

Quantification of repair effect for RC members using inelastic earthquake response analysis

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ABSTRACT: Earthquake response of repaired reinforced concrete structure was simulated using fiber model extended to include the stress-strain relation of repair material such as epoxy resin and steel plate. Repair effect is then evaluated by ductility, hysteretic energy and damage index. The first part of this paper introduces the method of analysis, the second part shows the verification examples using the test results of repaired reinforced concrete specimen, and the last part describes the application of this method to strengthening of bridge piers with termination of reinforcement at mid-height. Results show that proposed method could give a reasonable estimation for behavior of repaired structures during future earthquakes.

1 INTRODUCTION

Old structures constructed according to old codes sometimes need strengthening for future earthquakes. Or some structures that were rendered nonfunctional by earthquakes could be reused after repair and/or strengthening of the damaged parts. The use of epoxy resin in repaired RC members and steel jackets in strengthening of RC structures have been found to be applicable and effective. However, a reasonable index to judge whether the structure can be retrofitted or has to be rebuild has not been established yet. Repair effect to check whether the retrofitted structures could survive for future earthquakes is not evaluated precisely either. Many experiments have been done for repaired and/or strengthened structures, however, there are a few analytical studies.

In this paper, repair effect for a damaged reinforced concrete structure repaired with grouted epoxy resin or steel jackets was quantified using inelastic response analysis using the force-displacement relationship according to extended fiber model.

2 FIBER MODEL INCLUDING REPAIR MATERIAL

An ordinal fiber modeling technique for RC structure can be easily extended to include repair material, such as epoxy resin and steel jacket. Fig. 1(a) shows assumed stress-strain relation of confined and unconfined concrete (Park and Pauley 1975), and Fig. 1(b) shows that of reinforcing bars. Using the ordinal bilinear relation, the maximum strength was added to describe the ultimate behavior. Steel jacket was assumed to contribute as additional reinforcing bar to

moment-curvature relation of each section. As the grouted epoxy resin usually locates randomly in each section, precise location is unpredictable from outside. Therefore, epoxy resin was defined as area ratio of its existence in this study. Epoxy resin was assumed to behave partially linear like Fig. 1(c). Young's modulus of epoxy resin is almost 10% of concrete, however, it is stronger than concrete in tension.

Using ordinal fiber modeling procedure, the moment-curvature relationship could be calculated for each section of a repaired or strengthened structure. Area of epoxy resin were calculated from a given ratio and area of concrete in each fiber. The moment for a given curvature was calculated to satisfy a given axial load. Four points were determined to approximate the moment-curvature relationship in 4 lines. They are the crack point when crack occurs in concrete, the yield point when yield occurs in reinforcing bars, the maximum point when the unconfined concrete crashes and the ultimate point when the confined concrete crashes or when the resistant force decreases until 80% of the maximum value.

Then the force-displacement relationship was derived according to the moment-curvature relationship of each section. Four points like the moment-curvature relationships were determined to approximate the relationship in 4 lines.

Using the force-displacement relationship as a skeleton curve, an inelastic hysteretic model was assumed for earthquake response analysis. One of the suitable hysteretic models for RC structures, 3 parameter model (Park, Reinhorn and Kunnath 1987) was used in this study.

Lastly, damage of the repaired or strengthened structure during a future earthquake was evaluated from the inelastic response of the estimated model.

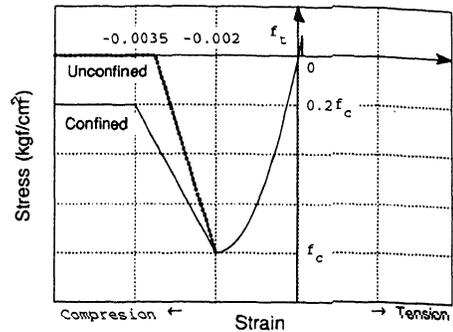
Three indices were used to evaluate quantitative repair effect; they were ductility, hysteretic energy and the damage index (Park and Ang 1984).

