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SEISMIC STRENGTHENING OF REINFORCED CONCRETE BRIDGE PIER WITH FRP COMPOSITES

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SUMMARY

In-situ lateral load tests were conducted on Interstate 15 to determine the strength and ductility of an existing concrete bridge. The bridge was designed and built in 1962 for gravity and wind loads; as such, it did not possess adequate reinforcing details for seismic resistance. The purpose of the tests was to examine the improvements that can actually be achieved using a carbon Fiber Reinforced Polymer (FRP) composite retrofit. Three tests were conducted; the first test was on an as-built bent (Bent #5); the second test was on an as-built bent (Bent #6) which was retrofitted with FRP composites for seismic strengthening; and the third test involved repair of Bent #5, FRP retrofit and testing. The paper presents analysis of the results of the as-is (Bent #5) and retrofitted (Bent #6) tests. The design of the FRP composite was developed based on rational guidelines for the columns, cap beam, and cap beam-column joints to provide a specific displacement ductility. Agreement between analytical and experimental results was observed which included the peak lateral displacement of the bent, the peak lateral load, and the locations of the plastic hinges. The FRP composite was able to strengthen the cap beam-column joints effectively for an increase in shear stresses of 35 percent, while the peak lateral load capacity was increased by 16 percent. The displacement ductility was improved from 2.8 for the as-is bent (Bent #5) to 6.3 for the retrofitted bent (Bent #6), which exceeded the retrofit objective of doubling the ductility. The tests demonstrated that reinforced concrete (RC) bridge bents which were designed without any seismic detailing can be rehabilitated effectively with FRP composites. The advantages of FRP composite rehabilitation include cost effectiveness, fast construction, minimum traffic disruption, and superior strength per unit weight.

INTRODUCTION

Recent earthquakes such as the one in Loma Prieta in 1989, in Northridge in 1994, and in Kobe in 1995, have repeatedly demonstrated the vulnerabilities of older reinforced concrete bridges to seismic deformation demands. While experimental validation of retrofit concepts for such bridge columns exists [Priestley et al. 1996, Seible et al. 1997], full-scale in-situ tests of structural systems, such as retrofitted bridge bents, are rare. A three-span segment of the standing portion of the Cypress Street viaduct was instrumented and tested for evaluating retrofit techniques, including addition of external post-tensioning rods to each bent cap, and placement of external shear reinforcement over the column clear height. It was found that retrofitting had little effect on overall lateral stiffness but increased strength, and displacement ductilities [Bollo et al., 1990]. Full-scale tests of reinforced concrete circular columns were performed in St. Louis [Gamble and Hawkins, 1996]. Several columns were tested in the as-built condition, after being retrofitted with tensioned steel bands, or glass FRP composite jackets. All of the retrofits functioned properly, and prevented the lap splices from failure at significant force and deformation levels. Rehabilitation techniques involving steel jacketing, concrete jacketing, and FRP composite jackets for columns have been developed at the University of California San Diego [Priestley et al., 1996]. Various circular and rectangular columns with carbon FRP composite retrofits were tested in the laboratory [Seible at al. 1997]. The tests showed that the carbon FRP composite jacket was as effective as a comparable steel jacket system. Several studies have been carried out regarding the shear and flexural strengthening of RC beams with FRP composites.

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Experimental studies [Priestley et al. 1997] have shown that older RC concrete joints do not meet the confinement and shear reinforcement details specified in the ACI Committee 352R-91 recommendations [ACI 1991]. Methods available for strengthening these weak joints include: (1) jacketing of the joint region [Alcocer and Jirsa 1993]; (2) joint section increase with special dowels and post-tensioning rods [Lowes and Moehle 1995]; (3) post-tensioning of the beam combined with fiberglass wraps on the beam and column [Sexsmith et al. 1997]; and (4) corrugated steel jacketing of the beam-column connection [Biddah et al. 1997]. Retrofit of RC joints using FRP composites has not received much attention. In a demonstration project, three columns, the cap beam, and joints of the Highland Drive Bridge at Interstate 80 in Salt Lake City were wrapped with carbon FRP composites in September, 1996. The bridge was in service during the retrofit. The details of the field application were presented in [Pantelides et al. 1997]; the analytical techniques for the retrofit design and comparison with full-scale experiments of similar joints were presented in [Gergely et al. 1998]. Detailed procedures for improving the shear capacity of RC T-joints using FRP composites were recently established [Gergely and Pantelides 1998].

