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SEISMIC RETROFIT OF NON-PRISMATIC RC BRIDGE COLUMNS WITH FIBROUS COMPOSITES

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SUMMARY

This paper describes the experimental part of a study on shake table response of reinforced concrete columns with structural flares. One as-built and two retrofitted 0.3-scale specimens are discussed. Two fiber-reinforced plastic (FRP) jackets, one with glass fibers and the other with carbon fibers were studied. The primary function of the retrofit was to improve the shear capacity of the columns. Both retrofit methods were found to be effective in changing the failure mode from flexure/shear to flexure and in improving the displacement ductility. The mechanical performance of the glass fiber and carbon fiber composites was nearly the same.

INTRODUCTION

The majority of the columns in the Mission Gothic Bridge in the Los Angeles area were severely damaged or failed in shear during the 1994 Northridge earthquake [NIST, 1994]. These columns had architectural flares, which had been assumed to spall off during a strong earthquake and to allow for plastic hinging at the end of the column. During the earthquake, however, the flares enhanced the flexural capacity at the ends and shifted the plastic hinge to a point near the mid height of the columns. The reduction in the shear span led to nearly doubling the shear force and failure of the columns.

A large number of recently constructed bridges in Nevada are supported on flared columns. Following the failure of the Mission Gothic Bridge, it was decided that an evaluation of these columns need to be made and necessary corrective measures should be taken. This paper presents a summary of the use of advanced composites in enhancing the seismic performance of the columns.

EVALUATION OF BRIDGES

Four representative bridges constructed from the early 1980's to early 1990's were selected for the study. All these bridges were supported on columns that had structural flares. The flares in these columns were reinforced to provide at least a part of the required flexural capacity. The more recent columns were reinforced with a core steel cage with constant diameter that extended over the entire column height in addition to having a structural flare. The earlier columns did not have the core steel. Inelastic lateral load analysis of the piers revealed that the columns with a core steel cage had sufficient ductility, whereas some of the columns without the core could experience shear failure at moderate levels of ductility [Wehbe and Saiidi, 1999]. Columns of Bridge 1250, a sixteen-span structure supported on nearly 100 columns were selected for further study. Cyclic testing of large-scale specimens revealed that only the columns with relatively large longitudinal steel ratio were susceptible to shear failure [Wehbe, et al., 1997]. Therefore, subsequent shake table tests were conducted on specimens that

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represented heavily-reinforced columns. Another conclusion of the cyclic tests was that the plastic hinge in columns with structural flares forms at some distance from the end, which is advantageous because it reduced the detailing demand at the connections. A continuous jacket retrofit was found to shift the plastic hinge closer to the connection. The tests showed that a jacket with an intermediate gap at a preassigned plastic hinge location is effective in controlling the column behavior.

TEST SPECIMENS

Three 0.3-scale specimens tested on one of the University of Nevada, Reno, shake tables are discussed in this paper. The first specimen, named FA, represented as-built columns, and the other two, named FRG and FRC, were retrofitted with glass fiber epoxy and carbon fiber epoxy composites, respectively. All the columns in Bridge 1250 were pinned at the base and flared over the upper 4.8 m. To simplify testing, the specimens modeled the columns in an upside down position with flares over the lower part of the column and the pinned connection at the top. Details of the specimens without retrofit are shown in Fig. 1. The footing in the specimens was overdesinged to ensure that damage is limited to the columns. The longitudinal bars were of f13mm and the transverse bars were of f 4.5mm. Both the longitudinal and transverse steel ratios closely modeled the corresponding ratios in the prototype columns. The measured yield stress for the longitudinal and transverse bars was 462 and 417 MPa, respectively. The average measured steel yield stress in the columns of Bridge 1250 was 452 MPa.

Normal weight concrete with a maximum aggregate size of 10mm was used. On the day of testing the concrete compressive strength was 37.6 , 48.0, and 47.9 MPa, for FA, FRG, and FRC, respectively.

