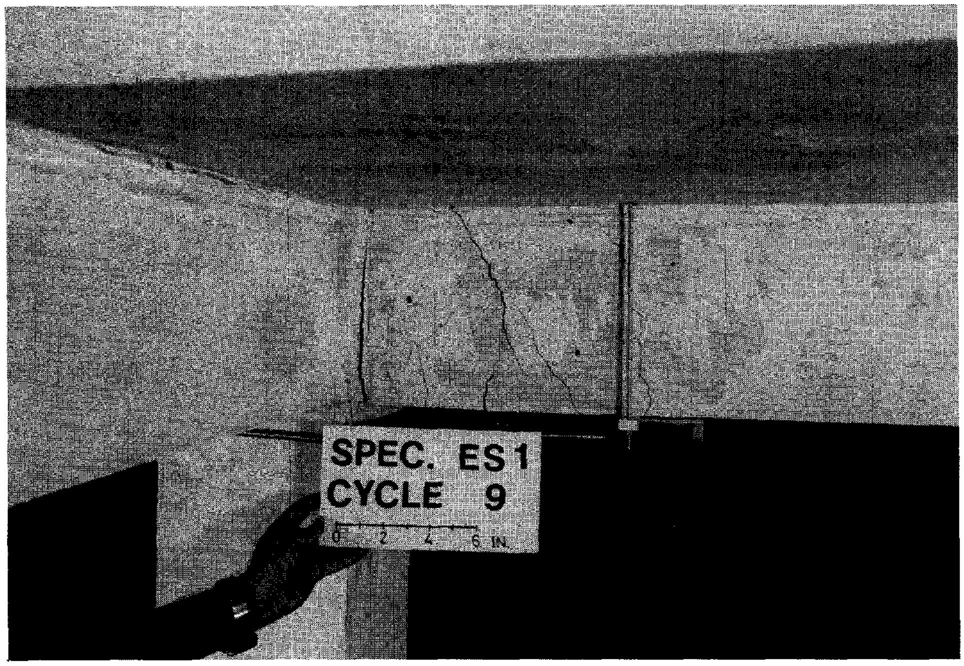


b) Discipated energy per cycle



c) Cumulative discipated energy

FIGURE 4-25 Strength and Energy Dissipation of Specimen IR



a) Test before repairs

b) Test after repairs

FIGURE 4-26 Damage in Specimen ES1R Before and After Repairs

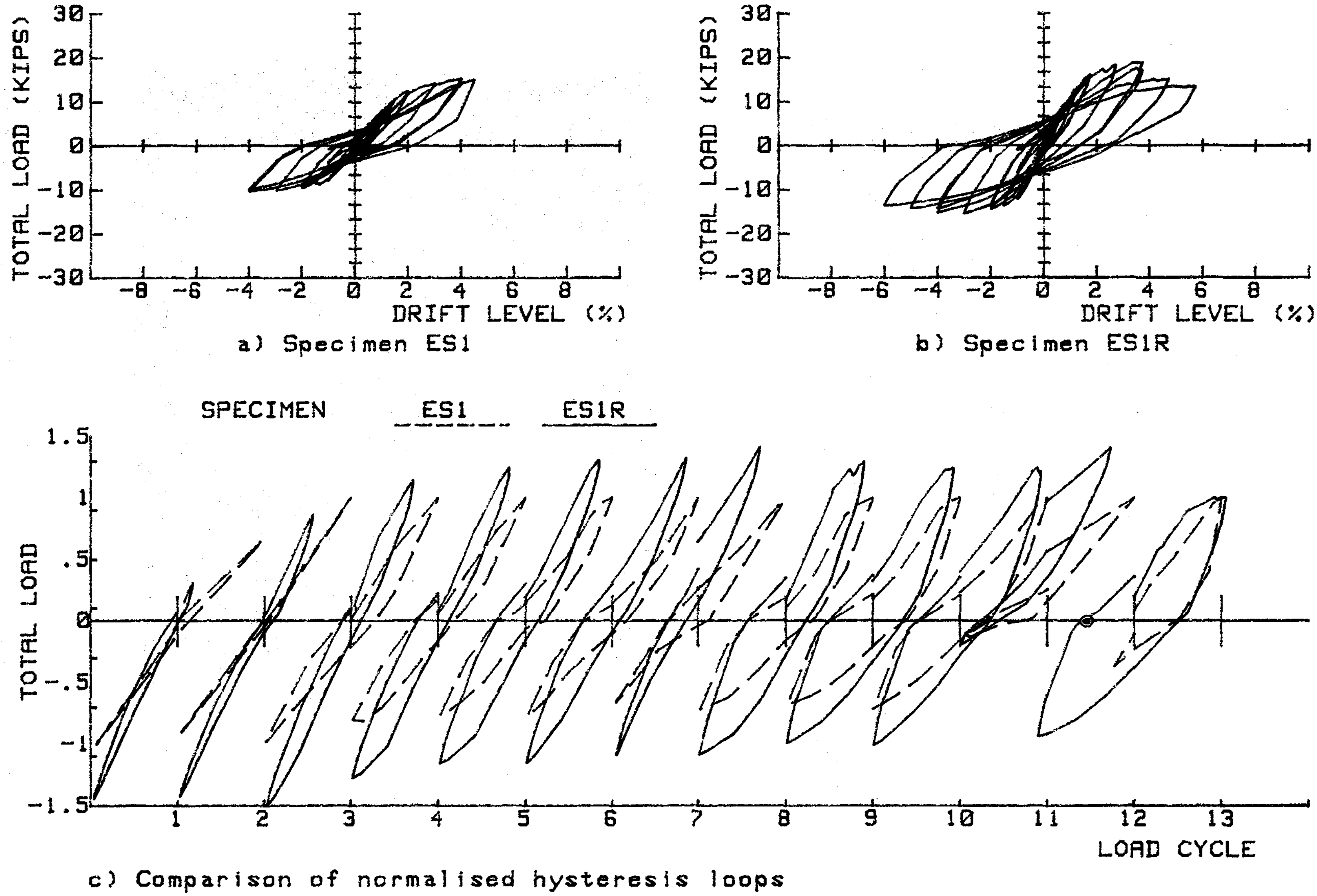
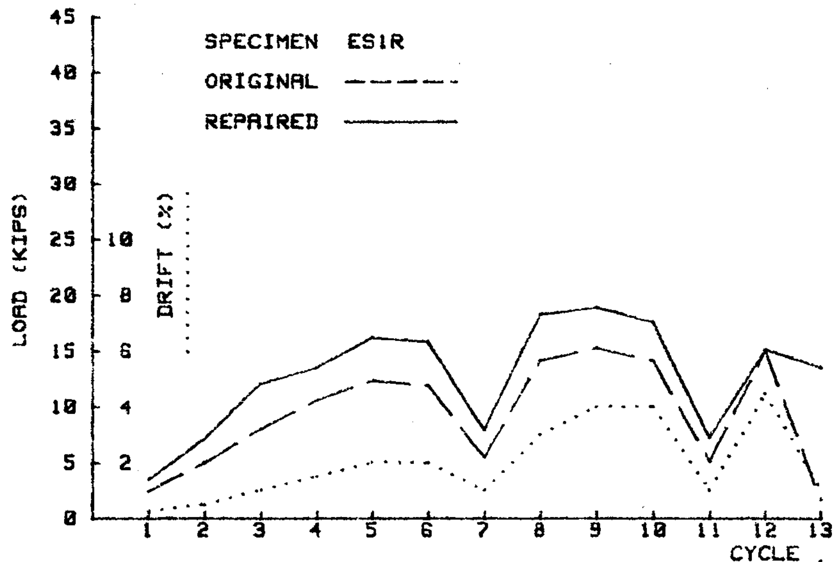
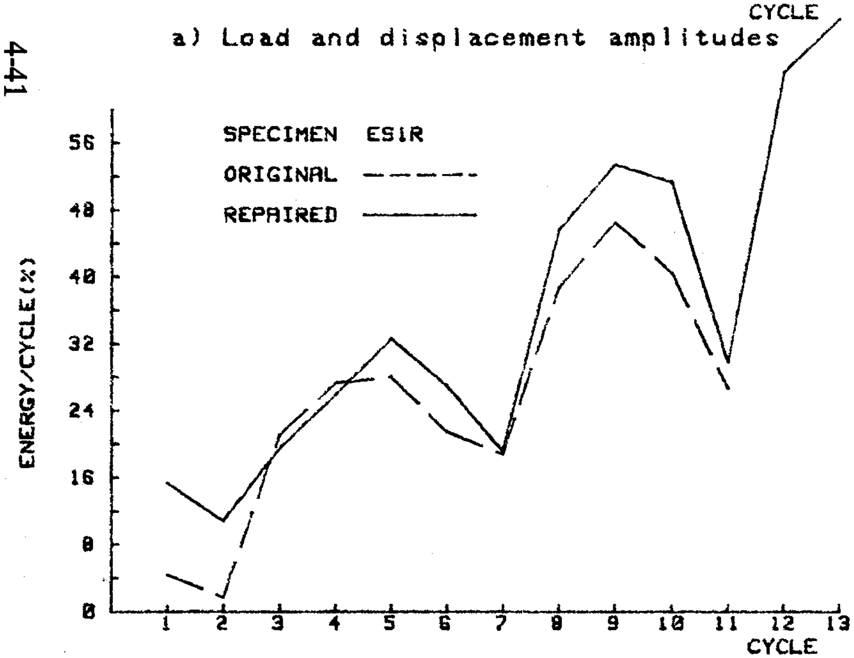


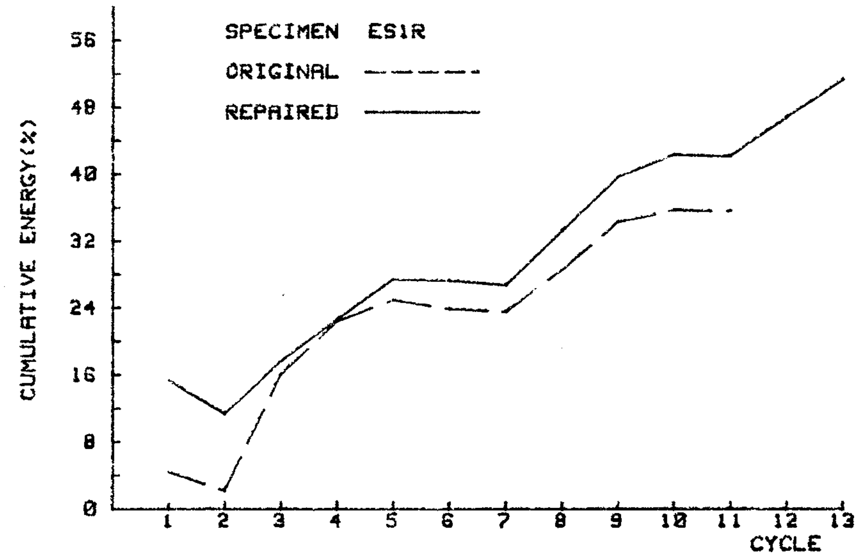
FIGURE 4-27 Load vs. Displacement Plots of Specimen ES1R



