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ABSTRACT: In continuous steel-reinforced concrete structures, redistribution of bending moments is permitted during design if certain ductility requirements are met. Ductility and rotation capacity allow some portion of 'undesirable' bending moment peaks (usually negative moments over supports) to be shed to more 'desirable' (usually positive moment) locations, so that efficient use is made of the chosen cross-section. When an existing continuous concrete structure is strengthened with CFRP laminates, its ductility and rotation capacity may be restricted due to the linear-elastic response of the CFRP, and the increase in stiffness due to the presence of the CFRP can prevent moment redistribution once steel has yielded. This research project investigates the issue of redistribution in continuous CFRP-strengthened concrete slabs where alternatively, the cross section with the lower and higher peak moment have been strengthened. Experimental results are presented.

1 INTRODUCTION

Most reinforced concrete (RC) structures are designed for internal forces found by elastic analysis from the factored loads. In a continuous beam, for example, the critical sections are designed with the knowledge that the steel will be well into the yield range and the concrete stress distribution nonlinear before final collapse. Clearly this is a very conservative approach since it is well known that a continuous beam or frame normally does not fail when the ultimate moment capacity is reached at just one critical section. Moment redistribution will occur and provide additional capacity to the member beyond the point when yield stresses are reached at that particular critical location for the structure (i.e. in continuous beams or slabs, first in the negative moment regions, usually over the supports). A plastic hinge will form at that section, permitting large rotation to occur at essentially constant resisting moment (i.e. plastic moment), thus transferring moment to other locations along the span where the limiting resistance has not yet been reached. Research has shown that the degree of rotation and consequently of the redistribution that can occur in a structural member, is function of the ratios of tension (?) and compression (?`) steel reinforcement. The current ACI 318-02 building code (Section 8.4) states that the level of redistribution which may be assumed in a RC structure is 1000e_t %, up to maximum of 20%, where e_t is the level of strain in the tension reinforcement, which must be at least 0.0075.

Nowadays the use of fiber-reinforced polymer (FRP) composites has become a widely accepted solution for strengthening of existing reinforced concrete structures that are deficient for their current use in both the positive and negative moment regions. FRP strengthening schemes offer rapid, cost-effective and durable (at least according to today's research) solutions for the retrofit of concrete structures, with only one main drawback: the reduction in ductility that is experienced in the strengthened member and also the fact that the plastic range, that previously the section was encountering, is no longer a flat plateau due to the presence of the FRP.

This loss in ductility has led various design guidelines on FRP external strengthening, ACI 440-2R-02 that does not even address the issue, to prohibit or discourage the application of moment redistribution from strengthened cross sections (FIB Bulletin 14, Section 3.1.2), leading to onerous conditions for such strengthening particularly when the original design was based on moment redistribution. The reason for such attitude is based on the relatively unpredictable ultimate failure that a section strengthened with FRP composites systems can exhibit (i.e. peeling, delamination).

In this experimental program, the authors had the unique possibility of studying the behavior of an existing reinforced concrete deck under a loading condition that would demonstrate moment redistribution. By studying slabs with and without FRP strengthening, the question as to whether moment redistribution could be applied to the design of indeterminate strengthened members could be addressed.

2 EXPERIMENTAL PROGRAM

2.1 Building characteristics

The parking garage used for the tests was located in St. Louis, Missouri, and was scheduled for demolition in July 2002. The structure was constructed in the 1950s, consisting of a concrete-encased steel frame, supporting a one-way RC floor system (see Figure 1a and Figure 1b). Due to the old age of the structure, no construction or maintenance records were available from the owner. For this reason, a field investigation to evaluate geometry and material properties of the structure was carried out. Based on the survey, it was determined that the typical RC slab was 12.7 cm thick (in the area dedicated to these tests), 512 cm long and 255 cm wide. The main reinforcement consisted of one layer of 12 mm diameter steel bars spaced 30 cm center-to-center at mid span, and similar bars spaced 30 cm center-to-center at the support in the E-W direction. In the N-S direction, 12 mm steel bars, spaced 45.7 cm center-to-center were used as temperature and shrinkage reinforcement. All steel bars tested showed an average yield strength of f_y = 415N/mm^2. Concrete properties were evaluated using six cores taken from different locations in the slab prior to testing and an average concrete cylinder strength of f'_c = 31 N/mm^2 was found.