Composite Retrofit

Most of the space under Bridge 1250 is utilized as offices and garages for the Regional Transportation Commission of Reno. Walls and other facilities have been constructed around the bridge column. It was important to identify the least intrusive method of retrofit for the columns to minimize reconstruction of the existing facilities. Because of their flexibility, fiber reinforced plastics (FRP's) can be installed in areas with limited working space. The use of unidirectional FRP's for seismic retrofit of prismatic columns has increased significantly in the past few years. However, no application of the composites on flared elements has been reported. What makes flared columns different with respect to FRP composite installation is that the fiber angle changes as the composite is wrapped around the column. Typically, in prismatic columns the fibers are perpendicular to the element axis so that they enhance confinement and shear strength. Fibers that are not perpendicular to the column axis introduce a force component that is parallel to the axis. This force increases the flexural strength of the column, which is generally undesirable.

To minimize the angle change for the fibers, the composite jacket was formed by a series of butted straps. The width and the number of the straps were reduced in the zone at which the plastic hinge was to be formed. Figure 2 shows the details of the glass fiber composite jacket. The carbon fiber jacket was similar. The gap at the bottom was placed to avoid bearing of the jacket on the footing. The Tyfo Fiberwrap composite was used. The jacket thickness was designed using the provision of [USDOT, 1995] for shear enhancement. The objective of the retrofit was to improve the shear capacity. A factor of safety of two was used against shear failure. The concrete and steel contribution to the shear strength was accounted for using the provisions of [Caltrans, 1998]. The design tensile stress for the glass and carbon fiber epoxy composites was 124 and 393 MPa, respectively, as recommended in Ref. 5. In FRG, two wraps were installed in the plastic hinge area and three elsewhere. The plastic hinge area in FRC had one wrap with two wraps installed over the rest of the column.

Shake Table Testing

The as-built specimen was analyzed subjected to a large number of earthquake records using a nonlinear response history analysis program, RCShake, before an earthquake record for shake table testing was selected [6]. The criteria considered included relevance of the record to the type of earthquakes expected in the region, ductility demand on the column, and the shake table limitations. The Sylmar Hospital record obtained during the 1994 Northridge earthquake was selected. The measured peak ground acceleration for this record was 0.6g. The peak was adjusted in the course of the shake table tests to achieve different levels of ductility demand. The time axis of the record was compressed to account for the difference between the frequency of the test specimens and that of the prototype. Each specimen was subjected to successive earthquake runs with increasing peak amplitude. The first two input had a peak amplitude of 0.12g and 0.24g. The input peak acceleration in the third run was 0.45g, and was incremented by 0.15g in subsequent runs. Intermittent free vibration tests were conducted to measure the change in frequency and damping of the specimens as they were damaged. In all the tests, the columns were subjected to an axial load of 285 kN that represented the gravity load effect in the prototype. This load fluctuated during the tests, but the effect of the axial load variation was nine percent or less on the flexural capacity of the columns [Martinovic, et al. 1999].

The actual recorded peak table acceleration varied from the target. The spectral analysis of the actual table motions, however, indicated that the spectral accelerations for all three specimens were nearly the same in the frequency range of interest [Martinovic, et al. 1999].

TEST RESULTS

As-Built Specimen

Flexural and shear cracks were became visible during the third earthquake run in Specimen FA. As the amplitude of table acceleration increased, cracks became wider and concrete began to spall in the compression areas under bending. Spalling of the concrete was followed by an outward buckling of the longitudinal bars and rupture of some of the ties. During the last run with the peak table acceleration of 1 g, several of the longitudinal bars ruptured due to low-cycle fatigue. Figure 3 shows FA at the end of the test. The test data showed that many of the column ties yielded under the peak load and that the shear capacity of the column in the plastic hinge zone had been reached. The envelope of the measured load-displacement curves for FA is compared with those of the retrofitted specimens in subsequent sections. More details about the performance of the specimen are presented in [McElhaney, et al., 1999].

Retrofitted Specimens

The response of the two retrofitted specimens was generally the same. Minor cracking of epoxy was observed during Run 2. In subsequent runs, more cracks were observed mostly at the junction of composite straps and in the plastic hinge zone, where a smaller number of composite wraps had been installed. No shear cracks were visible in the plastic hinge area. Following the tests, the composite jackets were removed in the plastic hinge area. Again, no shear cracks were observed. Both specimens failed in flexure due to the rupture of the longitudinal bars in the preassigned plastic hinge area. Figure 4 shows FRG at the completion of the test.