3 VERIFICATION EXAMPLES USING TEST RESULTS

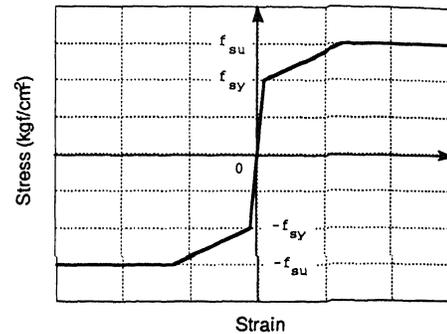
Seismic behavior of repaired and/or strengthened RC members by epoxy resin and steel plates was tested using the hybrid (pseudo-dynamic) testing procedure (Izuno, Yamada and Iemura 1988). Specimens had the dimensions 100×150×1900 mm and doubly reinforced by deformed bars. Concrete was confined by stirrups every 70 mm. The NS component of the 1940 El Centro record was used as input earthquake motions to simple supported specimens. First, the specimens were pseudo-dynamically tested to get the damaged specimens. Second, some of them were repaired by grouted epoxy resin or covered with steel jackets. Then, the repaired specimens as well as the unrepaired damaged specimens were tested again using the same input motions to see the repair effect.

In this chapter, these experimental results were numerically simulated using the proposed analytical procedure for verification. Area of epoxy resin in each section was determined from the views of the repaired specimen cut into pieces after the loading tests shown in Fig. 2.

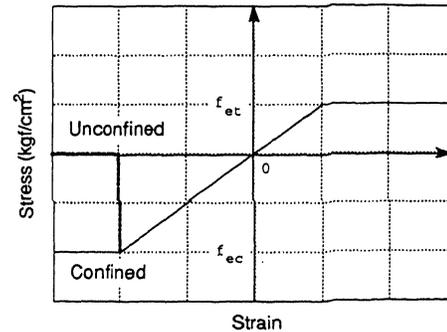
Fig. 3 shows one of the results comparing the experimental and analytical hysteretic loops for before and after repair work by epoxy resin. Analytically obtained hysteretic loops successfully simulated the experimental results. Repaired specimens became stiffer than originals because the grouted epoxy resin has higher tensile strength than concrete. The evaluated maximum responses showed similar values as the experiments. Fig. 4 compares analytically obtained hysteretic energy with experimentally obtained hysteretic energy and Fig. 5 compares damage indices of them. The dashed line means the same values as the experiments were obtained from the analysis. All dots scattered around this line, which shows the estimated hysteretic energy and damage index were in good agree with the experimental results.



(a) Concrete



(b) Steel



(c) Epoxy resin

Fig. 1 Assumed stress-strain relationship

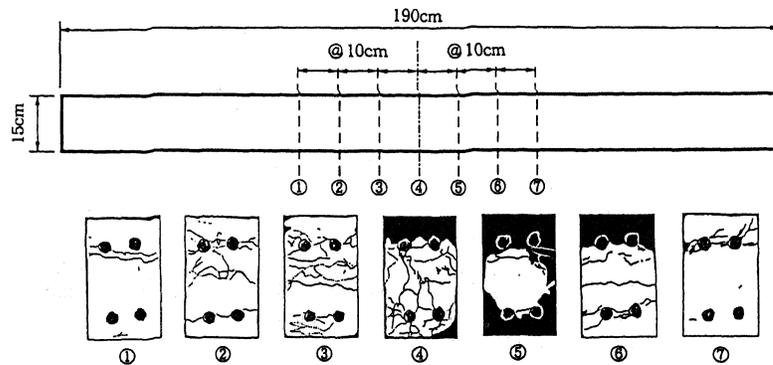
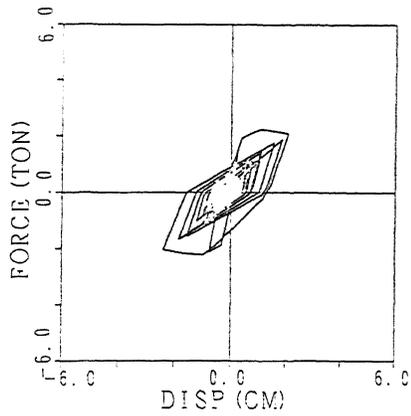
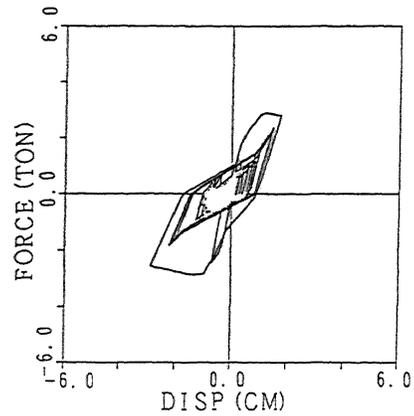


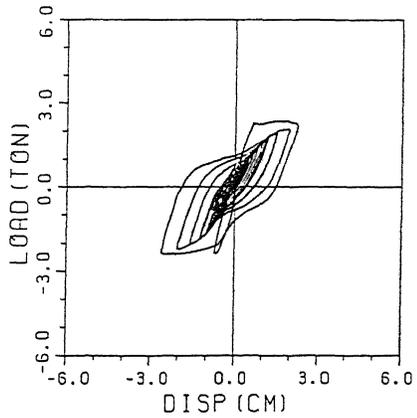
Fig. 2 Cross-sectional views of repaired specimen.