(a) Top Slab View (b) Bottom Slab View
Figure 1 – Parking Garage

2.2 Test matrix

A total of four continuous beams specimens were available to be tested within the deck of the garage, by saw-cutting the deck (full depth) along carefully defined lines.

Continuous beams (R1 to R4) were tested to failure to investigate how the CFRP-strengthening affects the redistribution of moments in continuous structural members (see Figure 2).

All tested beams were of the same dimensions of 60.9 cm × 1173.5 cm × 12.7 cm (the length refers to the total length of the longitudinal cuts since the top end is not cut and continuity not interrupted). Each beam was cut in a way to have the same amount of reinforcement, that consisted of 2 steel bars as bottom reinforcement at mid span extending from support to support and the same amount as top reinforcement in the negative moment region, spanning only one third of the span (89 cm in each direction).

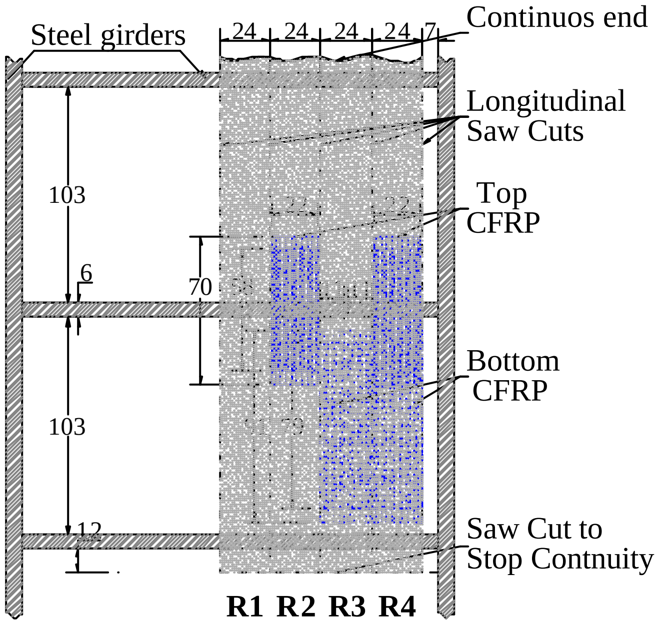
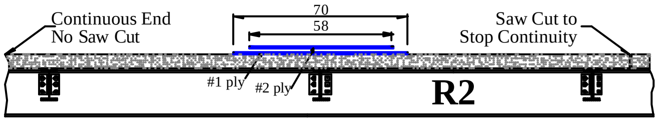
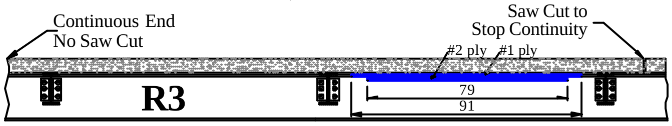


Figure 2 – Geometry and Detail of Strengthening (US units; 1 in = 2.54 cm)

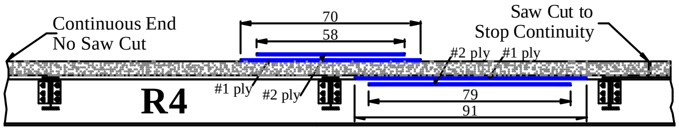
The concrete cover for each specimen was measured by means of a rebar locator. Specimen R1 served as the control specimen with no strengthening, while the other three beams (R2 – R4) were strengthened using CFRP laminates applied by a wet lay-up. The sheets had ultimate strength of 3790 MPa, tensile elastic modulus of 228,000 MPa, and each had a nominal thickness of 0.16 mm (Wabo®MBrace Composite Strengthening System Design Guide, 2002). The concrete surface was first sandblasted to assure good bond. The external reinforcement was applied in three ways, namely strengthening only in the negative moment region over the central support (beam R2), only in the positive moment region (beam R3) and on both negative and positive moment regions (beam R4). The strengthening was designed in order to achieve a high degree of strengthening in order to study the different degrees of redistribution that could take place in a continuous beam where the strengthened section is the lowest solicited section (R2) and the most solicited one (R3). For this reason, beam R2 (see Figure 3a) contained one strip of two plies, 55.9 cm wide and of varying length (177.8 cm and 147.3 cm) in order to reduce the stress concentration effect at the ends of the laminate. Beam R3 (see Figure 3b) contained two strips of two plies, 27.9 cm wide and of varying length (231.1 cm and 200.6 cm) for the same reason as for beam R2. Beam R4 had the same amount of strengthening as R2 in the negative moment region and of R3 in the positive moment region respectively (see Figure 3c).



