Structural Assessment of FRP-Strengthened Structural Members Using Cyclic Load Testing

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When a concrete building is being renovated or upgraded, the load carrying capacity of its structural system must be established. If the original structural drawings are unavailable, an in-situ assessment of the existing structure may be required to establish member geometry and determine material strengths as well as the size, layout and spacing of existing reinforcement. Even when all these parameters have been established, there could be many uncertainties due to unknown structural conditions such as the strength of deteriorated or previously repaired members. Fortunately, load testing can be used to verify that a structure can safely support the design loads and to verify the capacity of repaired or strengthened members.

Known as the Cyclic Load Test (CLT) method, this method of load testing was recently adopted by ACI Committee 437 and involves several cycles of loading and unloading that are used to obtain information on the strength and performance of a structure. CLT is typically conducted using hydraulic jacks - improving load test safety and reducing the risk of overloading or damaging the structure. Structural adequacy is then verified by examining the linearity of the measured deflection response and magnitude of permanent deformation after loading the member to near its strength.

Three case studies for structural assessment through load testing are presented in this paper. In the first case study, load testing was used to determine the load carrying capacity of a library slab, verify the cause of existing cracks, and confirm the reliability of the analytical models that were later used to design the fiber reinforced polymer (FRP) strengthening at various locations. In the second case study, load tests were used to determine the capacity of existing concrete joists and their governing failure mode. They also confirmed the performance of the strengthening solutions that included externally bonded FRP and bonded concrete overlay. The third case study describes the upgrade and full-scale load testing of a parking garage. Testing was performed to evaluate the previous repair that consisted of a reinforced gunite beams bonded to the underside of the garage slab. FRP reinforcement was considered for flexural strengthening of the garage slab and the performance of FRP strengthening was verified by full-scale in-situ load testing. The load test also demonstrated poor performance of the previously repair while the performance FRP strengthened slab was satisfactory.

For all described cases, externally bonded carbon FRP (CFRP) reinforcement provided a cost-effective strengthening solution. The existing structural components of the structures were verified to have adequate capacity to carry the design service loads without the contribution of the CFRP, so it was designed only to supplement the existing capacity of the structural element. Similarly, the strengthened structural components were verified to have adequate fire resistance without the contribution of the FRP. As such, no additional fire protection was needed for the FRP and only an intumescent top coat was applied to provide the required Smoke Density and Flame Spread ratings per building code requirement.

INTRODUCTION

In situ load testing has been specified in the ACI building code since 1920. Chapter 20 of the ACI 318-08 Building Code specifies the procedure, load intensity and loading criteria for an in-situ load test (ACI 318, 2008). According to ACI 318, during a load test, the test load must be applied in at least four increments, and a set of response measurements (mainly deflection) must be taken after the total test load has been applied, and after at least 24 hours of loading. A final set of response measurements are also required 24 hours after the test load has been removed, so the total duration of the load test can exceed 72 hours. When multiple load tests are required on a structure to verify capacity of various elements, the use of the ACI 318 load test procedure can lead to long test duration, and probably additional cost.

In the past 10 years, researchers and practitioners in the Unite States have been evaluating an alternative load test method that examines the structure in several loading cycles. Known as the Cyclic Load Test (CLT) method, this procedure requires less time to perform than the ACI 318 procedure and since 2003 has been incorporated into ACI 437R (ACI 437R). In the CLT method, six cycles of loading and unloading are typically used to obtain information on the performance of the structure. Structural adequacy is verified by examining the linearity of the measured deflection response and magnitude of permanent deformation after loading the member to near its required strength. CLT investigations are typically conducted using hydraulic jacks that allow for the test member to be quickly unloaded at any sign of distress. ACI 437.1R describes several methods for providing reactions to the hydraulic system based on the characteristics of the member to be tested and the overall site conditions.

In the following case studies, load tests were used to verify analytical behavior and capacities of existing structural components, and improvement after strengthening with concrete enlargement and externally bonded fiber reinforced polymer (FRP) reinforcement.