a) Load and displacement amplitudes

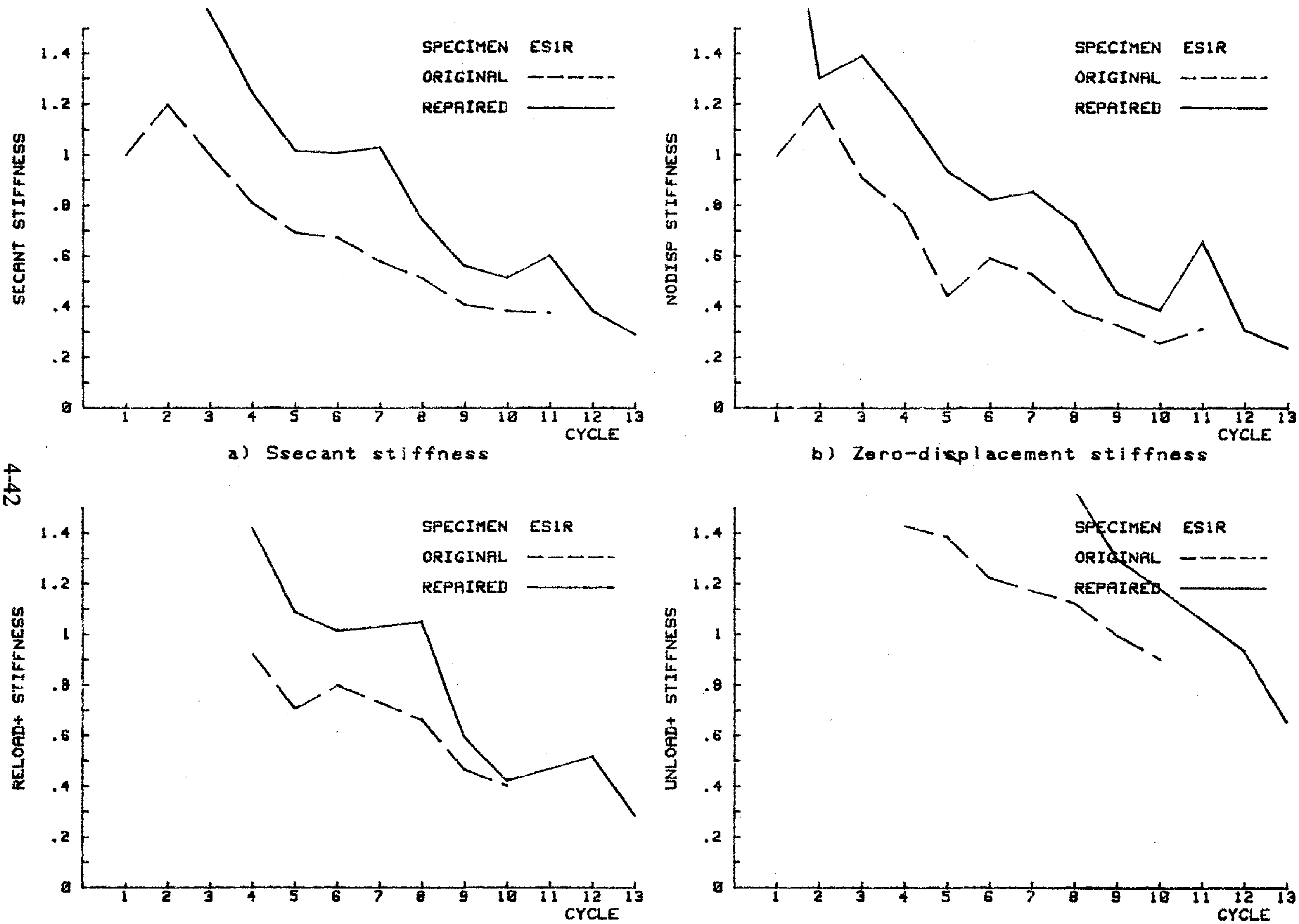


b) Discipated energy per cycle



c) Cumulative discipated energy

FIGURE 4-28 Strength and Energy Dissipation of Specimen ES1R



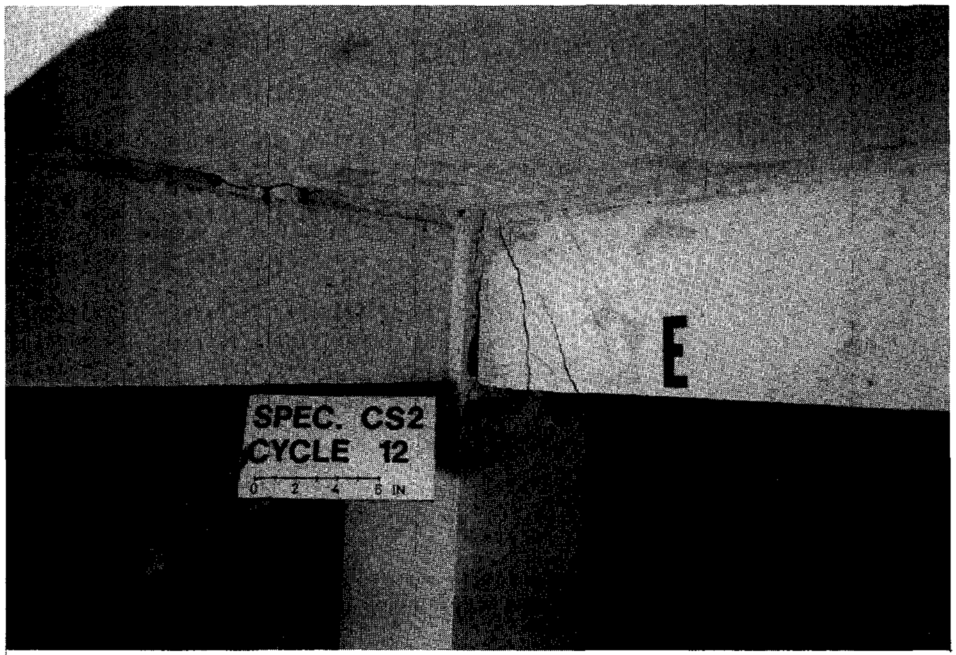
a) Ssecant stiffness

b) Zero-displacement stiffness

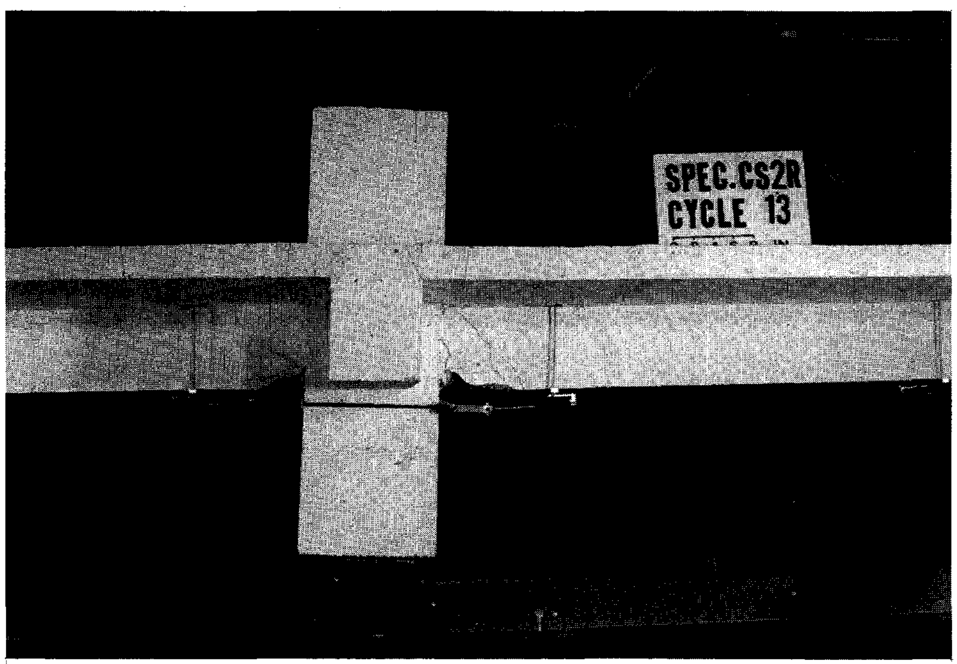
c) Reloading stiffness

d) Unloading stiffness

FIGURE 4-29 Stiffness Parameters of Specimen ES1R



a) Test before repairs



b) Test after repairs

FIGURE 4-30 Damage in Specimen CS2R Before and After Repairs

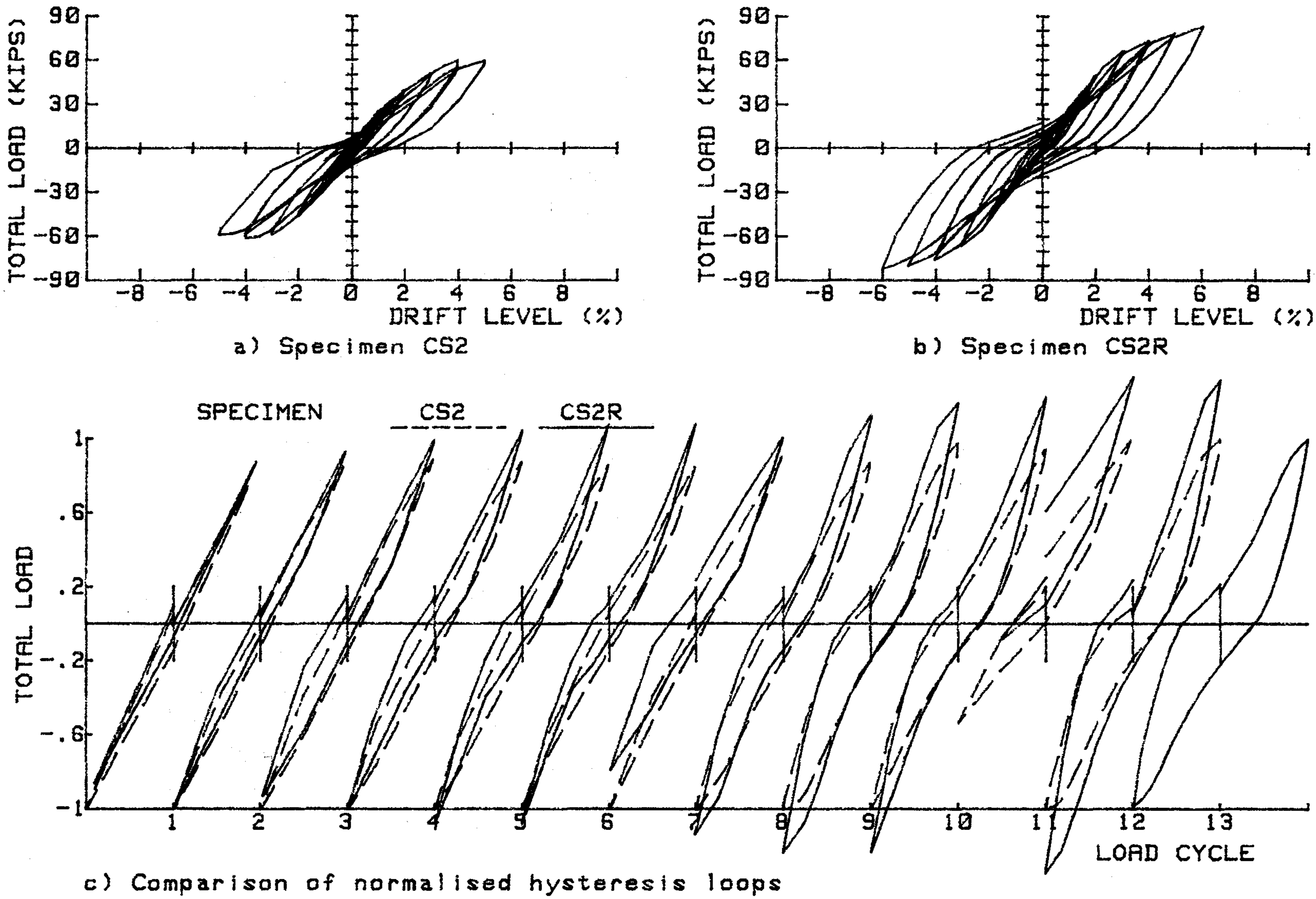
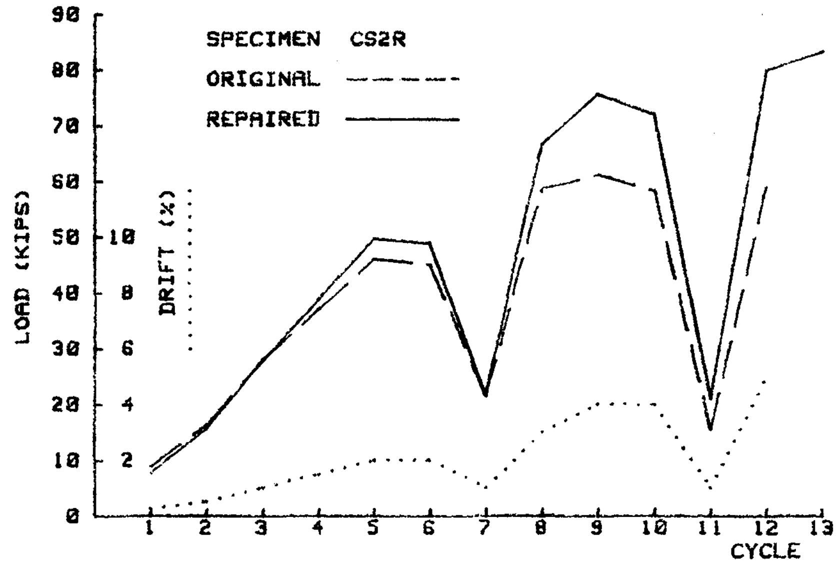
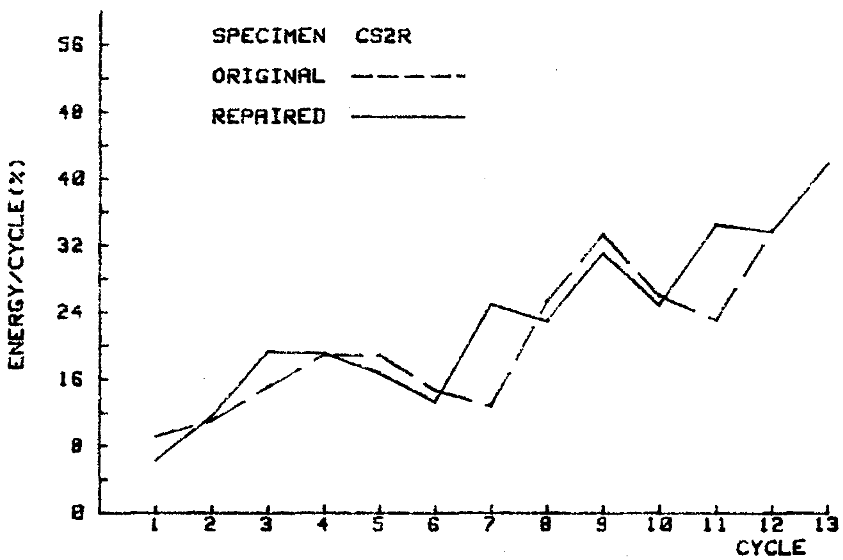