As part of the Interstate 15 reconstruction project, the I-15 bridges along the Wasatch front are currently being demolished and are being replaced with a new freeway system. This provided an opportunity to test two bents of the old freeway. The bridge under consideration was designed and built in 1962, before the 1971 San Fernando earthquake, and was missing the basic reinforcement details necessary to provide adequate lateral load capacity and ductility. In-situ lateral load tests of three RC bents of the South Temple Bridge were performed, with and without FRP composites, to demonstrate the ability of composite materials to improve resistance to seismic loading. Specifically, the FRP composite was designed to double the displacement ductility of the as-is bent. The objectives of the retrofit and in-situ tests were to: (a) determine the capacity of the bent in the as-built condition (Bent #5); and (b) determine the improvement in strength and ductility of the bent with the FRP composite retrofit (Bent #6). Two bents (Bent #5 and #6) and the deck between them, which were part of the Northbound lanes of the South Temple Bridge at I-15, were made available for testing prior to demolition.

ANALYSIS OF AS-BUILT BENT #5

An estimate of the peak lateral load and the maximum horizontal displacement of the bent were determined by performing a nonlinear static pushover analysis, using the program DRAIN-2DX [Prakash et al. 1992]. The yielding sequence of the structural members was observed, and member forces were calculated at the ultimate displacement. This information was necessary for the design of the rehabilitation using FRP composites. To construct the 2-D model of the structure, the existing conditions had to be evaluated and compared with the original design plans. The bent general dimensions are shown in Figure 1. The column transverse reinforcement was inadequate in the lap-splice region, and the anchorage of the longitudinal reinforcement in the pile cap and in the cap joint was insufficient. There were no transverse hoops in the column cap-beam connection, and in addition the column bars terminating in the connection were not anchored adequately.