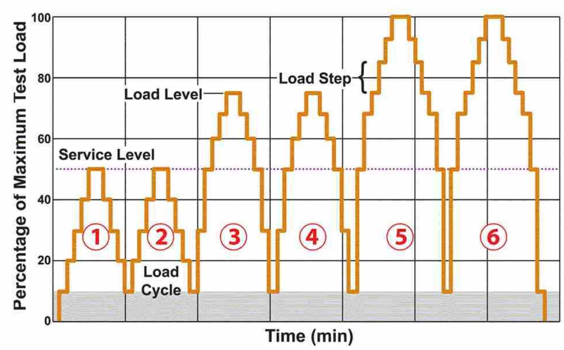


Figure 1. Typical Load Cycles

CASE STUDIES

In the following sections, three case studies are used to describe how the CLT method was used to examine the load carrying capacity of existing structures and to verify the performance of proposed strengthening solutions. In each investigation, the industry recommendations for assessing the existing condition of the structure were followed. These included studying existing drawings, reports, and calculations as well as verifying the information using on-site inspection (ACI 437R-03, ACI 364.1R-00, SEI-ASCE 11-99). Per ACI 437.1R, acceptance criteria including deflection repeatability, permanency, and deviation from linearity were used to examine the performance during and after the load test.

Case Study 1: National Institute of Health Library

Level B of Building 38 at the National Institute of Health (NIH) in Bethesda, Maryland houses the National Library of Medicine. This level was undergoing renovation to accommodate a new high density filing system that required the elevated slab to be upgraded from its original live load capacity of 6.0 kN/m^2 to a new live load demand of 9.6 kN/m^2. Flexural cracks observed on the top side of the slab prompted NIH officials to request load tests to verify the capacity of the existing floor slab.

Level B comprises a 265 mm thick concrete flat plate reinforced with 275 MPa deformed steel bars.. A typical slab area is supported by 610 x 860 mm reinforced concrete columns, typically on a 6.4 m x 6.4 m grid. The CLT was conducted on a 3.2-m wide column strip located along grid line 12 to evaluate the current bending capacity at mid-span and at the support (Fig. 2).

The test load magnitude (TLM) was determined using ACI 437.1R-07, as follows:

$$TLM_{437} = Dw + 1.1\ Ds + 1.6L \quad (1)$$

Where Dw is the slab self-weight, Ds is the superimposed dead load, and L is the specified live load. Using Equation (1), the uniformly distributed TLM was calculated to be 17.1 kN/m^2. Table 1 summarizes the existing moment and punching shear capacities, ϕMn and ϕVn, and the factored bending and punching shear demands, Mu and Vu, for the column strip under investigation based on the as-designed (original) conditions. Per Table 1, the slab was found to have adequate shear capacity.

Two loading configurations were used to test the slab. Scheme 1 was used to reproduce the negative bending at column H12 and Scheme 2 was used to reproduce the positive bending at mid-span between G12 and H12, as shown on Fig. 2. For the Scheme 2 load test, it was not possible to apply the load symmetrically with respect to column H12 due to the presence of piping at those locations.

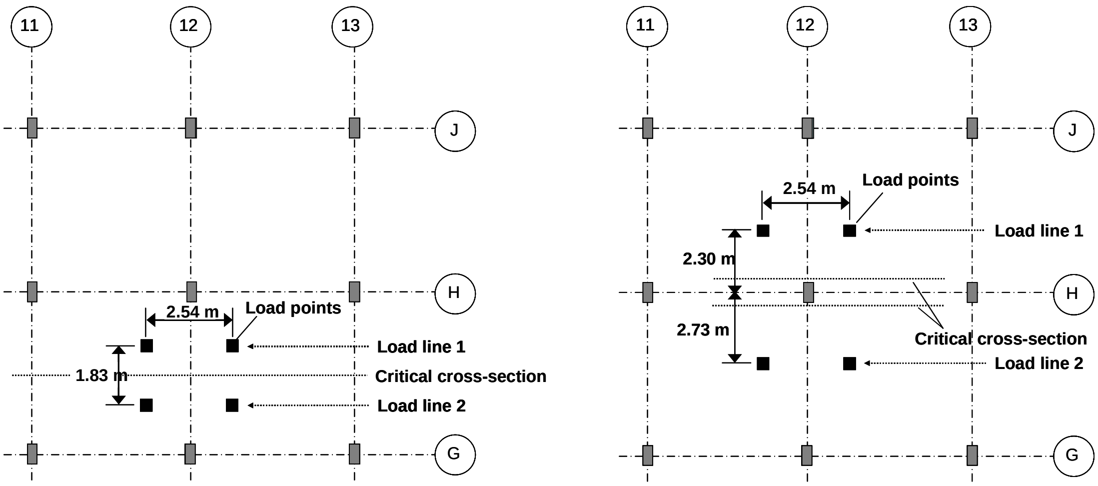
The push down load-test that was used in this project is schematically shown in Fig. 3. Reaction to the hydraulic jacks was accomplished by utilizing the deadweight of the floors above. The slab was tested using six loading-unloading cycles for each test configuration - three loading levels with two cycles for each load level. The maximum load TLM_{437} was achieved in Cycles 5 and 6.