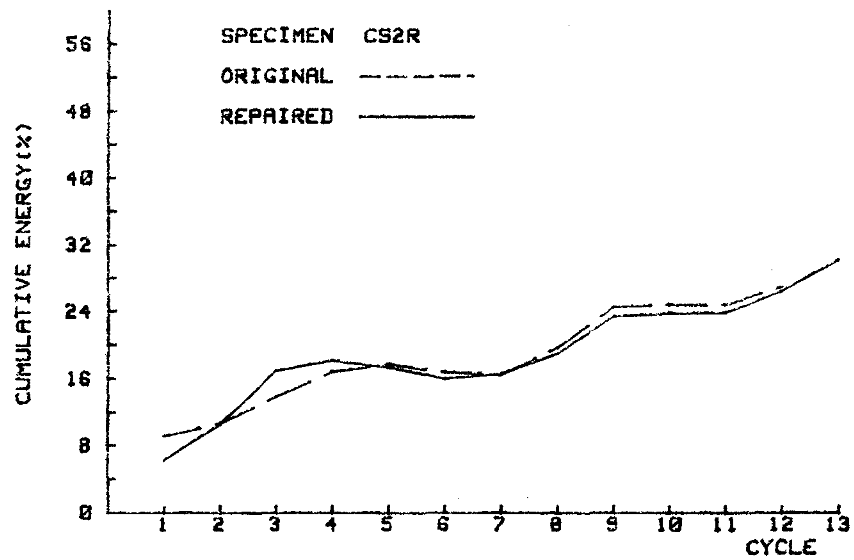
FIGURE 4-31 Load vs. Displacement Plots of Specimen CS2R



a) Load and displacement amplitudes



b) Discipated energy per cycle



c) Cumulative discipated energy

FIGURE 4-32 Strength and Energy Dissipation of Specimen CS2R

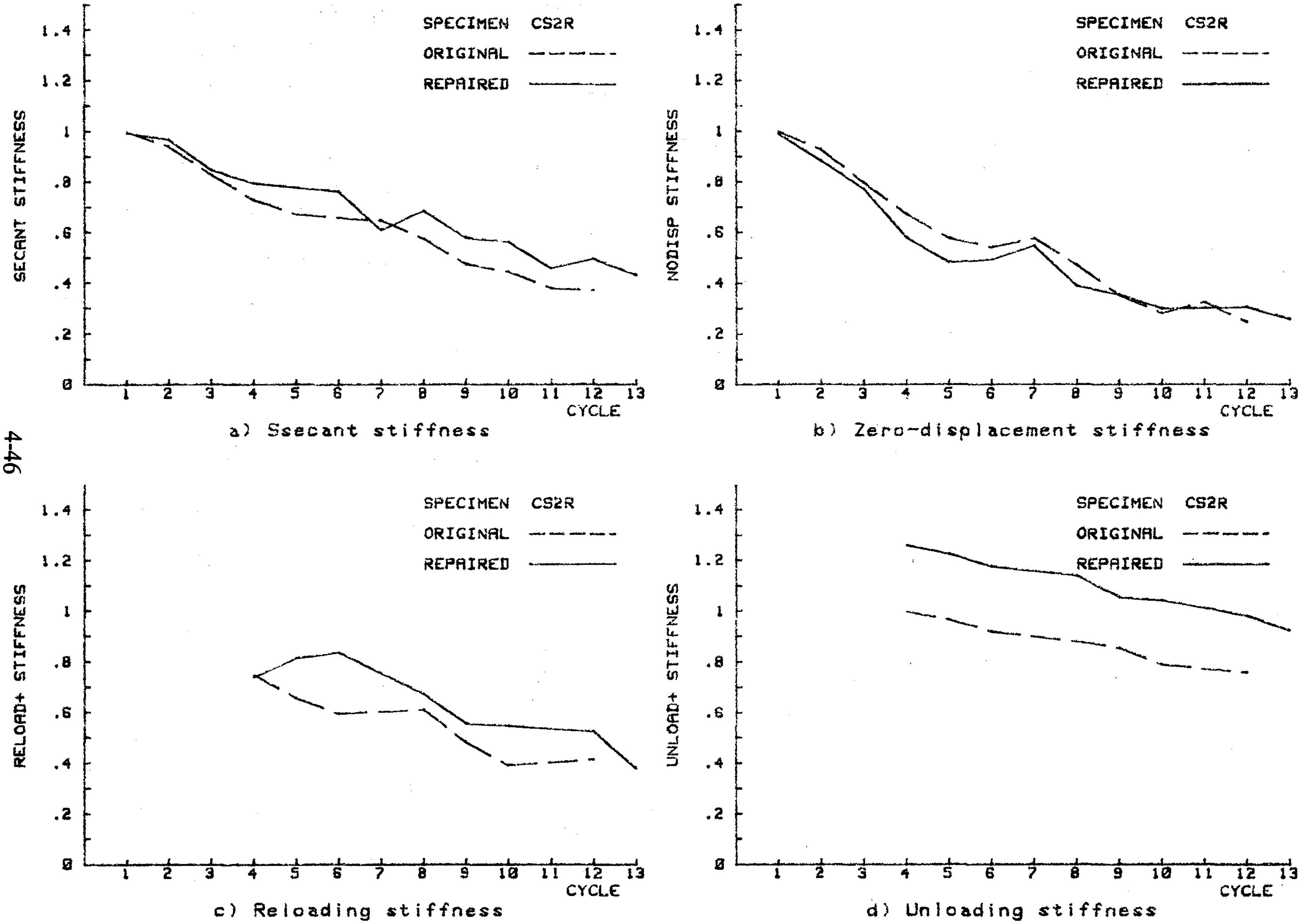
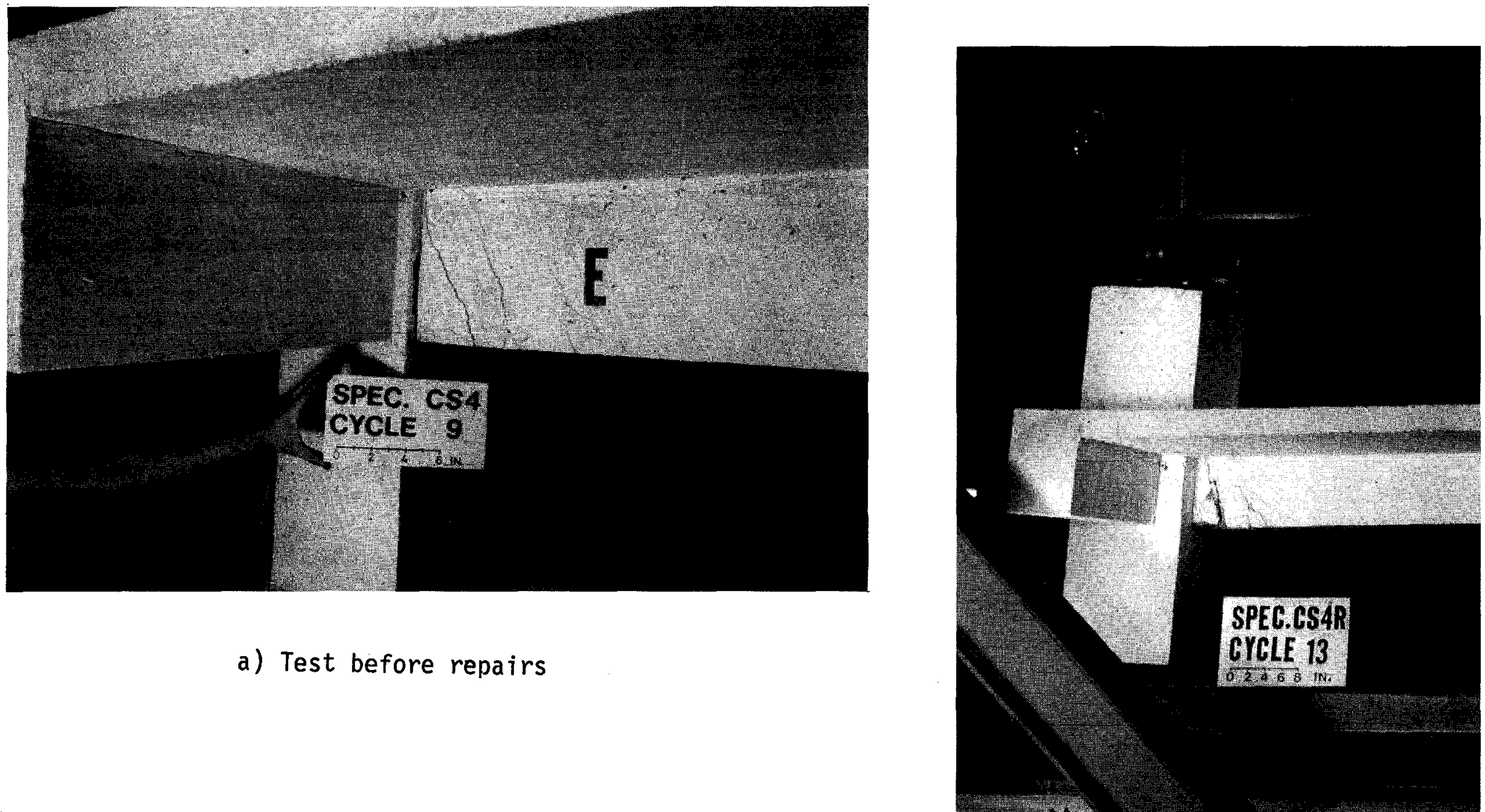


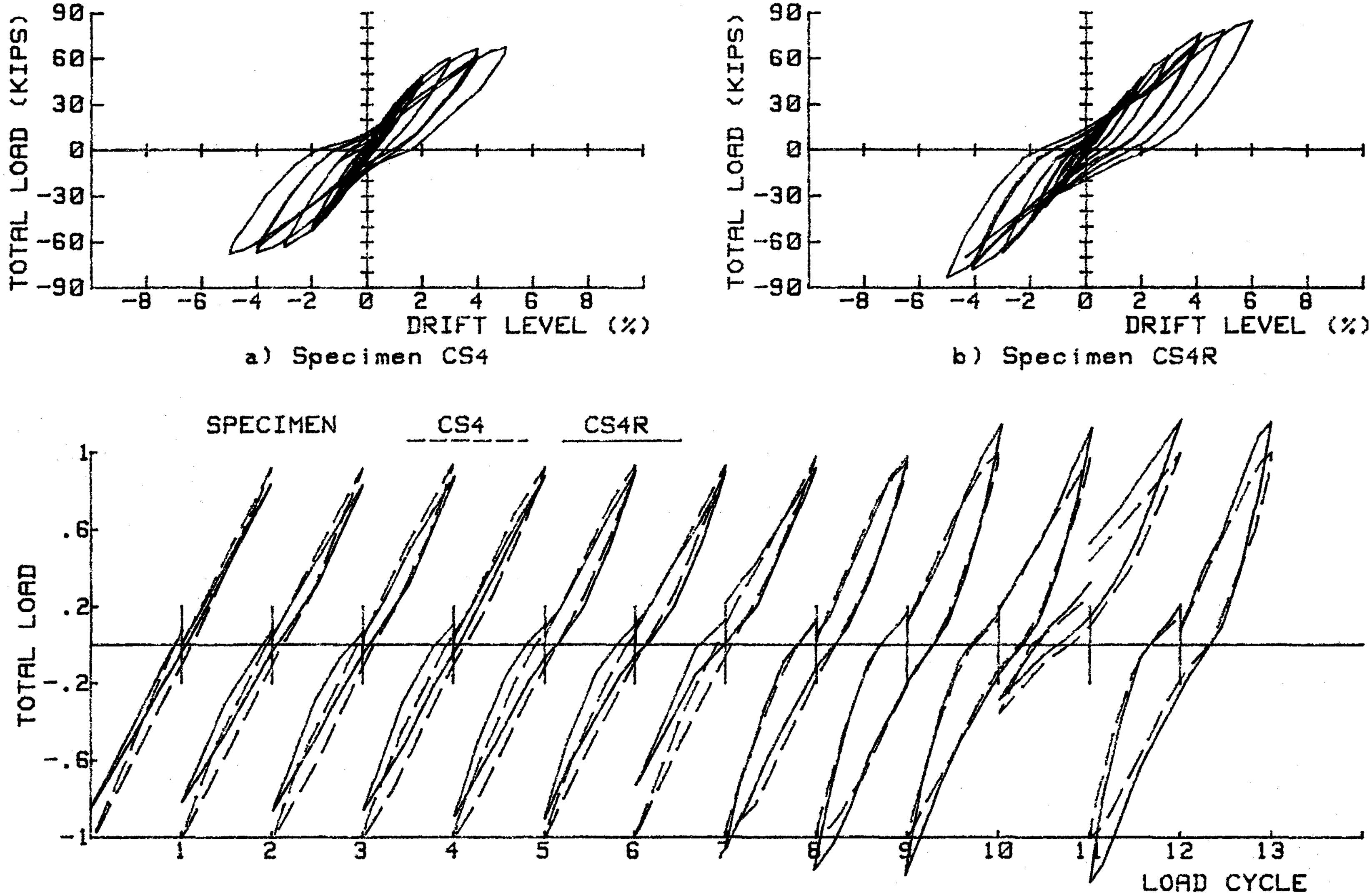
FIGURE 4-33 Stiffness Parameters of Specimen CS2R