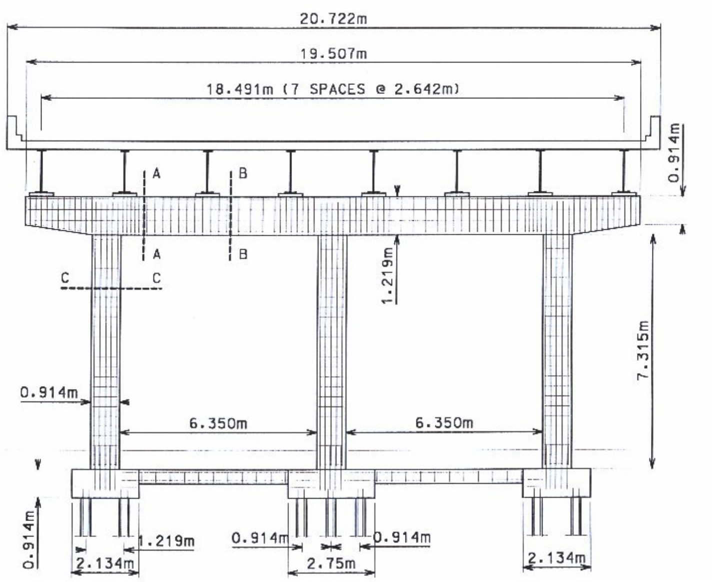

Figure 1. Bent dimensions

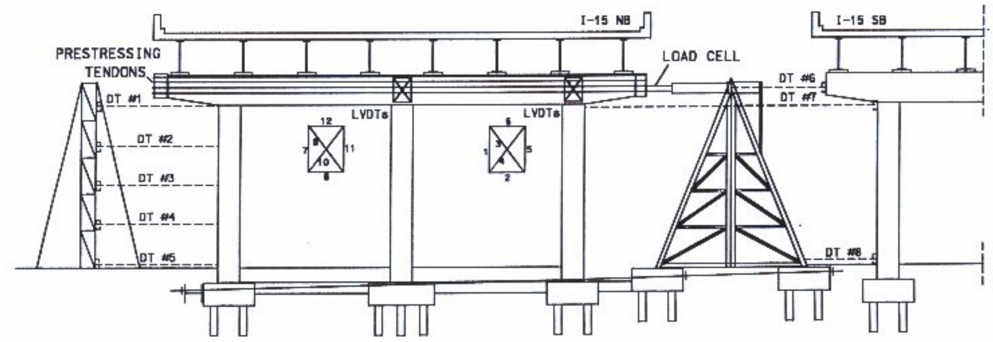

Figure 2. Loading condition

The loading configuration of the bent is shown in Figure 2. The horizontal load was applied at the cap beam level. To pull on the bent, the load was transferred to the far end of the bent by twenty prestressed tendons. The tendons were prestressed to take out the sag and the slippage in the anchorage system. The connection details were such that there was no difference between pulling from the far end and pushing from the near end. Half of the originally present dead load from the deck was acting on each bent, which was transmitted to the cap beam by welded plate girders, bearing on eight reinforced concrete pedestals. The specified concrete strength was 21 MPa, and the yield strength of the longitudinal and the transverse reinforcement was 276 MPa. Due to the loading system configuration, lateral movement was prevented at the level of the pile caps (nodes 1, 2 and 3).

COMPOSITE RETROFIT DESIGN FOR BENT #6

The goal of the seismic retrofit was to improve the displacement ductility of Bent #6 by a factor of two as compared to the as-is Bent #5. From the pushover analysis, it was concluded that the bent had deficiencies in the following areas: the confinement of the column lap splice region, the confinement of the plastic hinges, the column shear, the shear in the joint region, and the anchorage of the column longitudinal reinforcement into the cap beam. To address these issues, each element was analyzed and the structural retrofit using FRP composites was specified. A complete description of the design is given elsewhere [Gergely and Pantelides 1998, Pantelides et al. 1999].

Flexural plastic hinge confinement for columns

The benefits of the composite jacket in the plastic hinge region include providing confinement of the core, and prevention of the concrete cover from spalling off which provides the longitudinal reinforcement with lateral stability. The composite layout was designed as a square jacket with twice the FRP composite thickness required for an equivalent circular jacket, as recommended from tests carried out by Seible et al. [1997]. The thickness of the FRP composite layers was calculated using the expression derived in [Seible et al. 1997], in which the thickness was found to be proportional to the column dimensions, and inversely proportional to the product of the ultimate stress and ultimate strain of the composite. The following parameters were used: (1) the ultimate concrete strain was taken as 0.0065 based on the required ductility increase [Gergely et al. 1998]; (2) the compressive strength of confined concrete was taken as 24 MPa; (3) the FRP composite material used in this application was 48,000 fibers per tow unidirectional carbon fiber. The number of tows per 25.4 mm of sheet (pitch) was 6.5, and the width of carbon fiber sheets was 457 mm. The ultimate FRP composite strength was evaluated, using tensile tests according to ASTM D-3039 specifications (ASTM 1996), as 628 MPa and the ultimate FRP composite strain was found as 0.01; and (4) a flexural capacity reduction factor of 0.9 was used. Since the columns had a rectangular section, the calculated thickness was doubled.

Lap splice clamping for the columns

The thickness of the FRP composite required for clamping the lap splice region was determined based on an equivalent circular jacket, and was then doubled. The thickness of the FRP composite required to clamp the lap splice region was found to be directly proportional to the effective column diameter D_e, and inversely proportional to the modulus of elasticity of the composite, E_j [Seible et al. 1997]. The modulus of the composite jacket was determined experimentally using tensile coupons as 64,730 MPa.