Results of the Scheme 1 test indicate a fairly linear behavior of the structure for positive moments. The acceptance criteria in terms of deflection repeatability, permanency, and deviation from linearity were within the limits prescribed by ACI 437. No new cracks were observed during the load test; the effect of the applied loads mainly increased the width of the existing cracks, which returned to the original width at the conclusion of the load test. As such, the performance of the slab was deemed satisfactory.

Analytical predictions were determined by developing a two-dimensional finite element model, using commercial software [SAP, 2000]. Cracking of the slab during the test was introduced into the model by reducing the stiffness of the slab to the effective stiffness as defined in ACI 318. Figure 4 compares the analytical predictions with the experimental results for the Scheme 2 test. This figure shows that deflections measured in the first two cycles matched that of the uncracked slab, while the measured deflections in the last two cycles are closer to deflections predicted based on a cracked slab condition. A transitional behavior can be observed on the third to sixth cycles. This behavior indicated that as the test load increased, cracks developed in the slab bringing the behavior close to that of a cracked slab at the higher load levels. To accommodate the new design load for the Level B floor, externally bonded CFRP was used to increase the bending capacity of the slab. CFRP strips were installed in two directions on the top and bottom sides of the slab. Design and detailing of the CFRP was performed according to ACI 440.2R guidelines.

Table 1. Existing Moment and shear capacities and demands

Test	**ϕM_n (kN-m)**	**M_u (kN-m)**	**ϕV_n (kN)**	**V_u (kN)**	**Objectives**
Area 1	163.7	134.0	469.7	301.1	Evaluate performance of a column strip at positive moment region
Area 2	288.2	286.9	469.7	301.1	Evaluate performance of column strip at negative moment region



a) Scheme 1 - Positive moment testing b) Scheme 2 - Negative moment testing

Figure 2. Load Test Schemes and Point Loads Layout

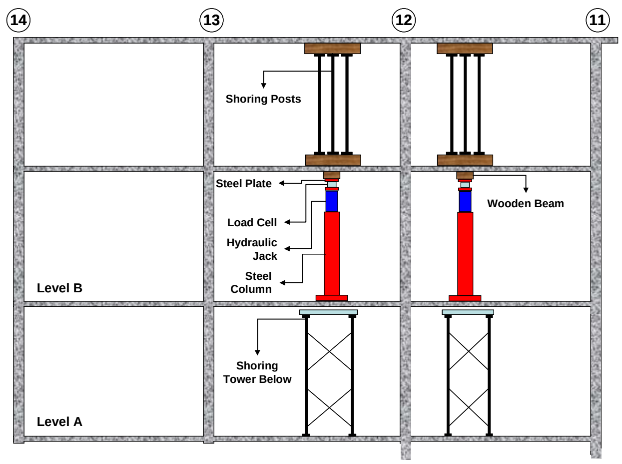


Figure 3. Test Setup

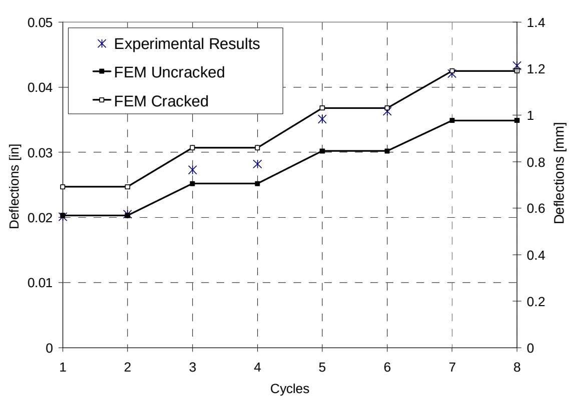


Figure 4. Comparison of Results for Scheme 2 Test

Case Study 2: Commercial Retail Building

To address the needs of a potential tenant, the owner of a commercial building located in Cleveland, Ohio evaluated options for upgrading the second level floor to house telecommunication equipment. The live load required for this equipment ranged from 6.0 to 7.2 kN/m^2.