a) Beam R2

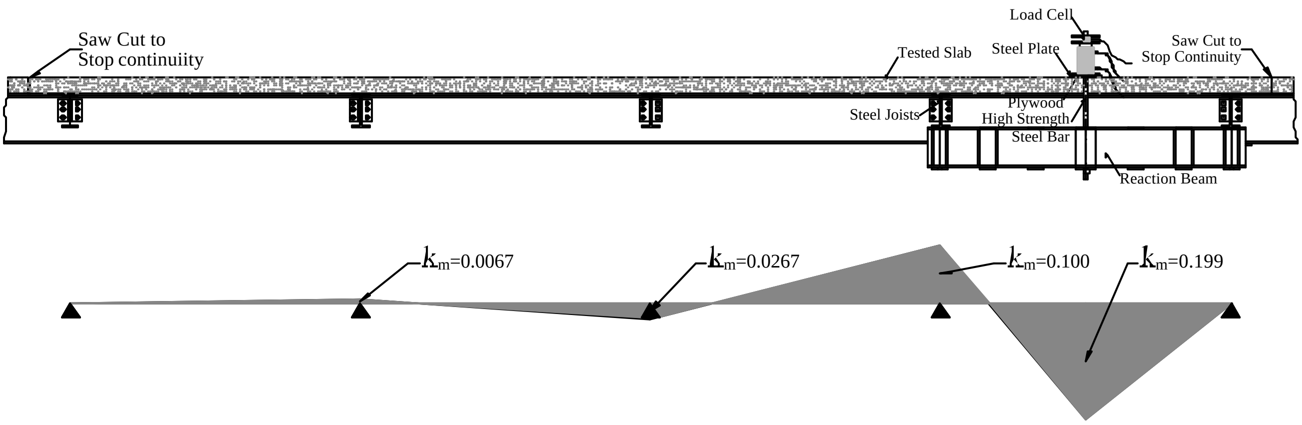


b) Beam R3



c) Beam R4

Figure 3 – Sequence of laminate application in Beams R_2 – R_4 (US units; 1 in = 2.54 cm)



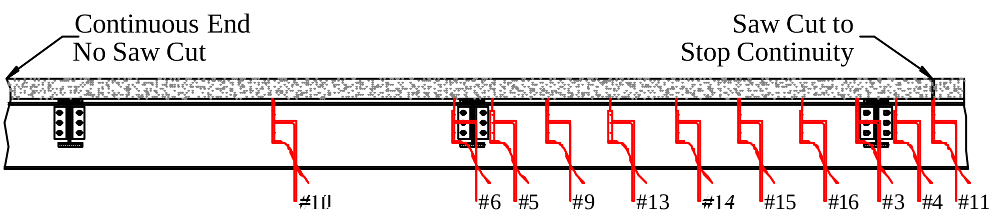
(a) Section View and Elastic Distribution of Moments ($\boldsymbol{k}_{\mathbf{m}}$ = elastic coefficient for redistributing moments)

(b) Bottom View

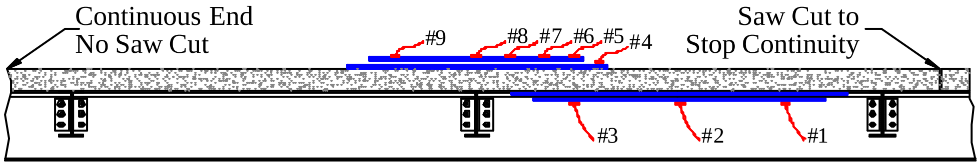
(c) Top View (R1)

(c) Top View (R2)

Figure 4 – Test Set up



(a) LVDTs



(b) Strain gages on CFRP

Figure 5 – Instrumentation Location

2.3 Test Set-up

A close loop scheme was chosen for all tests, as shown in Figure 4a. The beams were loaded at one point at mid-span in order to create the most convenient situation for studying moment redistribution. In this configuration the positive moment is larger than the negative one (see elastic distribution of moments in Figure 4a). This may not be the most common case for continuous members, nevertheless it allows the study of redistribution.

The load was applied in cycles by a hydraulic jack connected to a hydraulic hand-pump, and measured using a 450 kN load cell on top of the jack. Displacements were measured at eleven significant points using linear variable differential transformers (LVDTs), as shown in Figure 5a. The strains in the external reinforcement were measured using strain gages at the locations indicated in Figure 5b. It was not possible to apply any strain gauges to the concrete due to bad weather conditions during the time when the tests were performed.