Shear strengthening of the columns

Each of the shear resisting components were evaluated and then subtracted from the design shear. The design shear was estimated as 1.5 times the column shear at yielding. The column shear capacity was evaluated using the approach given in [Priestley et al. 1996]. It was found that shear strengthening was not required outside the plastic hinge region. The thickness of the composite jacket inside the plastic hinge region is inversely proportional to the composite's modulus of elasticity and the column dimension [Seible et al. 1997]. A shear strength reduction factor of 0.85 was used.

Flexural plastic hinge confinement for the cap beam

The shear capacity of the cap beam was found to be adequate outside the joint region. The design of the FRP composite for confinement of the plastic hinge followed the procedure presented earlier for the columns.

Shear strengthening of the column-cap beam joint

From analysis, the peak lateral load for the retrofitted Bent #6 was increased compared to the as-built bent, which resulted in higher joint shear forces. The design of the FRP composite for the joint region was calculated using the following procedure [Gergely and Pantelides 1998]: (1) the horizontal and vertical shear forces and stresses at the ultimate displacement were calculated for the middle joint; (2) the principal tension and compression stresses for the joint were obtained, from which the optimal orientation of the fibers was found as $\pm 48^0$ with respect to the longitudinal axis of the cap beam; however, an angle of $\pm 45^0$ was used in the actual test; (3) the increased demand for the joint principal tensile stress was calculated for the corresponding increase in the lateral load capacity of the bend; and (4) the number of layers required to satisfy this increase was found by dividing the increase in principal tensile stress by the stress developed using one layer of carbon fiber sheet; the latter is proportional to the effective depth of the joint, the thickness of one layer of the carbon fiber sheet, the elastic modulus of the FRP composite material, E_j, and the average axial strain of the composite at peak lateral load which was experimentally determined as 0.2% in laboratory experiments, and was later verified in the test by strain gage measurements on the FRP composite. This strain is limited by the bond failure between the FRP composite and the concrete; it is important to use this reduced level of strain in shear-critical applications. The number of layers found by this process was 3.65. Four layers of unidirectional fabric were applied to obtain a symmetric FRP composite jacket around the joint. These layers were provided in both directions to take into account the cyclic nature of the applied loads, they were continuous and covered all four faces of the cap beam.

The composite layout for Bent #6 is shown in Figure 3. The number of layers was found by dividing the required thickness by the thickness of one layer of the carbon fiber sheet, which was 1.32 mm. A 51mm gap was left at the top and bottom of the columns, as shown in Figure 3, to allow independent movements to occur at the joints, and to prevent bearing of the advanced FRP composite jacket on the footing or the bottom of the cap beam. The additional six layers of 152 mm wide composite sheets (tapes) shown in Figure 3 were provided in order to decrease the level of stresses in the column longitudinal reinforcement extended into the joint.

EXPERIMENTAL VALIDATION

Modifications were made to the substructure in order to resist the shear from application of the lateral load. The deck supports were also modified in order to perform the tests safely, without the danger of loosing the deck.

Test of the as-built Bent #5

The horizontal cyclic loading was applied in a quasi-static manner. In the first part of the test, a force-controlled test was performed with increasing load steps. In each load step, the load was applied for three cycles, and each cycle had a push and a pull segment. After first yielding, the test was continued by increasing the displacement as a fraction of the yield displacement. From a bilinear approximation of the load-displacement behavior, the displacement ductility of the bent at the end of the test was found to be equal to 2.8, as shown in Figure 4. This is reasonable for bridges of this type built in the 1960's, which were not designed for earthquakes. The bridge was designed for lateral wind loads. Significant diagonal tension cracks formed in the cap beam-column joint region, showing a gradual degradation of the concrete. Flexural cracks developed at the top of the columns that opened as the bending capacity was reached. These cracks had various widths with the largest being at the interface of the column and the cap beam which were 2 mm wide. The cracks went all around the column perimeter in the plastic hinge region. The first layer of the column longitudinal reinforcement extended into the joint was exposed on the south side, suggesting bar slippage and severe deterioration of reinforcement bond.

