REPAIR OF DAMAGED REINFORCED CONCRETE FRAME STRUCTURES

by

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SYNOPSIS

An investigation on the effectiveness of repair of reinforced concrete exterior beam-column subassemblages is presented. The epoxy injection technique, and the removal and replacement technique using high early strength materials were used to repair the beams. Based on the results of this investigation, it was concluded that epoxy injection, and removal and replacement techniques of repair can restore structural integrity to the members.

INTRODUCTION

It is recognized as impractical, if not impossible, to design a structure which will withstand a severe earthquake without damage. Structural and nonstructural damage should be anticipated during a severe earthquake. The repair of this damage should at least restore the structure's capacity to resist gravity and lateral loads. In many instances, an increase in the lateral load resistance capacity would be desirable. The damaged portions of the structure are usually repaired with a high strength material so that the repairs are sufficient to carry gravity loads. However, the capacity of the structure to withstand seismic loads is suspect. Poor behavior of a repaired area during a subsequent earthquake may result in a catastrophic collapse of the structure. There is a need for additional data (1,2) to determine the reliability of repaired structural members. The experimental research described here which was performed at The University of Michigan provides some of this data.

EXPERIMENTAL PROGRAM

Design of Specimens. The specimens used in this investigation were half size exterior beam-column subassemblages. These subassemblages were designed using two criteria. For the first design, referred to here as "Type I Design", the ACI 318-71 Code (3) for nonseismic areas was used. This design was assumed to represent existing structures which were designed without considering seismic loading. For the second design, referred to as "Type II Design", ACI 318-71 Code (along with its special provisions for the design of ductile moment-resisting space frames) and recommendations from ACI-ASCE Joint Committee 352 for the design of beam-to-column joints (4) were used. The primary difference between the subassemblages resulting from the two design criteria was the amount of transverse reinforcement in the beam,

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column, and beam-to-column joint. Overall dimensions of the beam-column subassemblages and the details of the reinforcing steel for the Type II Design specimens are shown in Fig. 1.

The specified concrete strength for the beam-column specimens was 4,000 psi (28 N/mm^2). The nominal yield strength for the column longitudinal reinforcement was 60,000 psi (410 N/mm^2), and the nominal yield stress for the beam main reinforcement, and all shear reinforcement was 40,000 psi (280 N/mm^2).

Testing. The original and repaired specimens were tested in a horizontal position. Rollers were placed vertically on each side at both ends of the column to obtain a simple support condition. This condition represented inflection points at midstory height. Either a zero axial load or a 40,000 lb. (178,000 N) axial load, which represented about 35 percent of the design balanced axial load for the column, was applied to the column and then the tip of the beam was slowly deflected by a hydraulic actuator.

To obtain different degrees of damage during original testing, two beam displacement patterns were used, the first represented the type of loading which may occur during a moderate earthquake and the second represented a severe earthquake loading. These displacement patterns are shown in Fig. 2. Ductility is defined as beam-tip deflection divided by the corresponding deflection at first yielding of the top beam bar. For retest, each repaired specimen was subjected to the same beam displacement pattern as used during original testing so that direct comparisons of the results could be made.

Data Acquisition. During original testing and retesting of the specimens, a continuous record of the beam-tip force and deflection was recorded and used to monitor the progress of the test. These data were later used to analyze the stiffness, strength, and energy dissipation capacity of the beam-column specimens.