EFFECT OF RETROFIT

The objective of the retrofit was to increase the shear capacity of the columns. Even though the as-built column did not fail in a predominantly shear failure mode, yielding of the ties and wide shear cracks reduced the ductility capacity. It was, hence, anticipated that the retrofit would improve the ductility capacity in addition to increasing the shear strength. Figure 5 shows the measured load-deflection envelopes for all three specimens. It can be seen that the FRP jackets affected the response in three ways: the initial stiffness, peak load, and ductility. The increase in the initial stiffness was mainly attributed to the contribution of the composite layer. Another possible reason is the application of a layer of epoxy on the column prior to the attachment of the FRP wraps to provide for a smooth surface. This layer of epoxy tends to close the microcracks that are normally present due to concrete shrinkage and improve the initial stiffness. The increase in the peak load is also attributed to the jacket acting as a shell and resisting some flexural stresses in addition to the enhancing concrete confinement. The composite jacket does act as a flexural member even though the fibers are essentially in the transverse direction. The jacket contribution is due to a nominal number of fibers that run perpendicular to the main fibers to avoid fretting. Furthermore, the epoxy matrix has a reported tensile strength of 66 MPa, which would also provide some flexural strength. These contributions were quantified in [Martinovic, et al. 1999]. The third effect is the increase in the displacement ductility that is evident in the figure. To quantify the ductility increase, the load-displacement envelopes were idealized by elasto-plastic curves, taking the last data point shown for each specimen as the ultimate point. Note that the actual failure point was after the last point shown but it could not be accurately measured. It was found that the displacement ductilities were 5.4, 7.4, and 7.9 for specimens FA, FRG, and FRC, respectively, indicating an approximately 40 percent improvement in the displacement ductility capacity as a result of the retrofit. The difference between the response of FRG and FRC was minor.

The effectiveness of the jacket in reducing the strains in the ties is shown in Fig. 6. It can be seen that the tie in FA yielded even under moderate levels of table motions. In contrast, the peak strain in the corresponding ties in the retrofitted specimens was only one-half of the yield strain. This trend was typical for the ties at or near the center of the plastic hinges. The improvement in the shear performance and the displacement ductility led to the conclusion that the FRP jackets were effective in accomplishing their objective. Extensive analytical studies of the specimens were also conducted [Martinovic, et al. 1999], but could not be included in this paper due to space limitation. The tie strains in FRC and FRG were nearly the same indicating that either jacket can be used for seismic retrofit.

CONCLUSIONS

1) The method of applying FRP composite wraps in the form of butted straps developed in this study was found to be an effective mean of constructing composite jackets for non-prismatic members.

2) The composite jackets accomplished their primary goal of enhancing the shear and displacement ductility capacity of the columns with structural flares. The jackets changed the mode of failure from flexure/shear to flexure.

3) The seismic performance of the glass-fiber epoxy jacket was very similar to that of the carbon-fiber epoxy composite. It should be noted that durability of the two composite types was not evaluated in this study.

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REFERENCES

1. California Department of Transportation (Caltrans)g, (1998) “Memo to Designers, 20-4,” Engineering Service Center, Earthquake Engineering Branch, November 1998.

2. Laplace, P., (1999) “Shake Table Testing of Flexure Dominated Reinforced Concrete Bridge Columns,” M.S.C.E. Thesis, Department of Civil Engineering, University of Nevada, Reno, August 1999.

3. Martinovic, F., M. Saiidi, D. Sanders, and F. Gordaninejad, (1999) “Dynamic Testing and Analysis of Nonprismatic Reinforced Concrete Bridge Columns Retrofitted with FRP Jackets,” Report No. 99-xx CCEER (Under Review), Department of Civil Engineering, University of Nevada, Reno.

4. McElhaney, B., M. Saiidi, and D. Sanders, (1999) “Shake Table Testing of Flared Bridge Columns with Steel Jacket Retrofit,” Report No. 99-xx CCEER (Under Review), Department of Civil Engineering, University of Nevada, Reno.