a) Test before repairs

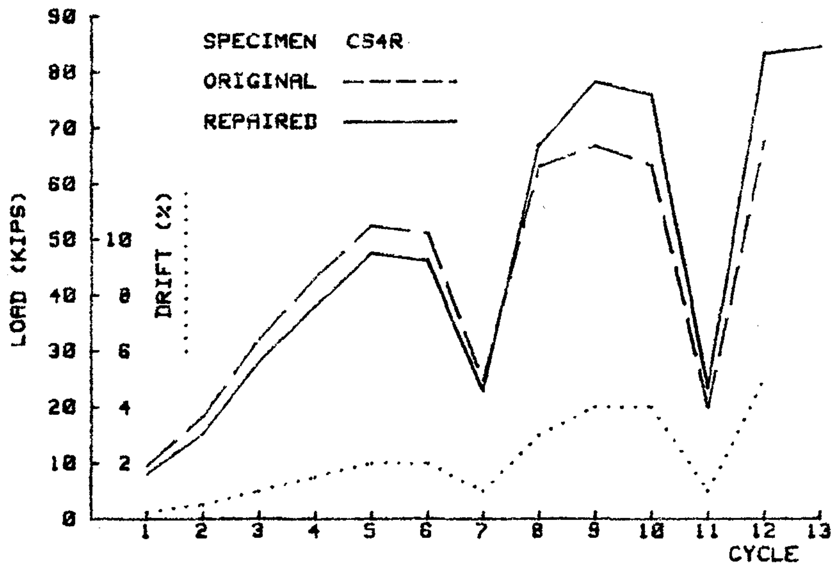
b) Test after repairs

FIGURE 4-34 Damage in Specimen CS4R Before and After Repairs

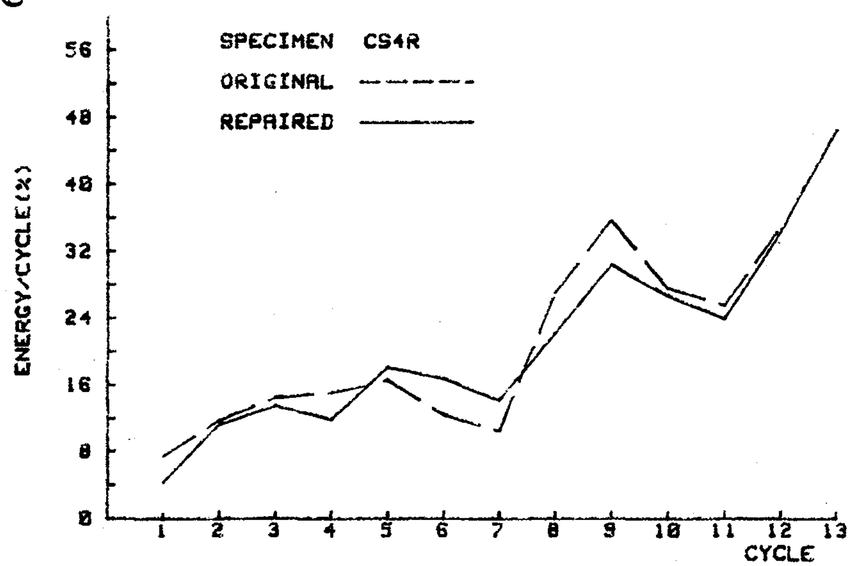


c) Comparison of normalised hysteresis loops

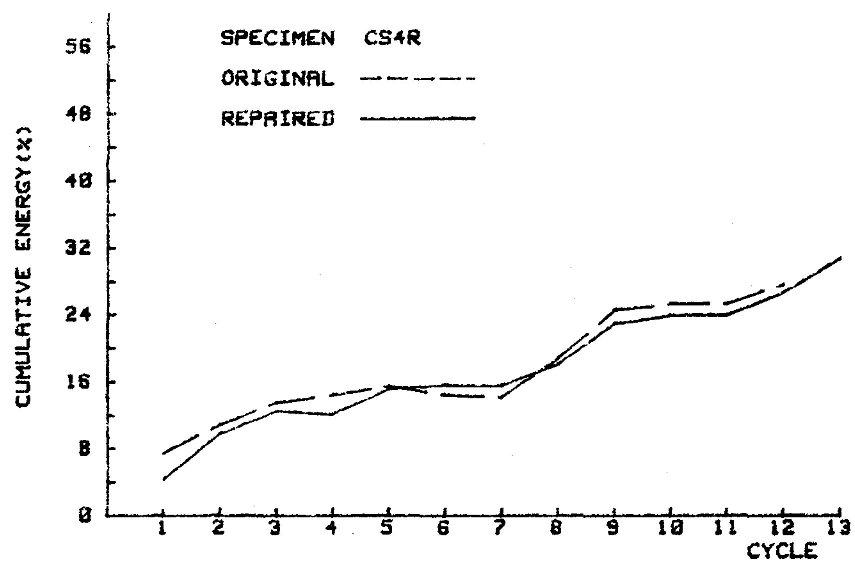
FIGURE 4-35 Load vs. Displacement Plots of Specimen CS4R



a) Load and displacement amplitudes

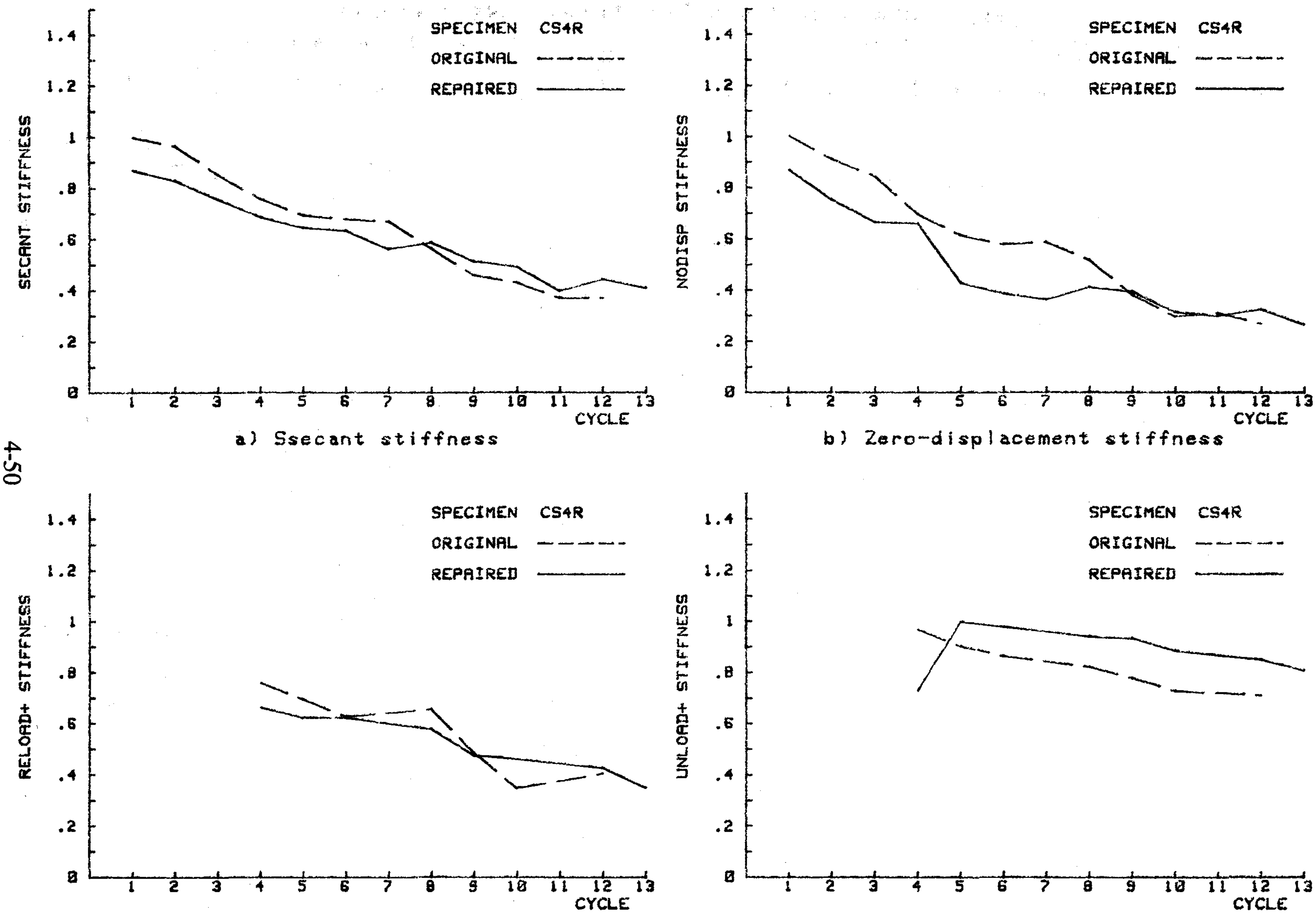


b) Discipated energy per cycle



c) Cumulative discipated energy

FIGURE 4-36 Strength and Energy Dissipation of Specimen CS4R



a) Ssecant stiffness

b) Zero-displacement stiffness

c) Reloading stiffness

d) Unloading stiffness

FIGURE 4-37 Stiffness Parameters of Specimen CS4R

4.2.4 Strengthening with External Steel Elements (Specimens ES2R, IS1R)

4.2.4.1 Repair Procedure

Addition of external steel elements proved relatively simpler than the repair procedures discussed earlier. The steel plates, angles, and dowels were prepared ahead of time in the shop. This preparation consisted mainly of cutting all elements to size, bending dowels where necessary, and sandblasting the plates to ensure good bond with the epoxy compound. The preparation work with the specimens was limited to drilling holes and roughening the surfaces to be in contact with new concrete. Transparent templates were used to transfer the patterns of holes in the specimens to the reinforcing plates.

After preparing the specimen, the dowels were anchored using a viscous epoxy resin mix and the plates bonded in place. The angles were then set in place using an epoxy mortar and welded to the plates. Finally, the concrete was cast around the connection region using forms that were quite simple to build.

Perhaps the only difficulty experienced in this procedure was encountered in drilling the holes at precise locations due to interference with reinforcing bars in the old concrete. This was however not a major problem, since corrections in the locations of holes could be made without great difficulty to avoid these interferences. The enlargements created in the holes were filled with the bonder used to set the dowels. The resulting irregular pattern of holes was easily transferred to the steel plates with the help of transparent templates.

It is important, however, to consider the resistance of this type of repair to heat. The epoxy resin is flammable and during a fire the structural integrity of the repair could be in jeopardy. At the same time, the toxic fumes released could be hazardous to building occupants. For these reasons the protection against fire should be a matter of serious concern.

4.2.4.2 Overall Performance

The cracking patterns for specimen ES2R during the tests before and after the repairs are shown in figure 4-38. Cracking during the test before repairs was considerable and the largest crack occurred at the joint face. After repairs, the most damage occurred in the beams near the end of the enlarged joint region and little damage was observed near the joint. The largest diagonal crack started in the beam and continued through the unreinforced corner of the concrete block. The cracks within the block were limited in size by the block reinforcement.

Figure 4-39 shows a top and bottom view of the connection at the end of cycle 13. Because of the enlargement of the connection, the damage was moved away from the column face. During the test after repairs, there was no evidence of any kind of damage in the column or its external reinforcement. Furthermore, no cracking or slippage was observed between the concrete and reinforcing plates.