REPAIR OF SPECIMENS

In this investigation, two techniques were used to repair the beam-column specimens. For the specimens which were subjected to the moderate earthquake loading and which sustained only cracking, the epoxy injection technique was used. For this technique, low viscosity epoxy was injected into the cracks for the purposes of restoring bond between the reinforcing steel and concrete, and continuity in the concrete. The specimens subjected to the severe earthquake loading sustained extensive damage and were repaired using the removal and replacement technique. This technique consisted of removing the loose concrete and inspecting the reinforcement for necking and fracture. For Type I designed specimens, transverse reinforcement was added in the void area to satisfy the ACI 318-71 Code seismic requirement for confinement. The resulting void was then filled with a high early strength replacement material. The high early strength of the material enables the structure to be reoccupied in the shortest possible time. The different replacement materials used were epoxy-sand mortar, Duracel cement concrete (produced by United States Gypsum Company, Chicago, Illinois, U.S.A.), high strength quick setting concrete (developed by Republic Steel, Cleveland, Ohio, U.S.A.), and high early strength (Type III cement) concrete. A detailed description of the epoxy injection and the removal and replacement repair techniques, and the replacement materials used are given in Reference 5. Regardless of which technique was used, hairline cracks were not repaired.

BEHAVIOR OF SPECIMENS

Data from three specimens which were considered to be representative of the eight specimens of Reference 5, are reported. Specimens 1 and 2 had Type II Design and a 40,000 lb. (178,000 N) column axial load. Specimen 6 had Type I Design and a zero column axial load. Specimen 1 was subjected to a moderate earthquake loading and Specimens 2 and 6 were subjected to a severe earthquake loading.

Original Tests. For all three specimens, the primary damage occurred in the beam adjacent to the column and most of this damage was attributed to flexural action. Minor crushing occurred in the compression zone of the beam during the first quarter cycle of loading for Specimens 2 and 6. Thin X-type diagonal cracks were observed in the beam-to-column joint with Specimen 6 containing more cracks than Specimens 1 and 2. No cracks were observed in the column and none of the reinforcement fractured.

The beam-tip force vs deflection hysteresis curves obtained during original testing are shown in Figs. 3(a) and 3(b) for Specimens 1 and 2 respectively. From these curves, the stiffness during initial loading, taken as the secant modulus between beam-tip deflection of 0.10 in. (2.5 mm) and 0.40 in. (10.2 mm), was 11.3 k/in. (1980 N/mm) and 10.3 k/in. (1800 N/mm) for Specimens 1 and 2 respectively. The peak-to-peak load at maximum beam-tip deflection and energy dissipated (area enclosed by the hysteresis curves) for each cycle of loading are summarized in Table 1 for Specimens 1 and 2 along with Specimen 6.

Tests of Repaired Specimens. Specimen 1 which experienced the moderate earthquake loading had its beam repaired by the epoxy injection technique. The removal and replacement technique was used to repair the beam for Specimens 2 and 6 which were subjected to the severe earthquake loading. The high early strength concrete used as replacement material in the repair of Specimens 2 and 6 had a compressive strength of 7,000 psi (48 N/mm^2) at the time of retesting the specimens. Briefly described below are the observations made during the retest of the three specimens.

Specimen 1. Epoxy injected cracks in the beam remained closed. Unrepaired cracks opened and new cracks formed in the beam both within and adjacent to the repaired region. The beam-to-column joint sustained no additional cracking during retest and no cracks were observed in the column.

Specimen 2. Damage occurred in the repaired region while the overall behavior was similar to that of the original test. Unrepaired cracks opened and new cracks formed in the beam adjacent to the repaired region. The beam-to-column joint sustained no additional cracking during retest and no cracks were observed in the column.

Specimen 6. The repaired region of the beam sustained only minor cracking and crushing. A considerable amount of cracking occurred in the beam-to-column joint during retest. Slippage of the beam reinforcement anchored in the joint was observed during cyclic beam action. Cracks were observed in the column.

The beam-tip force-deflection curves obtained during retest are shown in Figs. 4(a) and 4(b) for repaired Specimens 1 and 2 respectively. From these curves, the stiffness during initial loading was determined to be 10.3 k/in. (1800 N/mm) and 11.3 k/in. (1980 N/mm) for repaired Specimens 1 and 2 respectively. The peak-to-peak load and energy dissipated for each cycle of loading are summarized in Table 1 for Specimens 1 and 2 along with repaired Specimen 6.