The nine-story building was constructed in 1917 with steel frame and reinforced concrete floor systems. The existing engineering drawings provided only floor plans and geometry of the members, but no details were available for the structural steel members or steel reinforcement. The typical floor system consists of reinforced concrete joists supporting a 90 mm concrete slab reinforced with No. 10 bars spaced at 457 mm on center. A typical joist is 150 mm wide with a total depth of 40 mm and a span of 8.4 m. A site investigation revealed that, at mid-span, the joists are typically reinforced with two 25 mm square bottom bars that are bent up and extend into the adjacent spans and one No. 25 straight top bar at each end. No transverse reinforcement was located in the joists.

The proposed new loads included a superimposed dead load of 1.2 kN/m^2 for a new concrete overlay to level existing slab unevenness, and a service live load of 7.2 kN/m^2. Analysis indicated that the joists were deficient in both flexure and shear for the proposed loads. To eliminate the possibility of a brittle shear failure, all test joists were strengthened for shear using externally bonded CFRP prior to testing.

A pull-down type load was used in these load tests, in which the load was applied at two point on each joist using hydraulic jacks connected to (and pulled against) a reinforced concrete micro-pile that was installed on the ground floor below to provide necessary reactions (see Fig 5).

Two load tests were performed on the structural slab. Test 1 was performed on two joists isolated by saw-cutting the concrete slab. Failure of Test 1 was governed by yielding of reinforcement at the support, as evidenced by a large crack that developed on the top side of the slab. Based on the results of the Test 1, the joists were able to support a superimposed dead load of 1.2 kN/m^2 plus a live load of 6.5 kN/m^2. The shear performance was adequate up to flexural failure.

Test 2 was performed on the same two joists after a 75 mm thick bonded reinforced concrete overlay was installed and allowed to cure. The strengthened joists were loaded cyclically following the same protocol as before strengthening. Now that the existing capacity of the slab has been established, the maximum test load for Test 2 was limited to 85 percent of the factored design loads, as specified by Chapter 20 of ACI 318. This load level would not cause excessive damage and eliminated the need for additional repair after the test. As the load approached the maximum test load, a number of flexural cracks developed on the top side of the overlay at both ends of the joists (negative moment regions). The number and distribution of the cracks indicated that sufficient bond existed between the existing slab and the new overlay to monolithic behavior. Based on the test results, the strengthened joists were rated for 1.7 kN/m^2 superimposed dead load plus 7.2 kN/m^2 live load. Fig. 6 shows the measured deflection of the existing and strengthened floor joists at 64.5 kN.

Case Study 3: Parking Garage

The parking garage in this case study supplies approximately 1,000 parking spaces for a commercial office building in Atlanta, Georgia. The garage consists of four parking levels connected to the building by a link bridge. The garage deck is 200 mm thick, constructed of 2-way cast-in-place concrete, post-tensioned flat slab. The typical bay size is 8.23 m in the E-W direction, and 10.67 m in the N-S direction.

An earlier remedial repair consisted of heavily-reinforced, 75 mm thick and 900 mm wide Gunite beams that were added to the underside of the slab between the drop panels (see Fig. 7). During a non-destructive testing (NDT) investigation (pulse velocity test), it was found that 105 of the 133 beams tested had delaminated. Initial conclusions related the delamination to inadequate bond due to poor roughening of the underside of the slab prior to the installation of the gunite beams and an incorrect placement method - using the cast-in-place method instead of the form-and-pump technique.