The cracks in specimen IS1R during the tests before and after the repairs are shown in figure 4-43. Cracking during the test before repairs was not very extensive but most of it occurred near the joint. The repair moved the damage away from the column face to the end of the enlarged region. After repairs, the number of cracks and their sizes were larger due to the increased lateral load capacity of the specimen and hence the increased shear in the beam. Figure 4-44 shows the top and bottom views of the connection at the end of cycle 13. The cracks along the edge between the beam and the slab resulted from ultimate failure of the connections at the loading end of the beams and were not related to the behavior of the beam-to-column connection.

During the test after repairs, there was no evidence of damage in the column or its external reinforcement of specimen IS1R. However, some cracking was observed along the interface between concrete and reinforcing plates starting at cycle 6. This cracking was first found between the bottom steel plate and the beams, and later between the column and the surrounding plates. This cracking, however, was restricted by the dowel action of the bolts since no slippage was observed along these cracks. The increase in damage of this specimen, compared with ES2R, was due to higher lateral load resisted by IS1R, which is an internal joint.

4.2.4.3 Load vs. Displacement Loops

The load vs. displacement plots for these two specimens are shown in figures 4-40 and 4-45. In both cases, the specimens were stronger and stiffer after repairs even during the first few cycles. This is to be expected in repairs and strengthening with the addition of external steel elements.

The hysteresis loops of specimen ES2R showed very little pinching in the early cycles which indicates that enlarging the connection region adequately compensated for the shear damage in the joint. Furthermore, it significantly increased the strength in the postive loading direction which corresponds to compression in the bottom fiber of the beam. The increased stiffness is attributed to improved resistance of the enlarged connection and the surrounding region and to increased strength of column by the addition of steel plates.

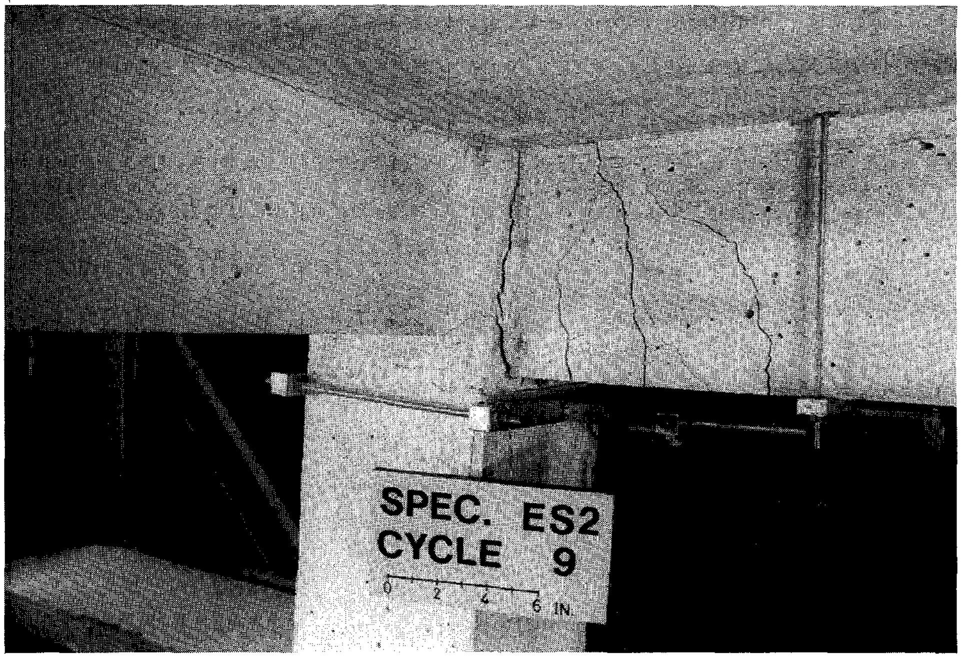
The joint region in specimen IS1R was further reinforced by adding a steel plate at the bottom of the enlarged region which helped in improving its resistance to both positive and negative bending. Thus the strength and stiffness of this specimen increased in greater proportion than in the case of specimen ES2R. The slight irregularities in the hysteresis loops of the repaired specimen are due to cracking and slippage along the interface between concrete and steel plates. When the cracks in the bonding material developed and slippage started, the forces were transmitted by dowel action of the bolts and the overall performance was not greatly affected. The strength of the specimen was limited by the strength of the beam which was considerably damaged by the end of the test. As shown in figure 4-46(a), the maximum load was reached during cycle 9 as was during the test before repairs. The peak loads during cycles 12 and 13 were limited by the strength of the connections at the loading ends of the beams.

4.2.4.4 Dissipated Energy

Figures 4-41b, c, and 4-46b, c show the plots of energy dissipated during the tests of these two specimens. The increase in energy dissipation of specimen ES2R after repair was in proportion to its increase in load amplitude during a given cycle. If it is assumed that the energy dissipated by steel in both specimens was about the same, both specimens appear to have damaged at about the same rate. Specimen IS1R dissipated considerably more energy during the test after repairs due to the slippage between concrete and steel plates. Since this slippage was limited by the dowel action of the bolts, it is believed that this minor slippage may not be detrimental to the safety of the structure and the additional dissipated energy would enhance its performance under dynamic loading.

4.2.4.5 Stiffness Parameters

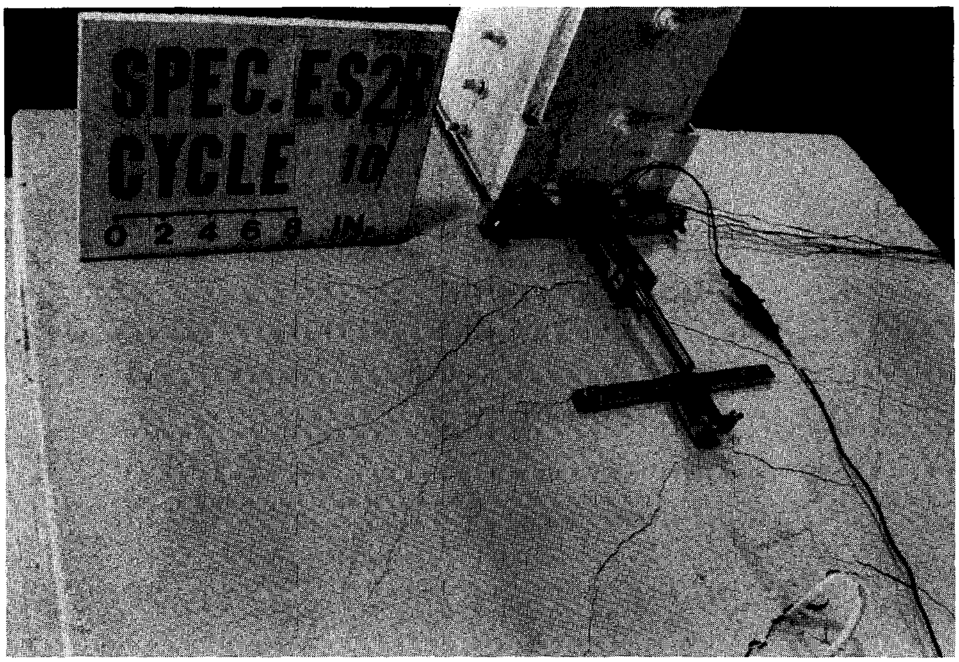
The stiffness parameters corresponding to each load cycle for these two specimens are shown in figures 4-42 and 4-47. The reloading and unloading stiffness of specimen ES2R increased by the addition of the reinforcements. The zero-displacement stiffness, however, was practically not affected by the repairs indicating that the rate of damage was about the same during both tests of the specimen. This observation substantiates the similar conclusion made earlier on the basis of energy dissipation capacity. In the case of specimen IS1R, all stiffness parameters increased as a result of the external reinforcing. The zero-displacement stiffness plots of the tests before and after repairs are essentially parallel and show again that the rate of damage during both tests was about the same.