Discussion. A comparison of the peak-to-peak load values between original and repaired Specimens 1 and 2 in Table 1 indicates that the repaired beams were stronger. This increase in strength was due to the use of high strength repair materials and the stressing of the main reinforcement in the beam further into the strain hardening range during retest. Because of the increase in beam strength, the beam-to-column joint is usually stressed to a higher level, thus creating the possibility of damage moving from the beam to the unrepaired joint and column. Damage occurring in the joint can cause a loss of strength as indicated in Table 1 for Specimen 6.

The original and repaired Specimens 1 and 2 dissipated about the same amount of energy as indicated by the values given in Table 1. Repaired Specimen 6 dissipated less energy as compared to its original behavior.

CONCLUSIONS

Based on this investigation and demonstrated by the three specimens reported on herein, the following conclusions are made.

(1) Epoxy injection, and removal and replacement techniques of repair can effectively restore the stiffness, strength and energy dissipation capacity to a reinforced concrete beam when the damage is caused primarily by flexural action.

(2) High early strength (Type III cement) concrete can be used as the replacement material in the removal and replacement repair technique. This material is compatible with the original concrete, it is economical, and it does not require new material handling techniques.

(3) The repaired beams were stronger than the original beams at the same maximum beam deflections. This additional strength was attributed to the strain hardened reinforcement and the higher strength repair materials.

(4) Exterior beam-to-column joints should be re-evaluated before repairs are made to determine if the joint is adequate to resist a future earthquake. Severe damage in the joint can result in the structure behaving poorly during an earthquake.

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TABLE 1 STRENGTH AND ENERGY DISSIPATION VALUES
(1 kip = 4450 N) (1 k-in. = 113 N-m)

Specimen		Cycle												
		1	2	3	4	5	6	7	8	9	10	11	12	13
		Peak-to-Peak Load (kips)												
1	Original	13.6	12.7	12.4	13.9	13.4	13.2	10.6	10.5	10.5				
	Repaired	16.0	15.0	14.6	16.1	15.5	15.1	11.9	11.9	11.9				
2	Original	12.8	12.0	11.7	11.5	12.1	11.9	11.8	11.6	11.3	10.2	10.1	10.0	10.0
	Repaired	16.3	14.8	14.2	13.8	14.5	14.0	13.6	13.3	13.0	11.7	11.7	11.6	11.5
6	Original	15.1	14.2	13.6	13.3	14.9	14.1	13.7	13.2	12.5	9.4	9.0	8.8	8.7
	Repaired	18.5	15.6	12.6	10.4	10.3	7.1	6.0	5.2	4.8	2.0	2.0	2.1	2.1
		Energy Dissipated per Cycle (k-in.)												
1	Original	17.8	13.2	12.1	22.5	21.4	20.4	10.3	9.4	9.4				
	Repaired	17.1	12.5	11.5	23.6	21.4	20.5	10.0	9.3	9.1				
2	Original	29.9	22.0	20.6	20.1	28.4	27.2	26.4	25.4	25.1	17.6	16.5	16.5	16.5
	Repaired	32.7	23.2	21.4	20.4	29.4	28.5	27.8	27.0	26.1	18.5	17.9	17.7	17.5
6	Original	29.7	22.0	19.3	18.4	34.0	31.6	29.3	26.6	24.5	10.5	8.5	8.1	8.0
	Repaired	25.9	14.1	9.8	7.4	15.2	8.6	6.8	5.7	5.1	2.8	1.6	1.6	1.6