Test of the retrofitted Bent #6

The test setup and the testing procedures were similar to the ones described for Bent #5. The load-displacement curve for Bent #6 is shown in Figure 5. This is a good example of ductile behavior, where the lateral capacity is maintained up to 200 mm with no significant damage to the structure. From a bilinear approximation of the load-displacement behavior, the displacement ductility of the bent at the end of the test was found to be equal to 6.3, as shown in Figure 5. Thus, the goal set out in the FRP composite design of doubling the displacement ductility was achieved. Large cracks in the concrete were visible at the interface between the column and the cap beam which reached 6 mm in width. In addition, at both column ends, shallow spalling cracks in the 51 mm

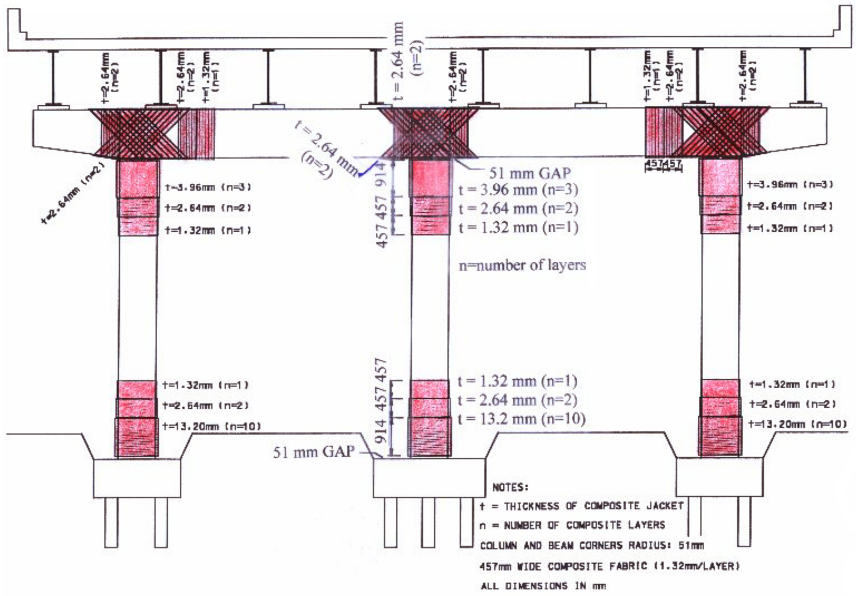

Figure 3. Composite layout for Bent #6

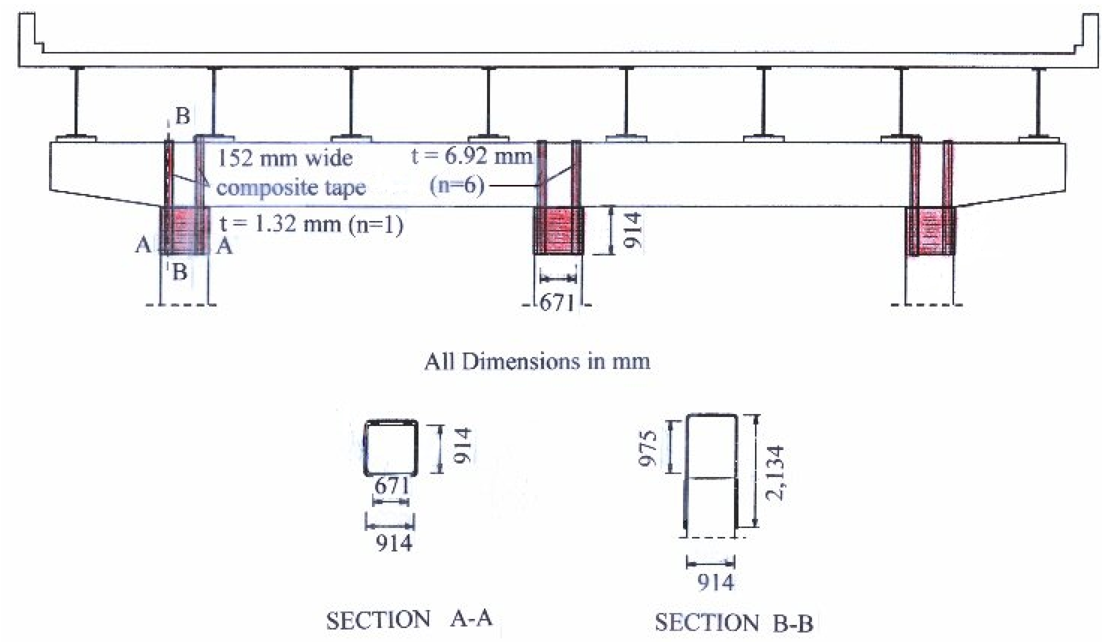

Figure 3. Composite layout for Bent #6 (continued)