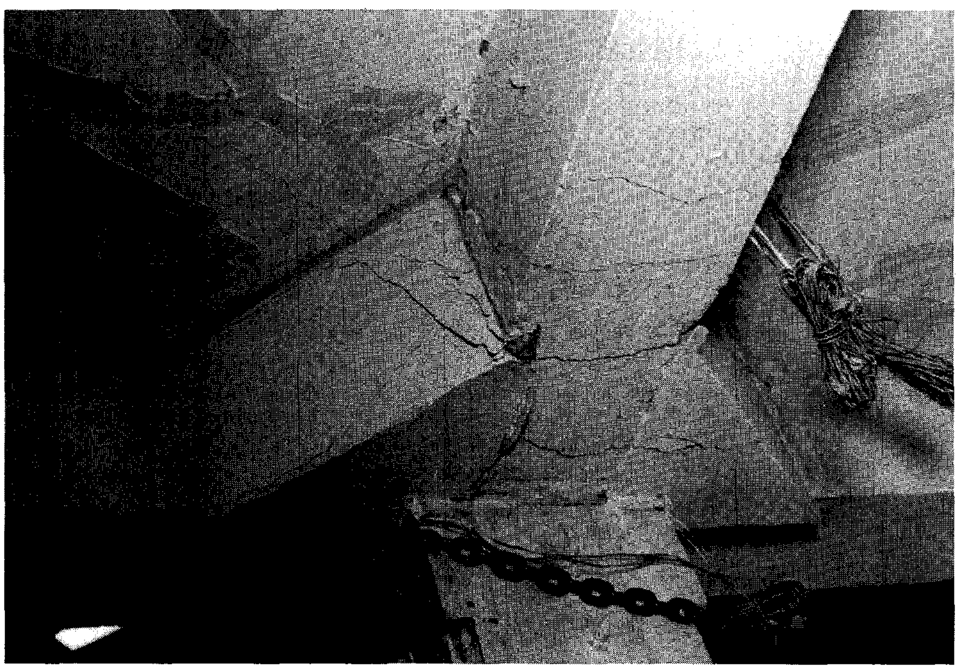
a) Test before repairs

b) Test after repairs

FIGURE 4-38 Damage in Specimen ES2R Before and After Repairs

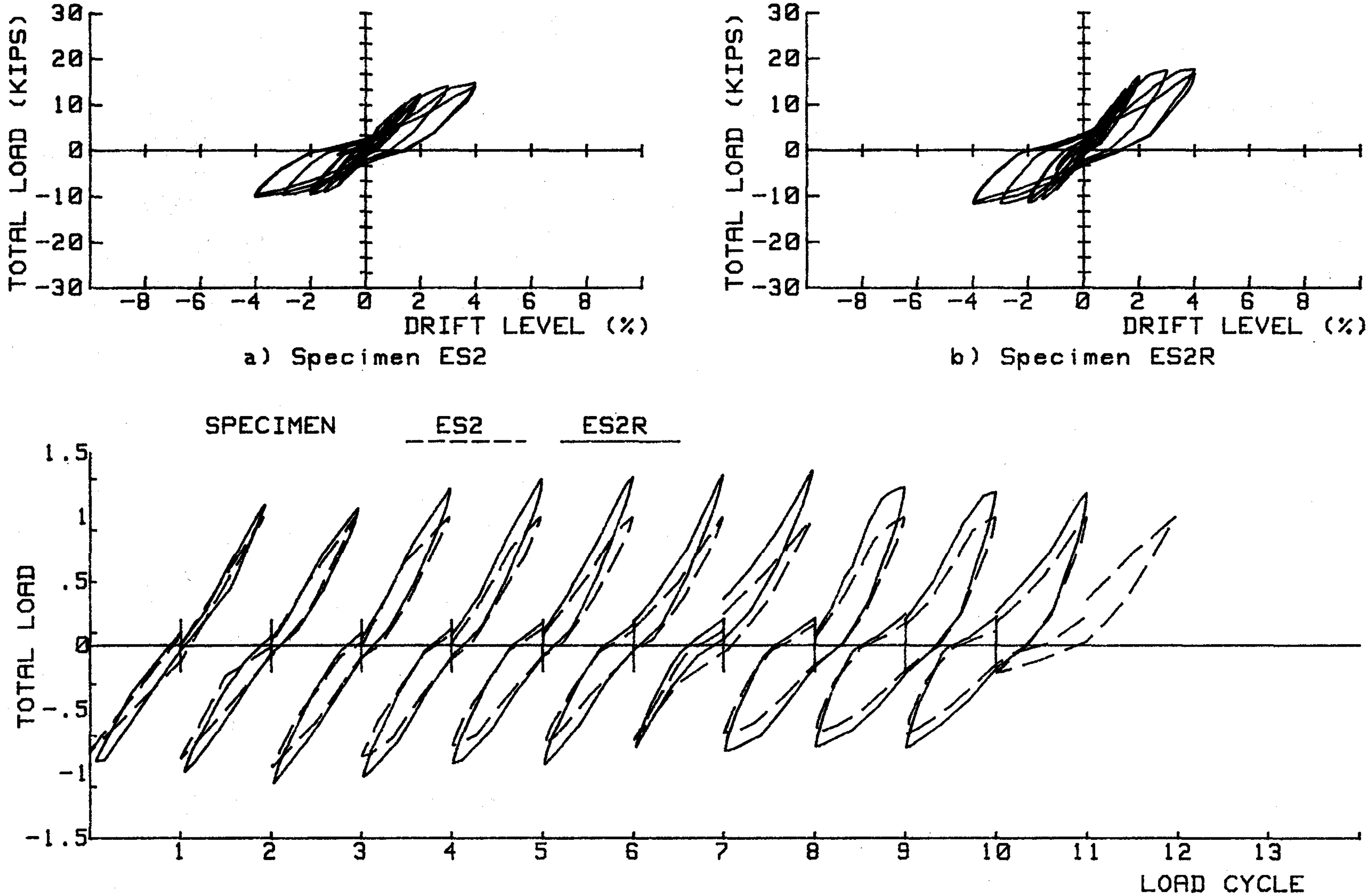


a) Cracking in the slab



b) Damage at the end of the test

FIGURE 4-39 Damage Detail in Repaired Specimen ES2R

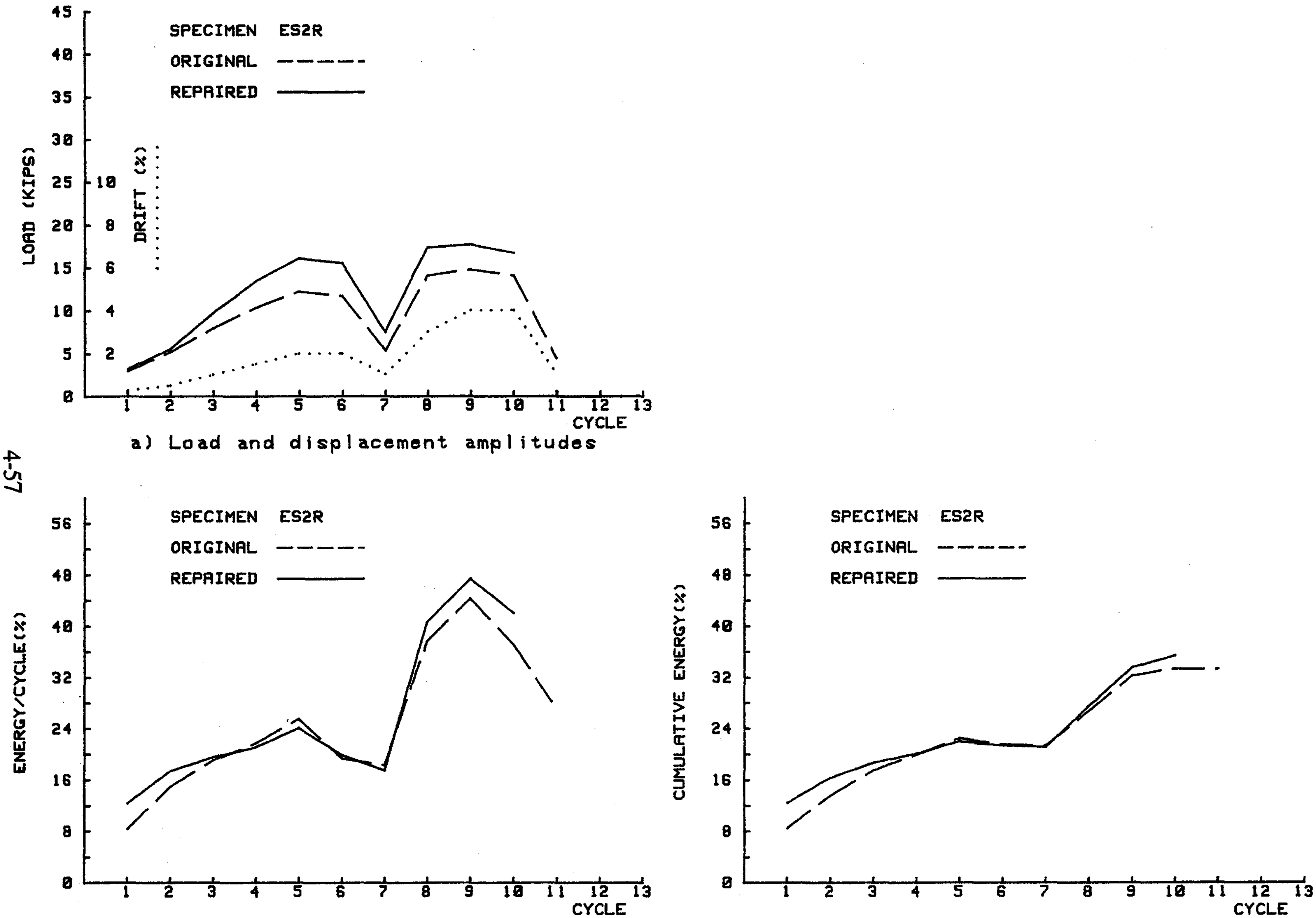


a) Specimen ES2

b) Specimen ES2R

c) Comparison of normalised hysteresis loops

FIGURE 4-40 Load vs. Displacement Plots of Specimen ES2R



a) Load and displacement amplitudes

b) Discipated energy per cycle

c) Cumulative discipated energy

FIGURE 4-41 Strength and Energy Dissipation of Specimen ES2R

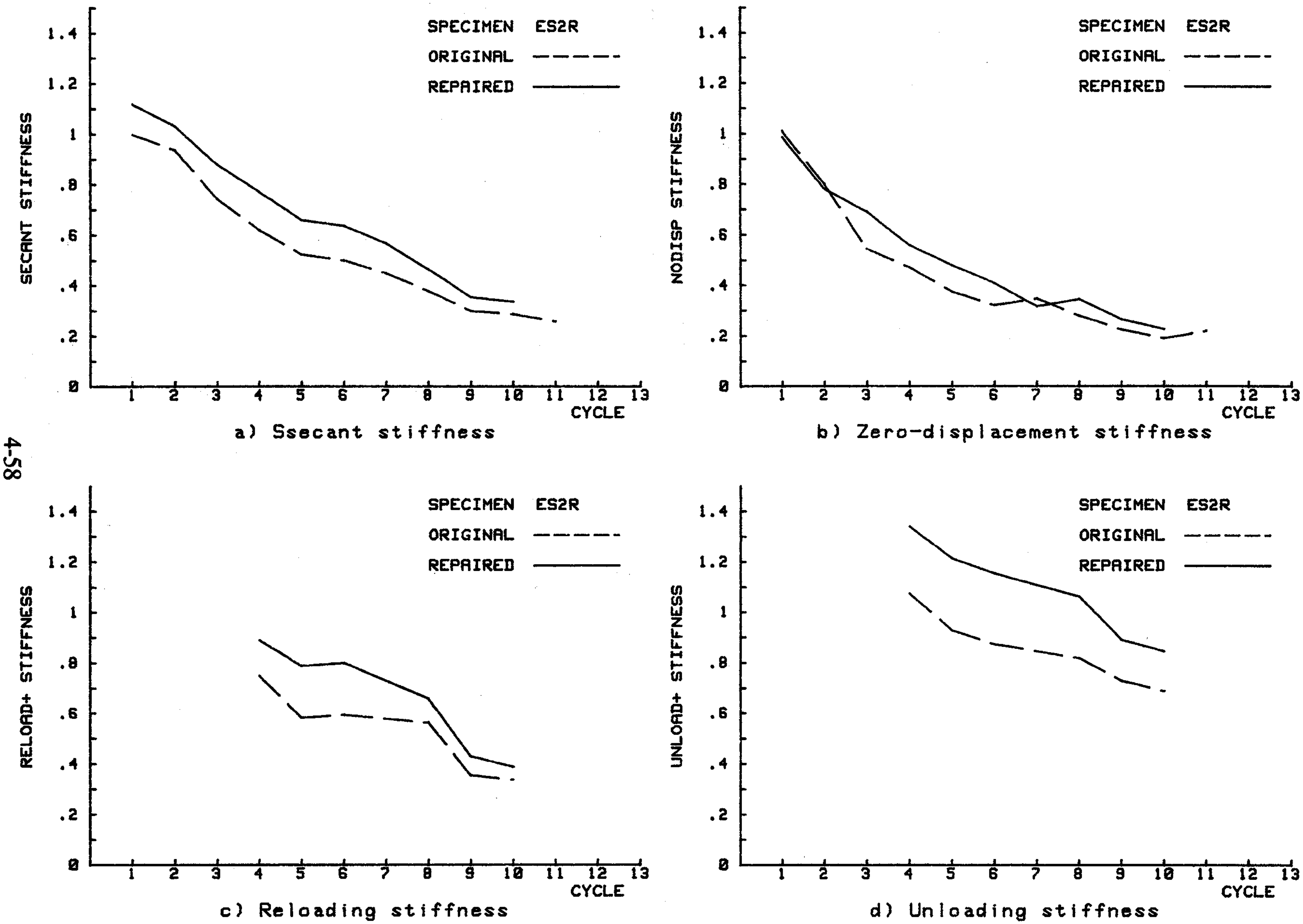
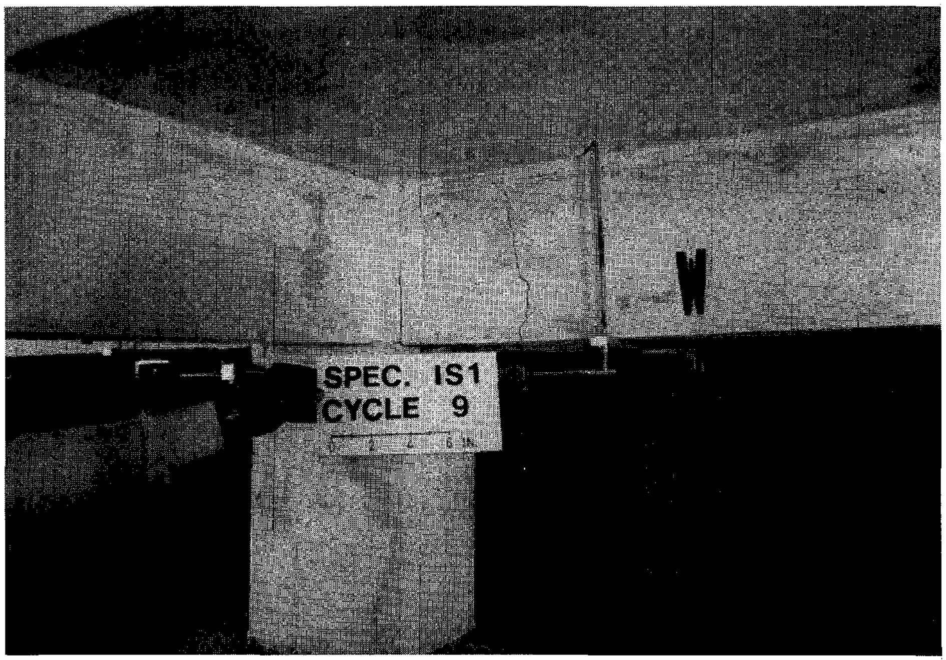


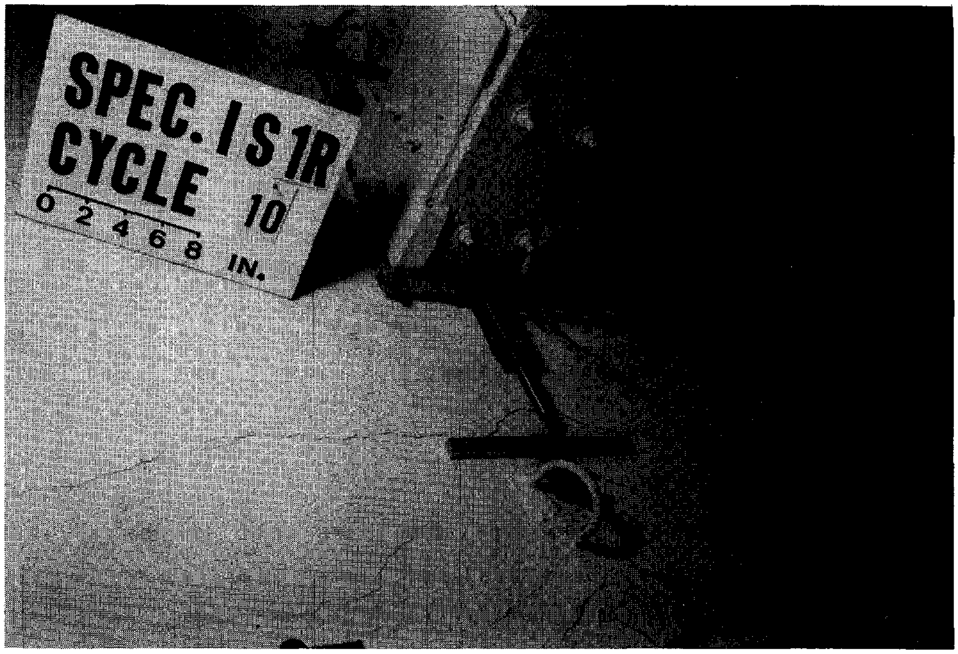
FIGURE 4-42 Stiffness Parameters of Specimen ES2R