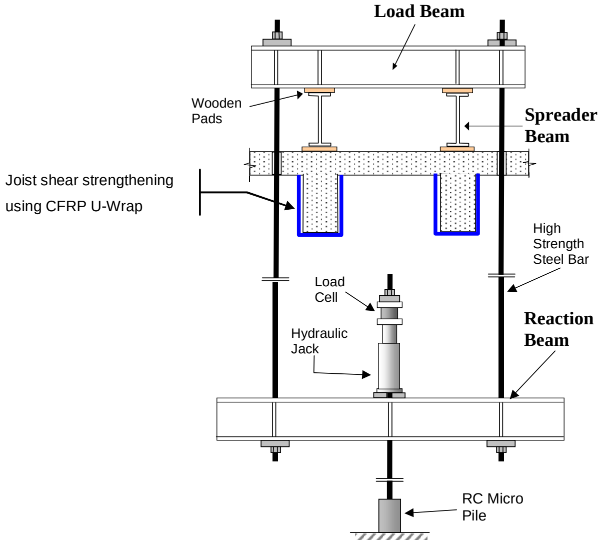


Figure 5. Cyclic Load Test Setup for Test 1 and Test 2

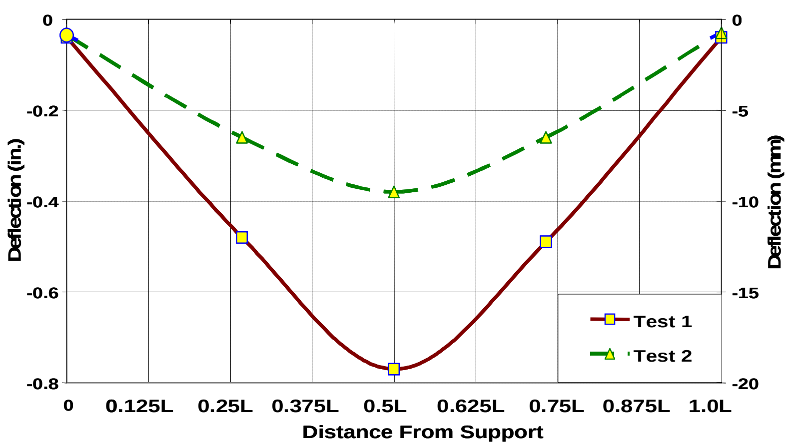


Figure 6. Comparison of Measured Deflections at P = 64.5 kN

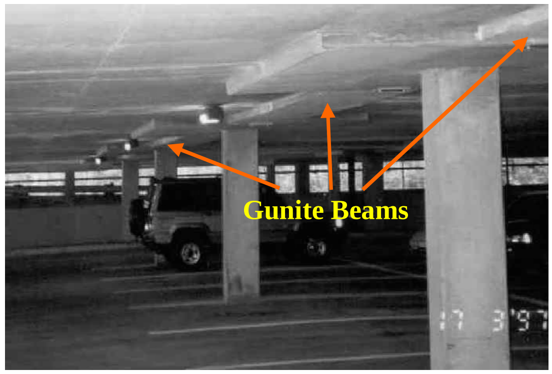


Figure 7. Gunite beams applied to the underside of the garage decks

Exterally bonded CFRP reinforcement was considered for flexural strengthening of the slab. Seven load tests were carried out that included testing an 8.23 m span with a Gunite beam in good condition, without Gunite beam, and with CFRP strengthening (Gunite beams were removed). Also, 10.67 m spans without strengthening was tested.

The CLT load testing procedure utilizes hydraulic jacks to induce the internal forces equivalent to those resulting from distributed loads. The load test equipment and setup used are shown in Figures 8 and 9. During the load test, the mid-span deflection was monitored for stability. When the deflection increased with a constant or dropping load, the test was halted.