3 TEST RESULTS AND DISCUSSION

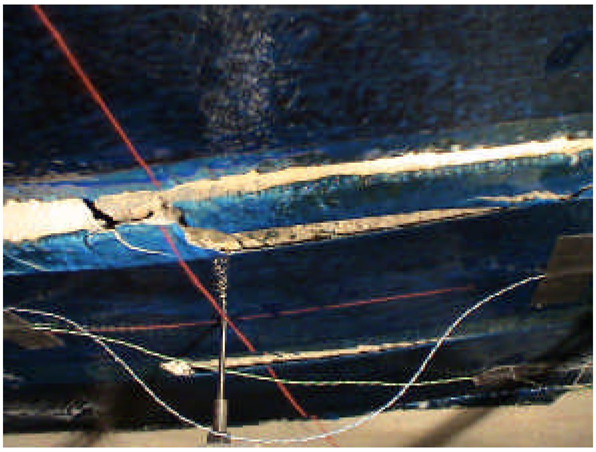
The failure mode of the beams can be summarized as follows: R1 and R2 failed in flexure, while R3 and R4 failed in shear near the central support. No FRP delamination was experienced in any of the specimens. Table 1 reports the test results.

Table 1. Test results R1-R4

Beam	Failure load (kN)	Max deflection at DCVDT#14 (cm)(1)	Max deflection at DCVDT#10 (cm)(1)	Failure mode
R1	39.5	+ 5.29	- 1.62	Flexure
R2	38.2	+ 6.14	- 2.77	Flexure
R3	92.5	+ 4.65	- 0.39	Shear
R4	97.4	+ 4.73	- 1.75	Shear

(1)Positive values indicate downward deflection while negative values indicate upward deflection

a) Flexural Failure (exposure of flexural reinforcement)



(b) Shear Failure

Figure 6– Failure modes for R1-R3

The behaviors of beams R1 and R2 were very similar. Both experienced the same ultimate load, although beam R2, strengthened in the negative moment region, showed marginally more ductility before failing for concrete crushing. Looking at Figure 7 and Figure 8 that report respectively the load-deflection curves at DCVDT #14 and 10, one can deduce important information regarding the ductile behavior of such specimens. Considering that both of them started from a cracked stage, one notices a considerable change in stiffness of both members, once a load of 20 kN was reached and, from this point, yielding of steel initiated. The only difference between the two curves is their ultimate displacement: looking at the loaded span (DCVDT #14), beam R1 experienced higher displacement (~1cm overall) with respect to beam R2, while in the unloaded span the exact opposite occurred, namely beam R2 had an ultimate uplift of 2.8 cm while beam R1 only deflected by 1.6 cm. Looking at the strain profiles, one can notice that the strain in the FRP at position #8 (see Figure 9) is very low (4.3 % of ultimate). This was probably caused by the fact that the internal reinforcement only spanned one third of the span (exactly the same length of the CFRP laminate), causing the cracking of the concrete and consequently preventing full engaging of the FRP.