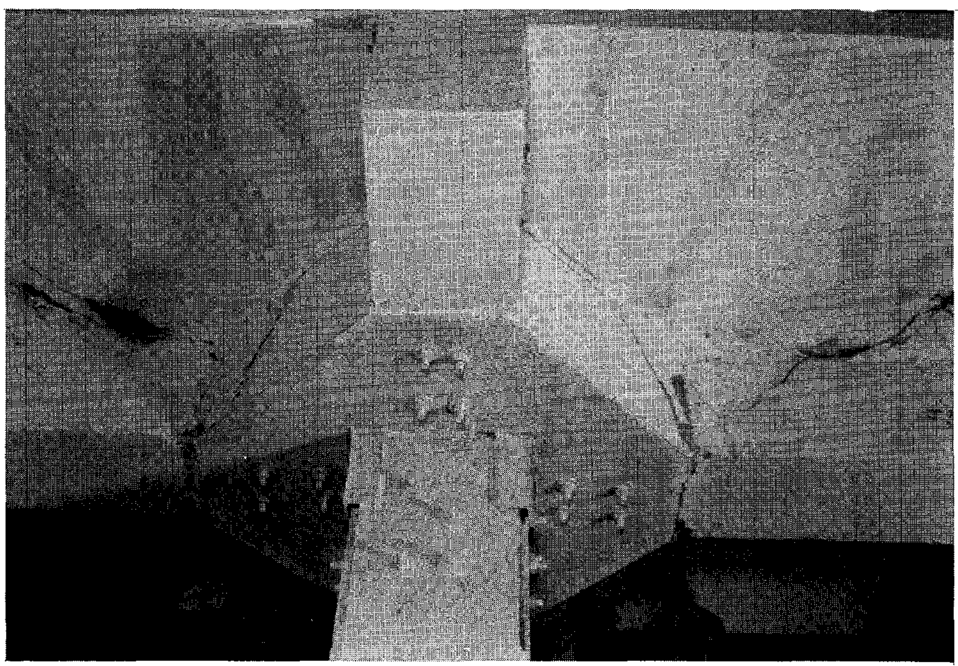
a) Test before repairs

b) Test after repairs

FIGURE 4-43 Damage in Specimen IS1R Before and After Repairs



a) Cracking in the slab



b) Damage at the end of the test

FIGURE 4-44 Damage Detail in Repaired Specimen IS1R

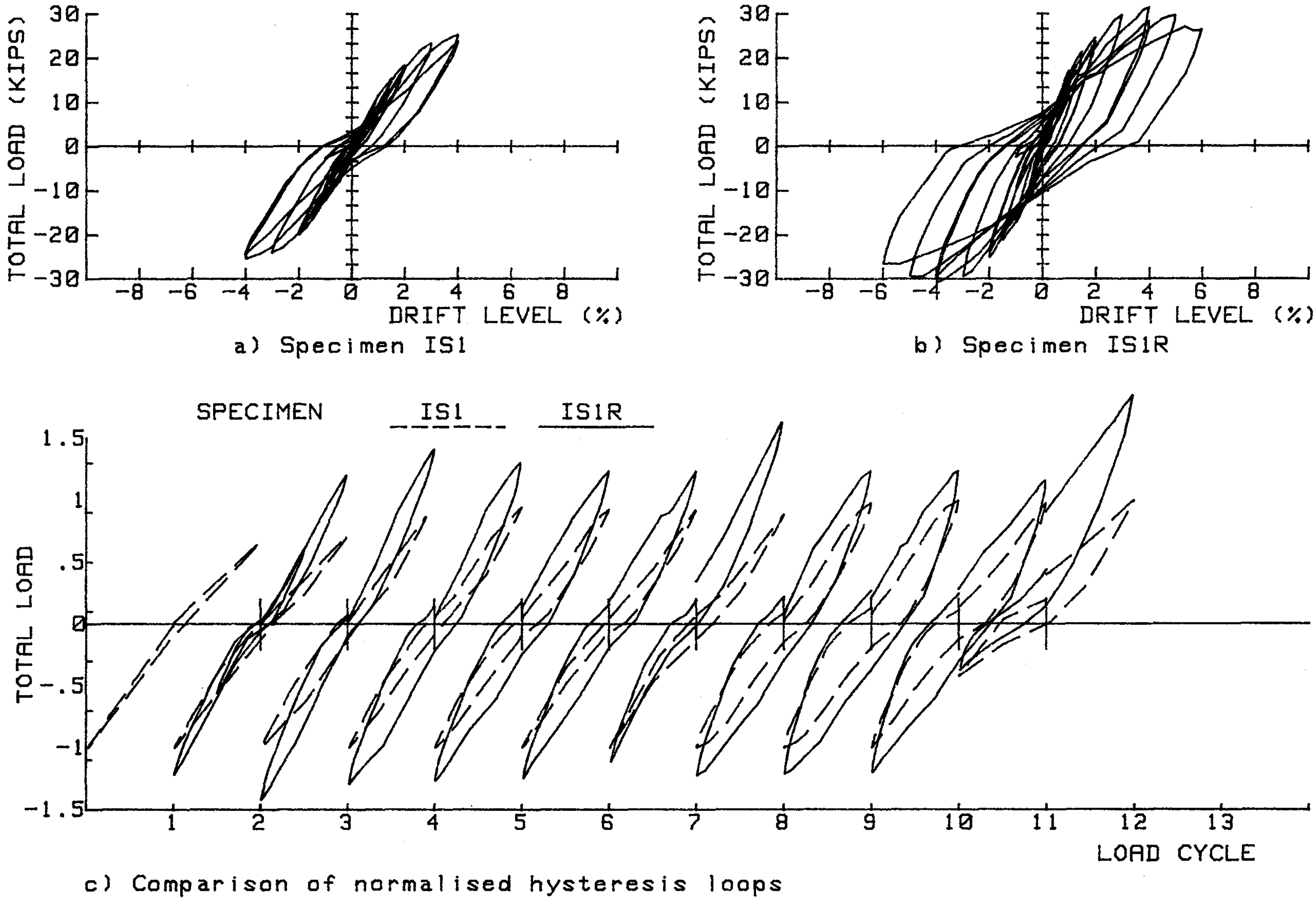
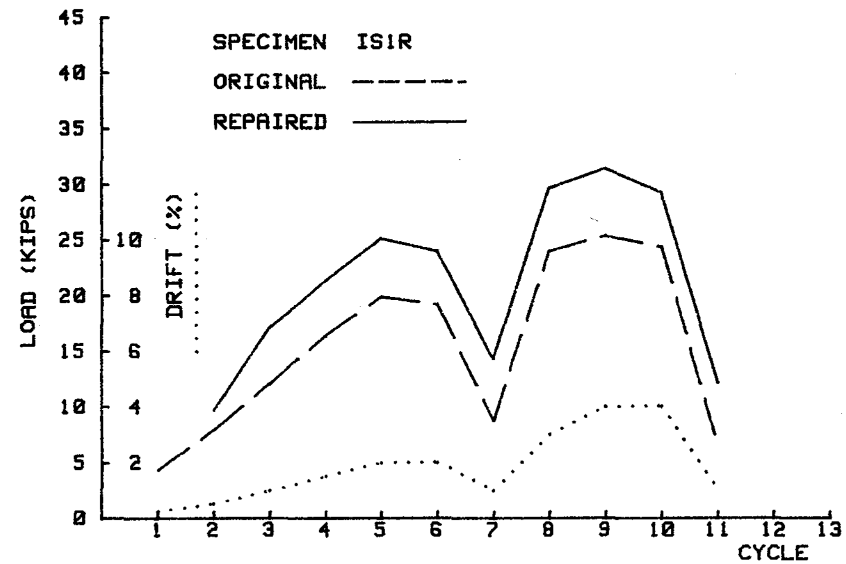
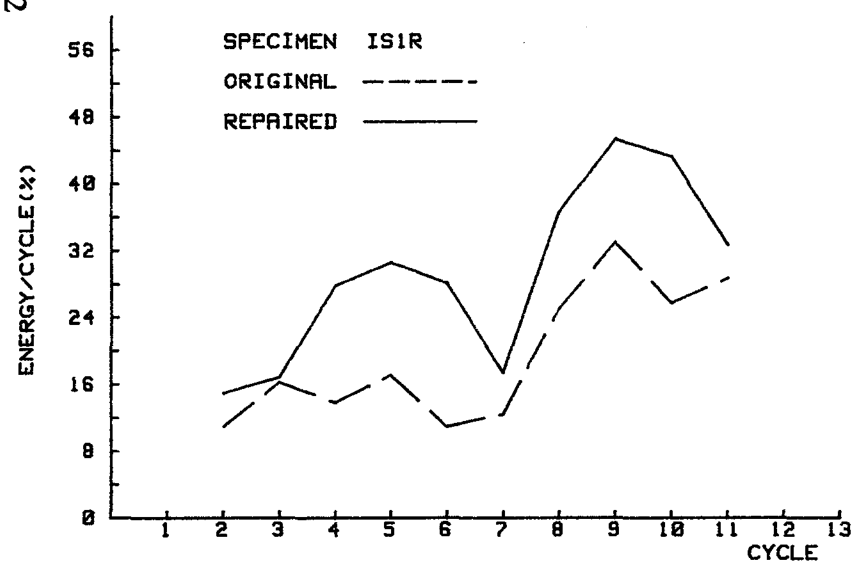


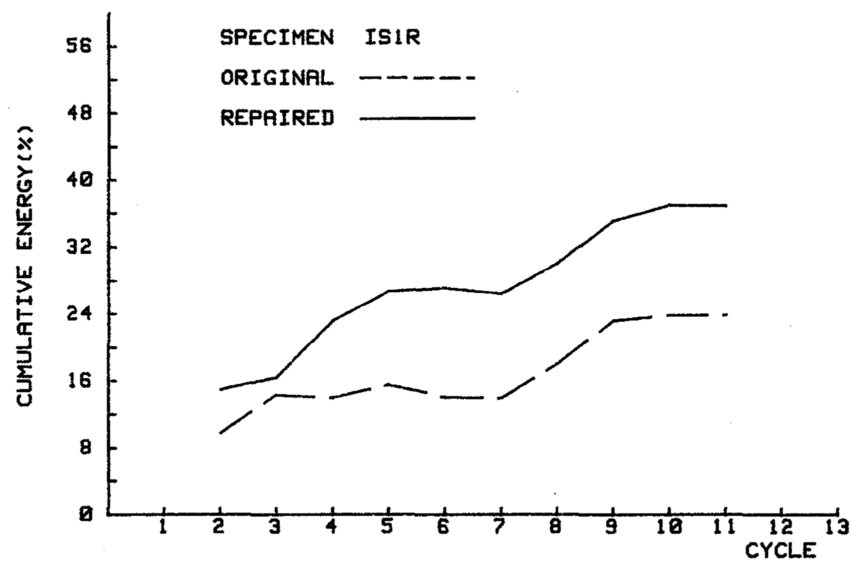
FIGURE 4-45 Load vs. Displacement Plots of Specimen IS1R