gaps were formed. The FRP composite layers delaminated in the column-cap beam joint region, and at the top and bottom of the columns. This occurred because of the low concrete strength in surface tension. The outside carbon fiber layers had flexural cracks along the fiber direction. Visible tensile failure of the 152 mm tensile composite tapes was observed. Strain gages measured the peak value of the strain in the composite at the cap beam-column joint as 0.22%, which was about a fifth of the ultimate value of 1%. This low strain level in the composite justifies the value used to design the composite layout for shear strengthening of the joints.

The failure of the composite and cracking patterns of the columns, i.e. delamination or bulging at the top or base of the columns due to concrete dilation, are similar to those observed in laboratory experiments [Seible et al. 1994]. The failure mechanism of the FRP composite at the joints, i.e. delamination of the composite due to failure of the concrete in surface tension, was also observed previously in laboratory experiments [Gergely and Pantelides 1998]. At a strain of approximately 0.2% the FRP composite sheets debond from the concrete surface and never reach their tensile strain capacity of 1%. Comparing the performance of the as-built Bent #5 (Figure 4) and the retrofitted Bent #6 (Figure 5), it can be seen that the FRP composite retrofit significantly increased the ductile capacity of the as-built bent, and allowed the bent to achieve a higher lateral load capacity while maintaining a significant gravity load.

CONCLUSIONS

The in-situ tests conducted at the South Temple Bridge have shown that externally bonded FRP composites can greatly enhance the displacement ductility of RC bridge bents, which were not designed to any seismic standards. In addition, the cap beam-column joint shear capacity was enhanced, the overall damage was controlled, and a residual strength was left to support the dead load. Delamination of the composite from the concrete surface was the dominant mode of failure of the composite and can occur at about only one fifth of the composite's tensile capacity. The as-built bent had extensive diagonal cracks in the joint region which extended into the cap beam at the bottom level of the horizontal reinforcement, and substantial flexural cracks in the upper region of the columns; however, it maintained a high lateral load capacity up to a displacement ductility of 2.8. The retrofitted bent reached a displacement ductility of 6.3, while the peak lateral load capacity was also increased by 16 percent. The FRP composite was able to strengthen the joint which had an increase in shear stresses of 35 percent.