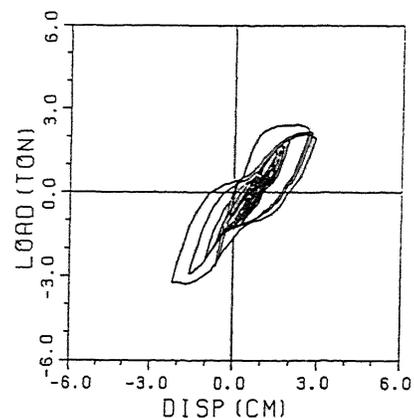
(a) Analytical result for original specimen



(b) Analytical result for repaired specimen



(c) Experimental result for original specimen



(d) Experimental result for repaired specimen

Fig. 3 Comparison of hysteretic loops between analysis and experiment for original and repaired specimens.

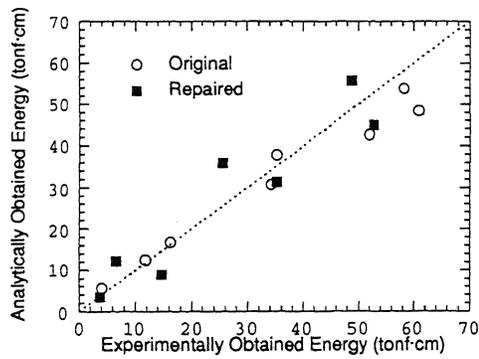


Fig. 4 Comparison between analytical and experimental absorbed hysteretic energy during experiments.

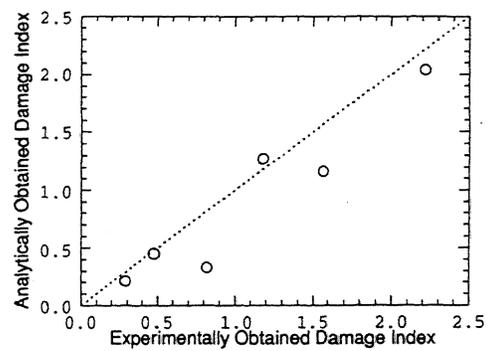


Fig. 5 Comparison of damage index obtained by analysis and experiment.

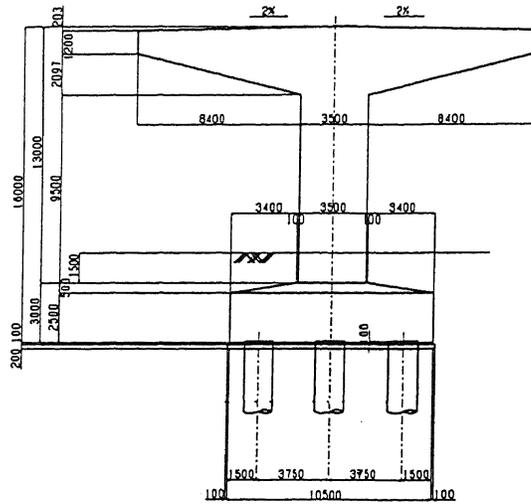


Fig. 6 T-shape bridge pier model used for the simulations.