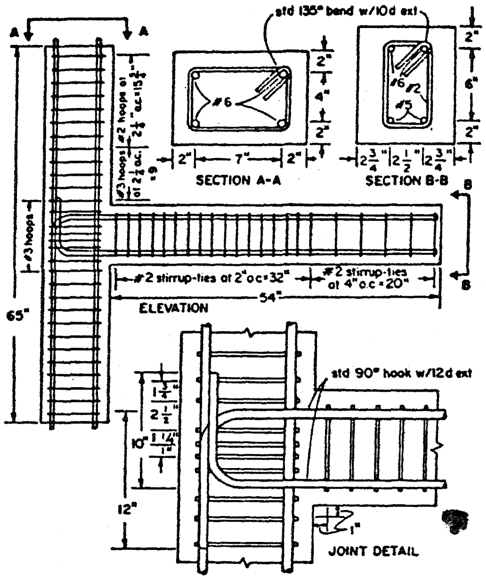
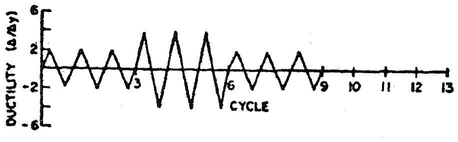
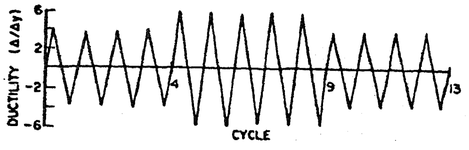


Fig. 1 Type II Design Specimen

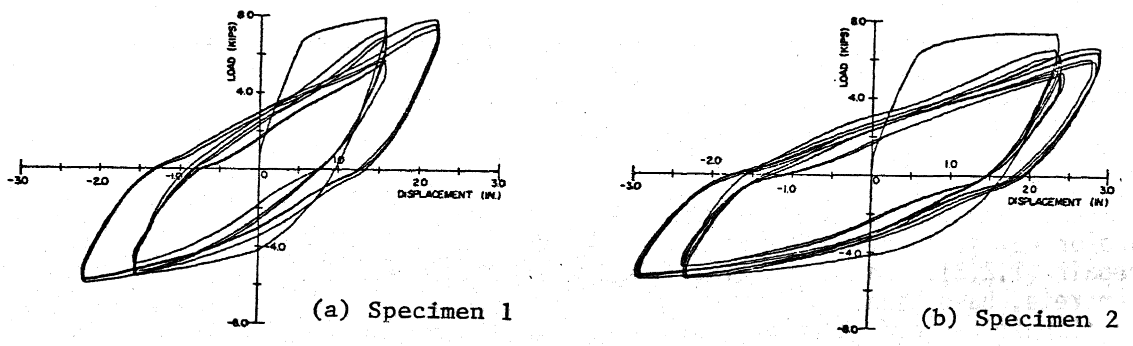


Moderate Loading



Severe Loading

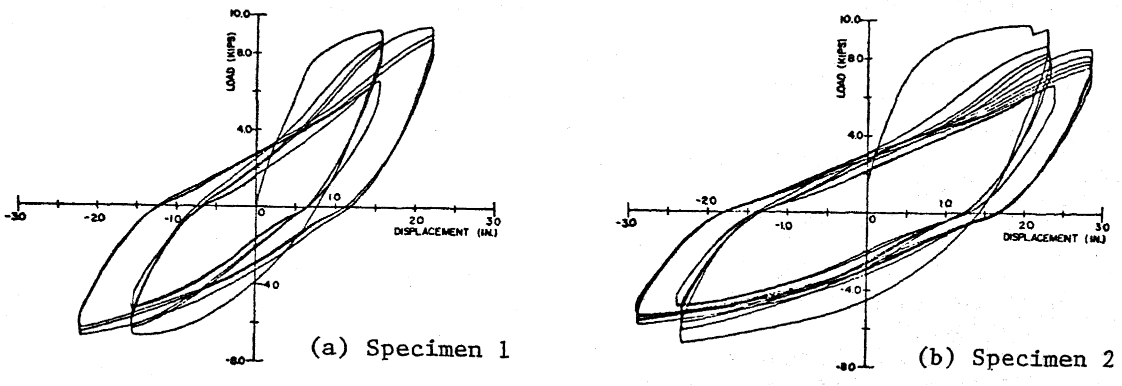
Fig. 2 Displacement Pattern



(a) Specimen 1

(b) Specimen 2

Fig. 3 Original Tests, Force vs Displacement Hysteresis Curves



(a) Specimen 1

(b) Specimen 2

Fig. 4 Tests After Repair, Force vs Displacement Hysteresis Curves