5. National Institute of Standards and Technology (NIST), (1994) “1994 Northridge Earthquake: Performance and Structures, Lifelines, and Fire Protection Systems,” United States Department of Commerce, Special Publication 862, May 1994.

6. U.S. Department of Transportation (USDOT), “Seismic Retrofitting Manual for Highway Bridges, (1995) ” Federal Highway Administration, Pub. No. FHWA-RD-94-052, May 1995.

7. Wehbe N., and M. Saiidi, (1999) "Effects of Confined Core on Seismic Vulnerability of Reinforced Concrete Column with Structural Flares," ACI Special Publication (SP-187), Seismic Response of Concrete Bridges, 1999.

8. Wehbe, N.I, M. Saiidi, and D. Sanders, (1997) "Effects of Confinement and Flares on the Seismic Performance of Reinforced Concrete Bridge Columns," Report No. 97-2, CCEER. Department of Civil Engineering, University of Nevada, Reno, September 1997.

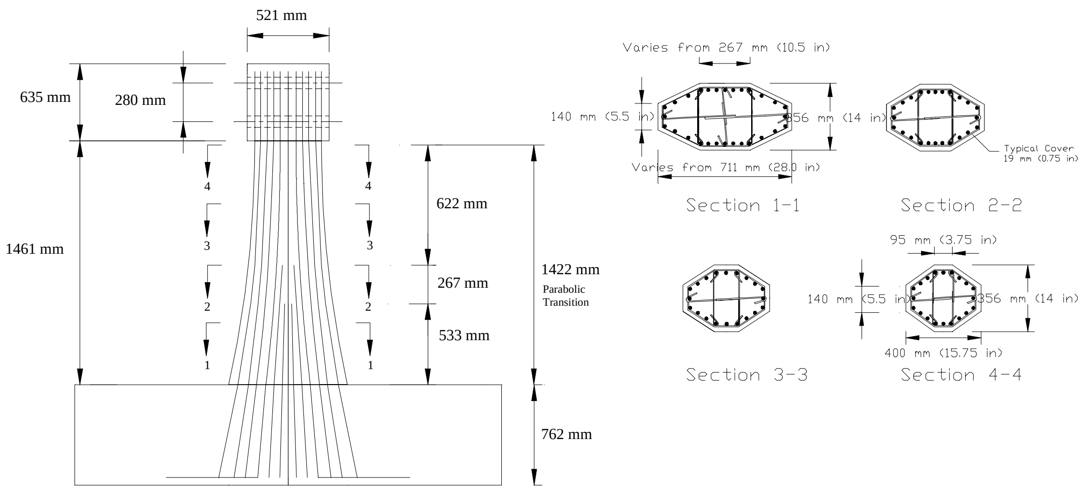