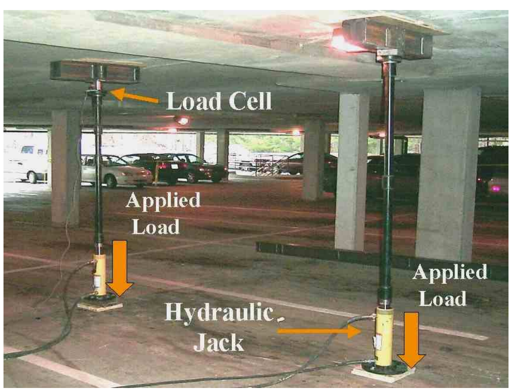


Figure 8. Hydraulic jacks

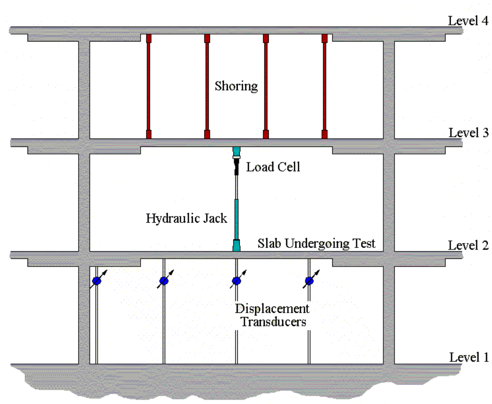


Figure 9. Test setup

In Test 1 (8.23 m span with a bonded Gunite beam), the span was loaded to 85% of the ultimate load and the performance was satisfactory. In Test 2 (8.23 m with unbonded Gunite beam removed), the span was loaded to 85% of ultimate. At the last loading cycle, the span became inelastic and the test was halted. In Test 3 (8.23 m with Gunite beam removed) the slab was tested to service load level only. For Tests 4 and 5 (10.67 m unstrengthened spans), the slabs performed well to the ultimate loads. Tests 6 and 7 were performed on the same 8.23 m with Gunite beam removed as tested in Tests 2 and 3, but after strengthening with CFRP. Both slab spans performed satisfactorily after strengthening. Figure 10 shows a comparison of load-deflection envelopes for 8.23 m spans with bonded Gunite beam in good condition (Test 1) and without Gunite beams but strengthened with CFRP (Tests 6 and 7). Based on the tests results, the CFRP strengthening of 8.23 m spans where delamination of Gunite beams was found was warranted and it performed satisfactorily. Figures 11

(Tests 2 and 6) shows the load-deflection envelopes for two 8.23 m spans tested without Gunite beams, before and after strengthening with CFRP.

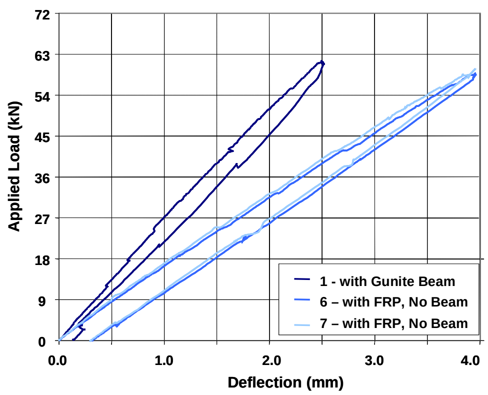


Figure 10. Load deflection for Tests 1, 6 & 7.

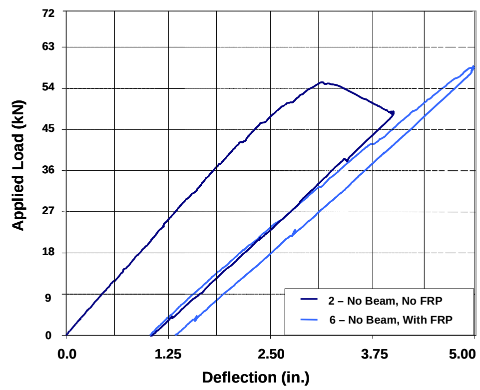


Figure 11. Comparison of Tests 2 and 6.

SUMMARY AND CONCLUSIONS

In the described case studies, the cyclic load test method allowed for efficiently verifying the capacities of the existing structures prior to renovation. For the first case study, load testing was used to determine the carrying capacity of the existing slab and confirmed the reliability of the analytical models that were later used to determine the required level of strengthening at various locations. For the second case study, the load tests provided information on the load carrying capacity of the existing joists and their governing failure mode, confirmed the performance of the bonded RC overlay, and the effectiveness of the FRP shear reinforcement.

For the parking garage, the load tests showed that strengthening of spans with delaminated Gunite beams is warranted while the spans with no delamination were found to perform well. The load tests further showed that the spans repaired with CFRP performed well.

For all described cases, externally bonded CFRP reinforcement provided a cost-effective strengthening solution. The CFRP was designed to supplement the existing capacity of the structural element. The existing structural components of the structures were verified to have adequate capacity to carry the design service loads without the contribution of the CFRP, so it was designed only to supplement the existing capacity of the structural element.

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