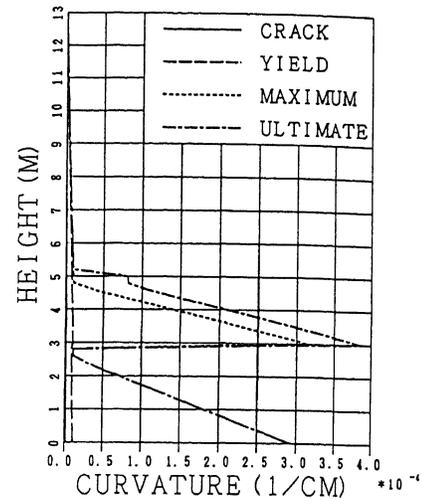
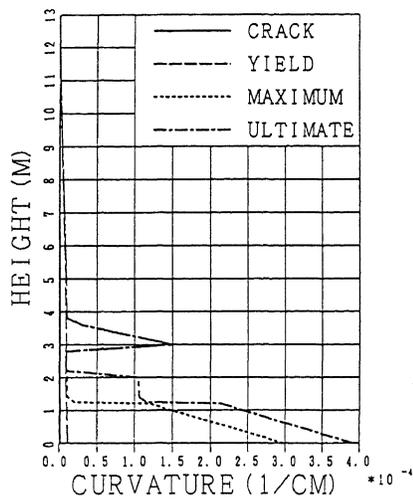
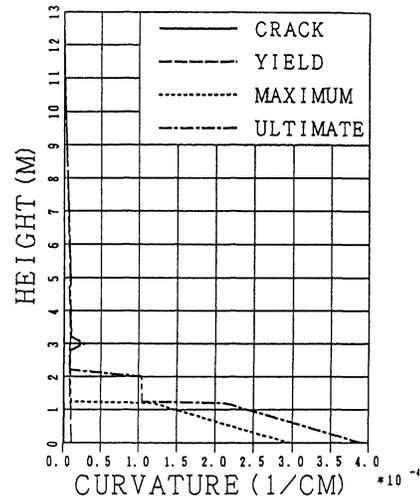


Fig. 7 (a) Original



(b) Strengthened with 1 mm steel jackets



(c) Strengthened with 2 mm steel jackets

Fig. 7 Moment-curvature relationships at each section of the bridge piers.

4 APPLICATION TO BRIDGE PIER WITH TERMINATION OF REINFORCEMENT

Some old bridge piers constructed under old codes have insufficient anchorage length and termination of their main reinforcement. These structures might suffer in a brittle shear failure and need strengthening for future earthquakes. Use of steel jackets has been found applicable to prevent brittle failure by experiments (Kawashima, Unjoh and Iida 1990). The proposed analytical method was then applied to evaluate effect of strengthening for the bridge piers with termination

of reinforcement at mid-height.

First, T-shape bridge pier of 13 m high shown in Fig. 6 was modeled to have the terminated main reinforcement. Then, the pier was divided into 50 sections and the moment-curvature relationship of each section was calculated as shown in Fig. 7(a). This figure shows crash may occur at the height of 3 m where the reinforcement terminated. Figs. 7(b) and 7(c) show the moment-curvature relationships for the strengthened bridge pier with steel jackets. 2 types of steel jackets were considered: one is 1 mm in thick (Fig. 7-b) and the other is 2 mm in thick (Fig. 7-c). Thicker steel jackets of course gave smaller curvature at where the

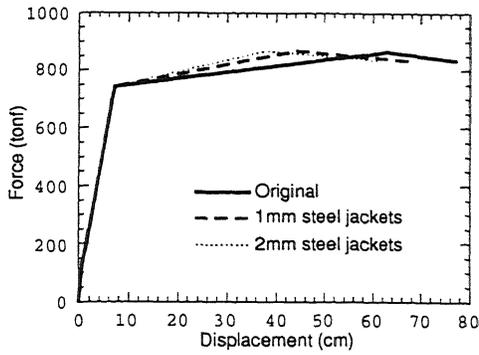


Fig. 8 Estimated force-displacement relationships of original and strengthened bridge pier.

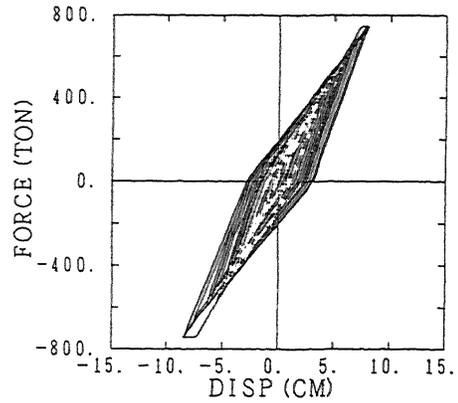
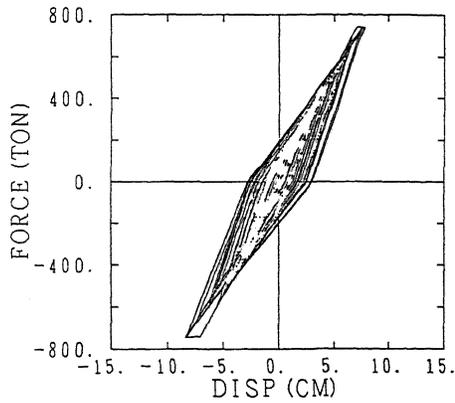
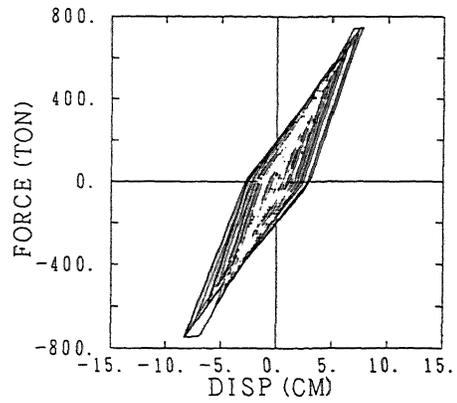