Fig. 1 – Elevation View of the As-Built Specimen

Glass Fiber-Epoxy Jacket

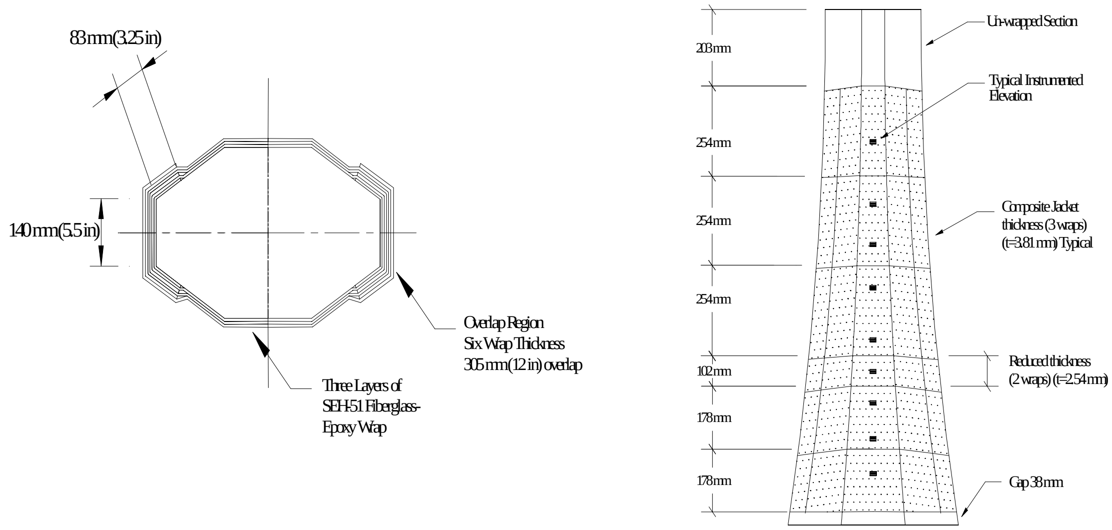

Fig. 2 – Details of Jacket in Specimen FRG

Fig. 3 – The Failure of Specimen FA

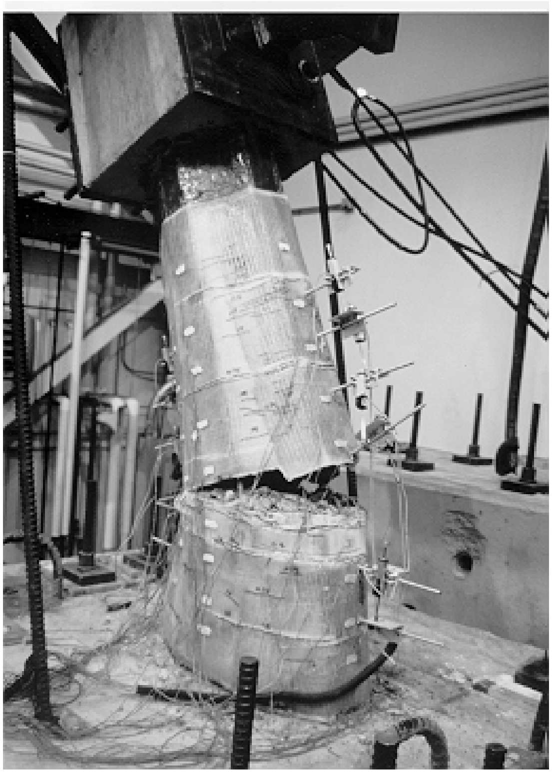

Fig. 4 - The Failure of Specimen FRG

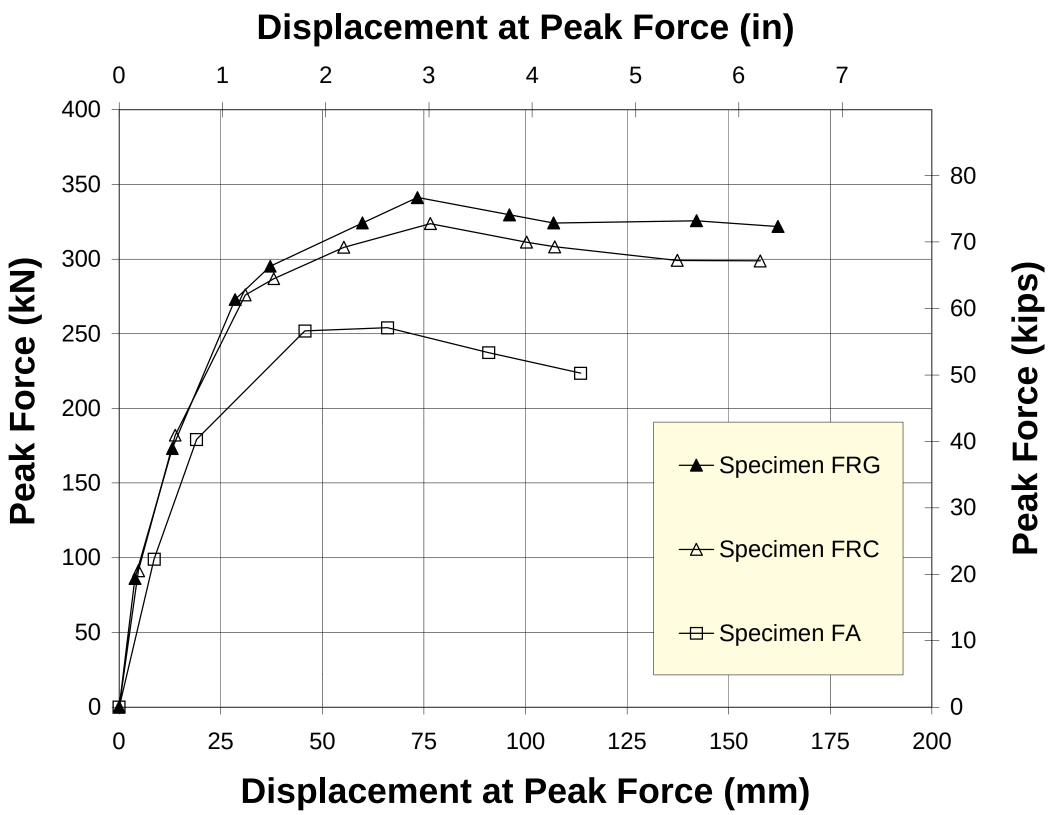

Fig. 5 – The Load-Deflection Envelopes for All Specimens

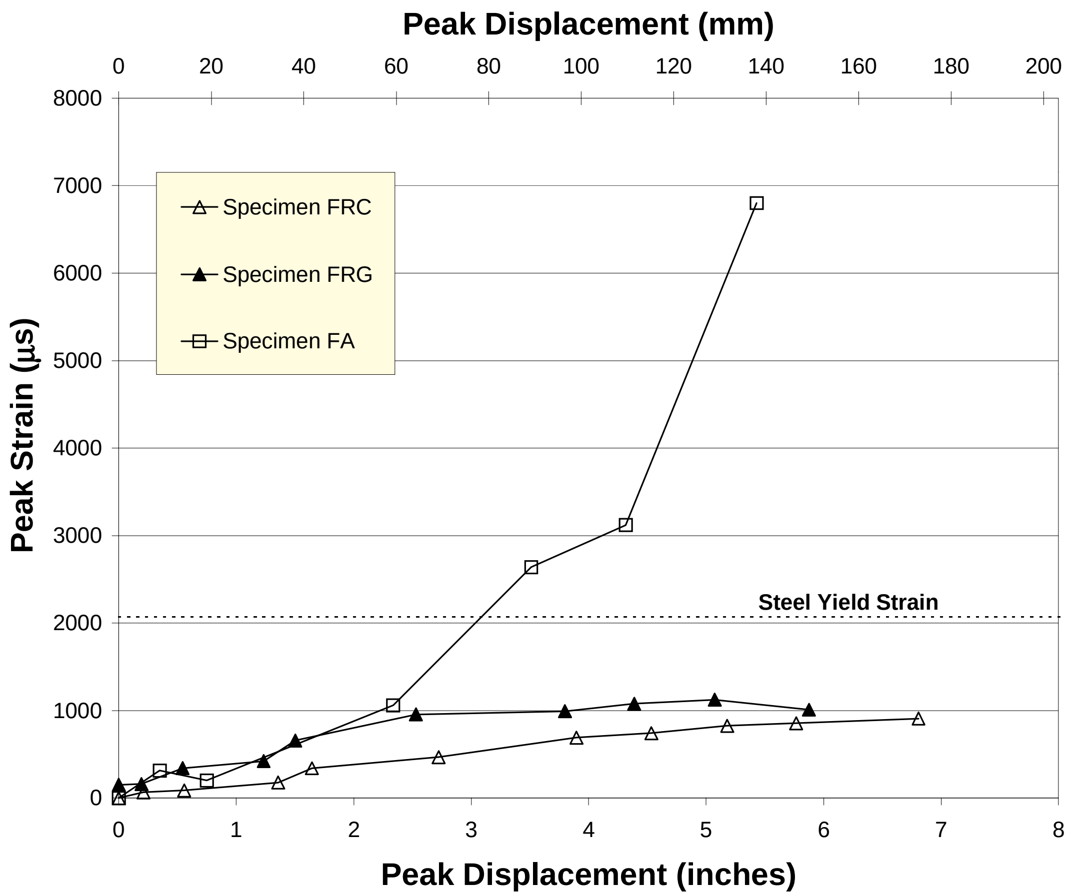

Fig. 6 – Tie Bar Strains in the Vicinity of the Plastic Hinge