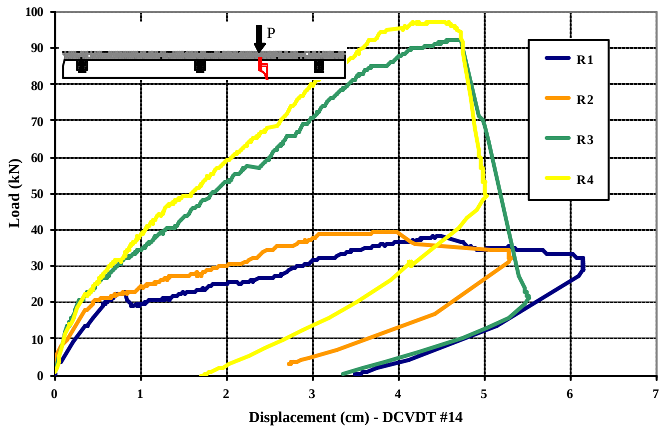


Figure 7 – Load vs Deflection curves at DCDVT#14

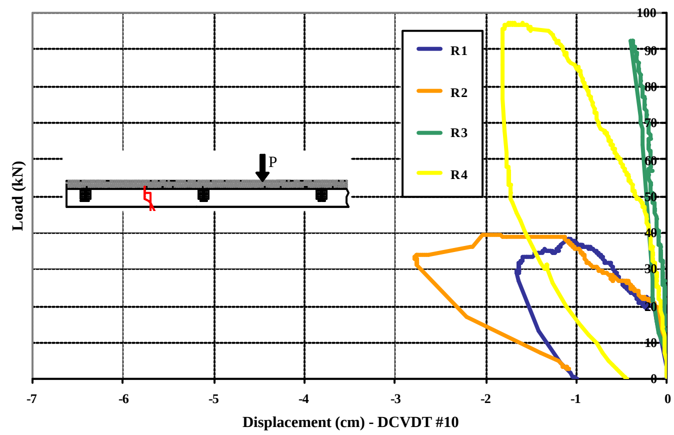


Figure 8 – Load vs Deflection curves at DCDVT#10

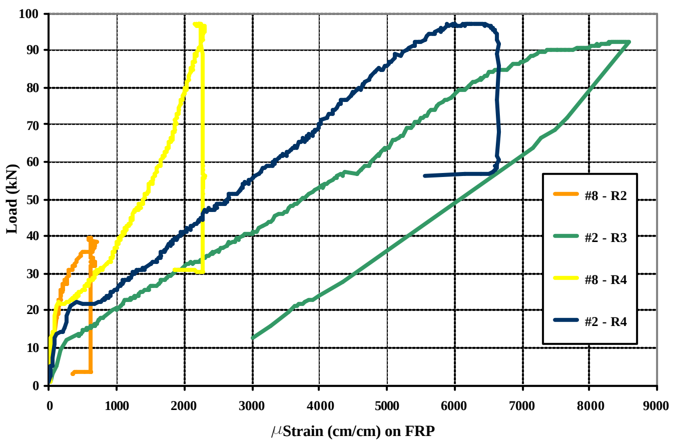


Figure 9 – Load vs Strain curves

Considering now beam R3, one can notice a different behavior. The strengthening in the positive moment region made the beam much stiffer and stronger with respect to either R1 or R2 (see Figure 7 and Figure 8). If shear would not have been the ultimate cause of failure, it is possible that even higher capacity. The strain profile at location #8 shows that the strain in the FRP is over 50% of the ultimate elongation (see Figure 9). Analyzing displacement of LVDT #10 (see Figure 8), one can notice that the cross section has rotated negligibly with respect to beams R1 or R2.

Beam R4 experienced a behavior similar to that of R3, a similar ultimate failure, but with a more ductile response, as can be noticed by the larger displacement in Figure 8. From the strain profiles, the FRP at location #2 (see Figure 9) experienced a strain close to 41% its ultimate value and at location #8 close to 14%, more than three times higher than that of beam R2.

4 ANALISIS

A simplified method for the non-linear analysis of RC structures has been used to evaluate the theoretical ultimate load and distribution of moments at the ultimate conditions. It is called method of the "concentrated distortions" as it concentrates the effect of diffuse concrete cracking into single properly computed distortions (La Tegola et Al. 2001). It must be noted that the effect of shear is not taken into account.

Table 2 summarizes the theoretical ultimate load determined using a step-by-step procedure. The load has been increased until it reaches the ultimate moment in one of the two most critical sections.

It can be observed that the theoretical ultimate loads, computed using the non linear approach, are in good agreement with the experimental ones. The experimental values are generally higher than the theoretical predictions as a consequence of the use of the ACI recommendations.

Table 2. Analytical Results

Beam	Theoretical Elastic Failure load [kN][(1)]	Theoretical Nonlinear Failure load [kN][(2)]	Experimental Failure load [kN]
R1	15.8	20.5	39.5
R2	27.8	37.5	38.2
R3	75.2	76.6	92.5
R4	85.3	86.1	97.4

[(1)]Without considering moment redistribution
[(2)]Considering moment redistribution

5 CONCLUSIONS

The unique possibility of testing an existing reinforced concrete deck has made possible the study of a combined solution with and without FRP strengthening, with the objective of better understanding if the concept of moment redistribution may still be applied to the design of indeterminate strengthened members.

The analytical results are in good agreement with the experimental ones and allow understanding of the mechanics of moment redistribution for both strengthened and un-strengthened cross sections. A future paper, will focus entirely on the analytical work, in order to further investigate this topic and have a deeper and more clear understanding of the problem.

Further tests should be conducted in a laboratory environment in order to validate allowable percentages of redistribution, monitoring steel reinforcement in order to check continuously the level of strain.

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