Fig. 9(a) Original



(b) Strengthened with 1 mm steel jackets



(c) Strengthened with 2 mm steel jackets

Fig. 9 Simulated hysteretic loops of original and strengthened bridge piers.

Table 1 Hysteretic energy and damage indices for original and strengthened bridge pier.

	Energy (tonf-cm)	Index
Original	59487	0.631
Strengthened (1 mm steel)	58463	0.706
Strengthened (2 mm steel)	57825	0.769

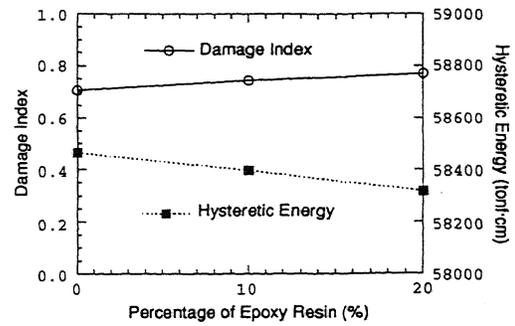


Fig. 10 Damage index and hysteretic energy according to amount of epoxy resin.

reinforcement had been terminated. The critical place moved from the mid-height to the bottom, which means the strengthening had been done effectively.

The force-deformation relationship for the pier top is shown in Fig. 8. The solid line is for the original pier, the dashed line is for the strengthened pier with steel jackets of 1 mm in thick and the dotted line is for the strengthened pier with steel jackets of 2 mm in thick. The pier bottom where no strengthening had been done became the critical place for the strengthened piers. Therefore, the ductility decreased because of the stiffer structure with the same ultimate strength as the original.

Then the inelastic earthquake response analysis was done using the Type 3 earthquake which was recommended in the Japanese seismic code (Japan Road Association 1990). The hysteretic loops of them for the 3 times larger acceleration as the design code are shown in Fig. 9. Strengthened structure showed more stable loops than the originals. The absorbed hysteretic energy and the damage index of the original and strengthened bridge piers are shown in Table 1. Though the absorbed energy became smaller for the strengthened structures, they showed larger values for the damage index because of their low ductility. Furthermore, steel jackets of 2 mm in thick showed larger damage index than that of 1 mm for the same reason. Thinner jackets are recommended if they are thick enough to change the critical place to the pier bottom. Strengthening at the pier bottom may also be needed to satisfy enough ductility.

Next, effect of epoxy resin grouted during the strengthening was studied. 10% and 20% of each strengthened section was assumed to be epoxy resin instead of concrete. These simulation may represent the case that the pier already had some cracks to be repaired at mid-height, or steel jackets were bonded to the pier with epoxy resin. The thickness of the steel jackets was selected to 1 mm. Fig. 10 shows the damage index and absorbed hysteretic energy for each case. As epoxy resin increases, the damage index also increases and the hysteretic energy decreases. However, the difference is smaller than 1% for the hysteretic energy and 9% for the damage index. Considering the large ratio of 20% for epoxy resin in each section, the effect of epoxy resin to seismic behavior of strengthened bridge pier is small enough to neglect it.

5 CONCLUSIONS

Main conclusions obtained are as follows:

1. Inelastic earthquake response analysis was proposed using the skeleton curves obtained from the fiber model extended to include repair materials.
2. Analytically obtained hysteretic loops, absorbed hysteretic energy and damage indices were in good agreeing the experimental results.
3. Aseismic behavior of the strengthened bridge pier was simulated, and effectiveness of thinner steel jackets was verified. Existence of epoxy resin in

strengthened bridge pier with steel jackets was negligible.

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