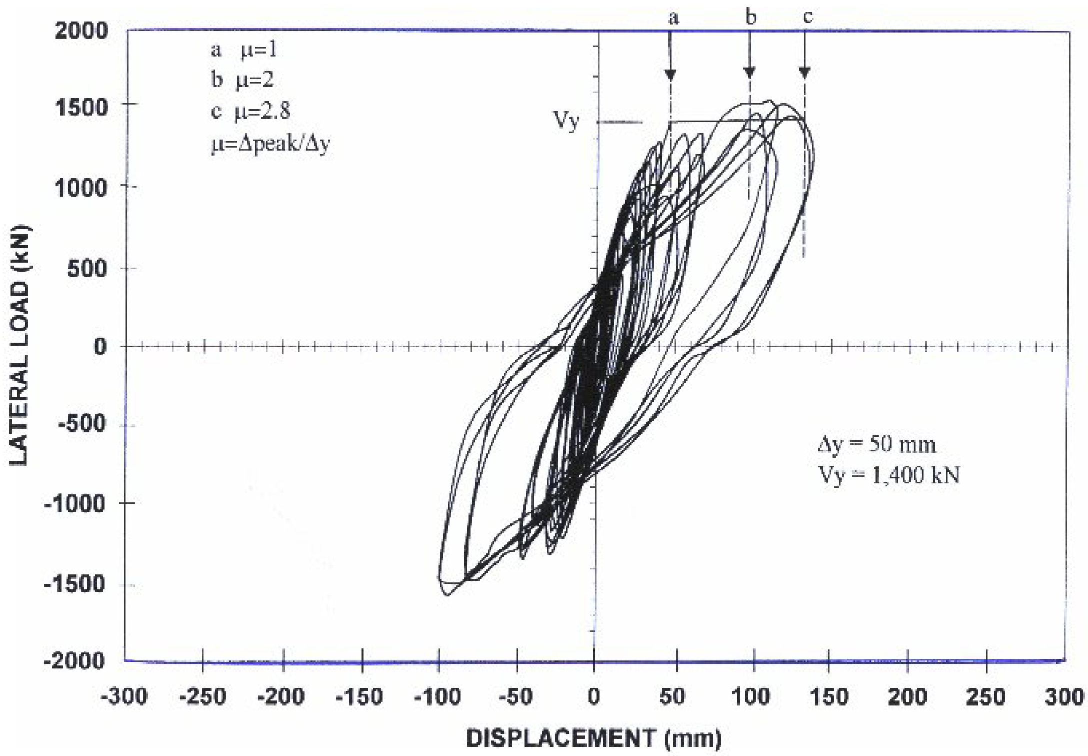

Figure 4. Load-displacement response of the as-built Bent #5

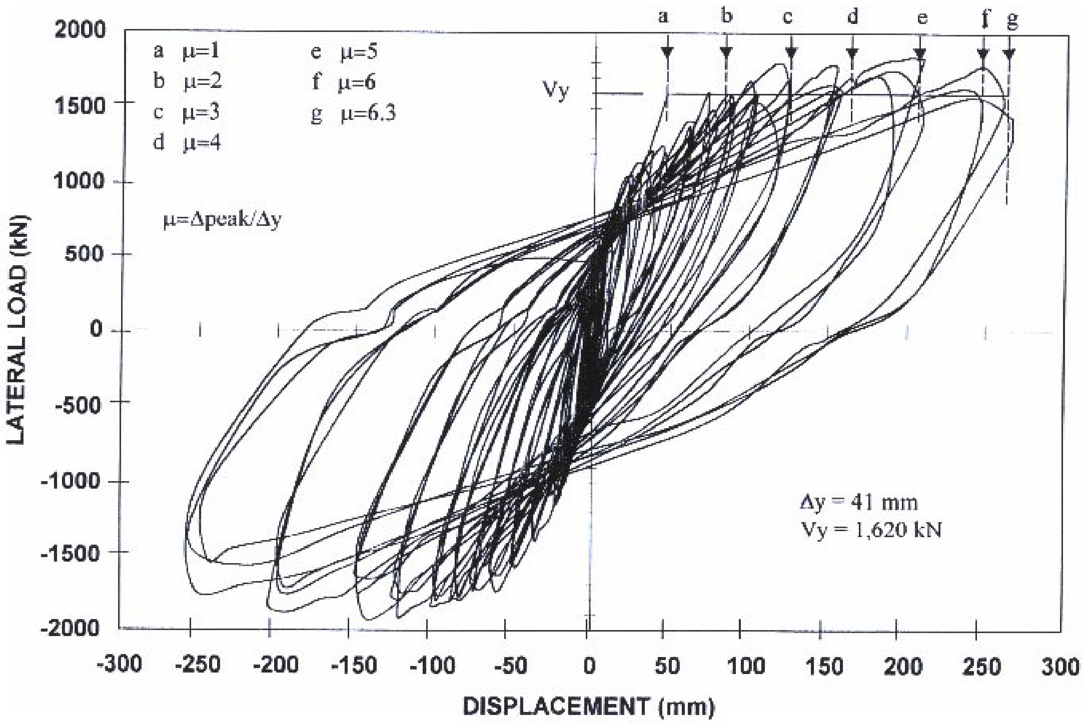

Figure 5. Load-displacement response of Bent #6 with composite retrofit

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