a) Load and displacement amplitudes



b) Discipated energy per cycle



c) Cumulative discipated energy

FIGURE 4-46 Strength and Energy Dissipation of Speciment IS1R

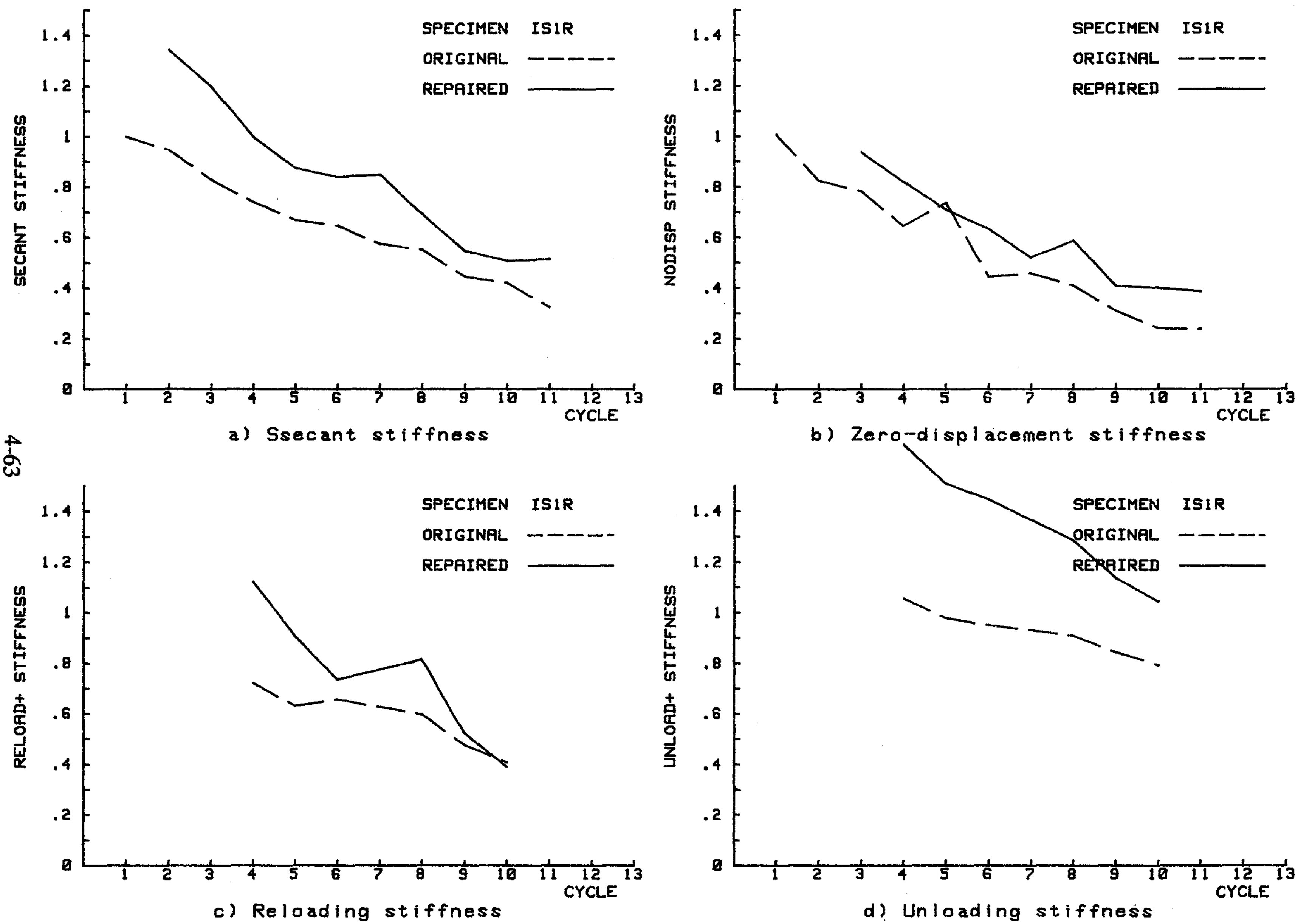


FIGURE 4-47 Stiffness Parameters of Specimen IS1R

SECTION 5

CONCLUSIONS

5.1 Feasibility of Repair Methods

5.1.1 Epoxy injection

Epoxy injection can be well suited for repairing beams and slabs. However, in repairing beam-to-column connections, the injection of epoxy can be difficult and its effectiveness depends highly on the quality of work.

5.1.2 Replacement of damaged concrete

When temporary shoring can be economically provided, the replacement of damaged concrete can be relatively simple. This technique is appropriate for repairing localized damage such as flexural hinging regions in the beams. Replacing concrete in the joints in a real building may not be very practical.

5.1.3 Enlargement of member sections with concrete jacket

Strengthening by jacketing required more labor than the other repair techniques implemented in this study. The placement of continuous ties around the column in the joint region can be extremely difficult if not impossible. For practicality, some compromises in details were made which proved equally effective. The major advantage of this technique is perhaps the reliability of the procedure since it is based on well established techniques used in reinforced concrete construction.

5.1.4 Addition of External steel elements

Strengthening by the addition of external steel elements can be most economically applied in practice due to lower labor requirement. Some secondary problems such as fire protection and aesthetics must however be properly addressed.

5.2 Behavior Of Repaired Specimens

5.2.1 Epoxy injection

Epoxy injection by itself may not be adequate for restoring the strength and stiffness of beam-to-column connections damaged by earthquake loading. Particularly, restoring the bond and anchorage of bars with epoxy injection can be difficult and unreliable.

5.2.2 Replacement of damaged concrete

The strength of members can be effectively restored by replacing damaged concrete. However, stiffness and energy dissipation characteristics of the subassembly may not be restored because of damage in the connection which cannot be easily detected and also may not be readily accessible.

5.2.3 Enlargement of columns with reinforced concrete

Enlarging the column cross section alone can be quite effective and reliable in increasing the strength of a connection. The stiffness and energy dissipation characteristics, however, may not restored by the same proportion unless the damage suffered by the beams is also properly addressed.

5.2.4 Enlargement of columns and adjacent beam segments with reinforced concrete

The strength, stiffness and, energy dissipation capabilities of connection subassemblies can be effectively restored by jacketing the columns along with the beam segments adjacent to the columns. An added advantage of this procedure is that the likely zone of damage during future earthquakes is moved to a predetermined location away from the joint.

5.2.5 Addition of external steel elements

This repair technique has been commonly used in practice in repairs of damaged buildings with mixed results. The tests have shown that design details that properly address the transfer of forces through the joint can be quite effective in restoring and improving the structural performance of connections. As in the case of enlargement of column and adjacent beam segments, the flexural hinge can be removed away form the joint using the design details and repair procedures used in this study.

SECTION 6
REFERENCES

1. Bass, R. A., "An Evaluation of the Interface Shear Capacity of Techniques Used in Repair and Strengthening Reinforced Concrete Structures", Masters Thesis, The University of Texas at Austin, August 1984.

2. Bett, B. J., et al ,"Behavior of Strengthened and Repaired Reinforced Concrete Columns Under Cyclic Deformations", PMSFL Report N0. 85-3, The University of Texas at Austin, December 1985.

3. Chung, H. W., "Epoxy-Repaired Reinforced Concrete Beams", Journal of the American Concrete Institute, May 1975.

4. Degenkolb, H. J. and Associates, "Repair and Strengthening of Reinforced Concrete Structures", San Francisco, California, 1983.

5. Hanson, R., D., "Repair, Strengthening and Rehabilitation of Buildings-- Recommendations for Research", University of Michigan, Report No. UMEE 77R4, October 1977.

6. Hanson, R., D., "Repair and Strengthening of Reinforced Concrete Members and Buildings", Proceedings, Workshop on Earthquake Resistant Reinforced Concrete construction, University of California, Berkeley, July 1977.

7. Higashi, Y. et al, "Behavior of Reinforced Concrete Columns and Frames Strengthened by Adding Precast Concrete Walls", Sixth World Conference on Earthquake Engineering, New Deli, India, 1977.

8. Higashi, Y. et al, "Effects on Behavior of Reinforced Concrete Frames by Adding Shear Walls", Proceedings of the Second Seminar on Repair and Retrofit of Structures, Ann Arbor Michigan, 1981.

9. Higashi, Y. et al, "Experimental Studies on Retrofitting of Reinforced Concrete Structural Members", Proceedings of the Second Seminar On Repair and Retrofit of Structures, Ann Arbor, Michigan, 1981.

10. Higashi, Y., "Strengthening of Reinforced Concrete Structures", Research Report, Higashi Laboratory, Tokyo, Japan, 1981.

11. Higashi, Y., and Kokusho, S., "The Strengthening Methods of Existing Reinforced Concrete Buildings", Proceedings of the Review Meeting U.S.- Japan Cooperative Research Program in Earthquake Engineering with Emphasis on The Safety of School Buildings", Honolulu, Hawaii, August 1985.

12. Hirosawa, M. et al, "Retrofitting on Medium-Rise Reinforced Concrete Housing Structures", Proceedings of the Second Seminar on Repair and Retrofit of Structures, Ann Arbor, Michigan, 1981.

13. Jones, E., A., and Jirsa, J., O., "Seismic Strengthening of a Reinforced Concrete Frame Using Structural Steel Bracing", PMFSEL Report No. 86-5, The University of Texas at Austin, May 1986.

14. Kawamata, S., and Ohnuma, M., "Strengthening effect of Eccentric Steel Braces on Existing Reinforced Concrete Frames", Proceedings of the Second Seminar on Repair and Retrofit of Structures, Ann Arbor, Michigan, 1981.

15. Lee, D. L. N., "Original and Repaired Reinforced Concrete Beam-Column Subassemblages Subjected to Earthquake Type Loading", Dissertation for the degree of Doctor of Philosophy, Report UMEE 76R4, Department of Civil Engineering, University of Michigan, April, 1976.

16. Liu, T., C., and Holland, T., C., "Design of Dowels for Anchoring Replacement Concrete to Vertical Lock Walls", Technical Report SL-81-1. U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, March 1981.

17. Luke, P. C. et al, "Epoxies for Use of Grouting Reinforcing Bar Dowels in Concrete", PMFSEL Report No. 85-3, The University of Texas at Austin, May 1984.

18. Luke, P., "Strength and Behavior of Rebar Dowels Epoxy Bonded in Hardened Concrete", Masters Thesis, The University of Texas at Austin, May 1984.

19. Mahin, S. A., and Bertero, V. V., "Rate of loading Effects on Uncracked and Repaired Reinforced Concrete Members", Report No. EERC 72-9, University of California, Berkeley, California, 1972.

20. Popov, E. P. and Bertero, V. V., "Repaired R/C Members Under Cyclic Loadings", 5th Congress of the Yougoslav Association of Structural Engineers and 3rd Congress of the Yougoslav Association for Earthquake Engineering.

21. Roach, C. E., and Jirza, J. O., "Seismic Strengthening of a Reinforced Concrete Frame Using Reinforced Concrete Piers", PMFSEL Report No. 86-4, The University of Texas at Austin, May 1986.

22. Shimizu, Y., et al, "Aseismic Capabilities of Strengthened Reinforced Concrete Frame", Research Report, Hgashi Laboratory, Tokyo, Japan, 1981.

23. Sugano, S., "An Overview of The State-of-the-Art in Seismic Strengthening of Existing Reinforced Concrete Buildings in Japan", Proceedings of the Third Seminar on Repair and Retrofit of Structures, Ann Arbor, Michigan, 1982.

24. Sugano, S., "Aseismic Strengthening of Existing Reinforced Concrete Buildings", Proceedings of the First Seminar on Repair and Retrofit of structures, Ann Arbor, Michigan, 1980.

25. Sugano, S., "Guideline for Seismic Retrofitting (Strengthening, Toughening and/or stiffening) Design of Existing Reinforced Concrete Buildings", Proceedings, Second Seminar on Repair and Retrofit of Structures, Ann Arbor Michigan, May 1981.

26. Sugano, S., and Endo, T., "Seismic Strengthening of Reinforced Concrete Buildings in Japan", Strengthening of Building Structures - Diagnosis and Theory, IABSE Symposium, Venice, Italy, 1983.

27. Sugano, S. and Fujimura, M., "Aseismic Strengthening of Existing reinforced Concrete Buildings", Seventh World Conference in Earthquake Engineering, Istambul, Turkey, 1980.

28. Townsend, W., H., et al,"Inelastic Tests of Repaired Concrete Connections", ASCE National Structural Engineering Meeting, San Francisco, California, April 1973.

29. Viwathanatepa, s., and Popov, E. P., "Seismic Behavior of Reinforced Concrete Interior Beam-Column Subassemblages", Report No. EERC-79/14, University of California at Berkeley, California, June 1979.

30. Wiener, D., "Behavior of Steel to Concrete Connections Used to Strengthen Existing Structures", Masters Thesis, The University of Texas at Austin, August 1985.

31. Wyllie, L., A. Jr., "Strengthening Existing Concrete and Masonry Buildings For Seismic Resistance", Proceedings of the Second Seminar on Repair and Retrofit of Structures, Ann arbor, Michigan 1981.

32. Wyllie, L. A., and Dean, R. G.,"Seismic Failures and Subsequent Performance after Repair", ASCE National Structural Engineering Convention, New Orleans, Louisiana, April 1975.

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NCEER-89-0010 "NCEER Interim Bibliography of Earthquake Education Materials," by K.E.K. Ross, 3/10/89.

NCEER-89-0011 "Inelastic Analysis of Three-Dimensional Reinforced Concrete Building Structures, Part I - Modeling," by S.K. Kunnath and A.M. Reinhorn, 1/29/89, to be published.

NCEER-89-0012 "Recommended Modifications to ATC-14," by C.D. Poland and J.O. Malley, 4/12/89.

NCEER-89-0013 "Repair and Strengthening of Beam-to-Column Connections Subjected to Earthquake Loading," by M. Corazao and A.J. Durrani, 2/28/89.