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Steel Jacket Retrofit For Bridge Columns With Structural Flares

M. Saiid Saiidi[1], Nadim Wehbe[2], David Sanders[3], and Cory Caywood[4]

ABSTRACT

Four 0.4-scale flared reinforced concrete bridge columns were tested to examine the as-built seismic behavior of the columns in a 16-span bridge in Reno, Nevada, and to determine the effectiveness of steel jacket retrofit. Because the bridge was built well after the 1971 San Fernando earthquake, a relatively large amount of lateral steel was used in the columns. However, analysis showed that some of the columns may be susceptible to shear failure under strong earthquakes. A steel jacket retrofit with a predetermined gap location to force the plastic hinge at that location was found to be effective in reducing shear distress in the columns.

INTRODUCTION

The catastrophic failure of the Mission-Gothic Bridge during the 1994 earthquake in Northridge, California, revealed the high shear forces that can develop in flared columns (Hall 1994). The flares in this bridge were provided with a nominal amount of reinforcement and were intended to be nonstructural. The primary steel was placed in the core in a circular pattern and was confined by a spiral. The nonstructural flares increased the moment arm for the primary steel substantially thus increasing the moment capacity at the flare. The plastic hinges were hence shifted to the end of the flares, thus reducing the shear span. The lower shear span led to a considerable increase in shear and failure of the columns.

A large number of bridges in Northern Nevada are supported on flared columns. In all cases the flare reinforcement is substantial. The columns in newer bridges utilize a reinforced core with spirals in addition to the flare reinforcement. All the bridges have been designed and built in or after 1981. A study was undertaken to evaluate the seismic vulnerability of these bridge columns. Based on an analytical study it was concluded that columns with a confined reinforced concrete core have sufficient shear capacity with considerable safety margin against shear failure. However, columns with steel only in the flares were found to be potentially susceptible to shear failure. A testing program to develop and assess retrofit schemes for these columns was undertaken. The purpose of the study was to design, construct, and test retrofit details that would enhance the margin of safety against shear failure in reinforced concrete bridge columns with structural flares. Both steel jacket and fibrous composites were considered. This article presents a summary of the study and the results for columns that were retrofitted with a steel jacket. Details of the study are presented in (Wehbe et al. 1997 and Caywood et al. 1999).

EXPERIMENTAL PROGRAM

The columns of a 16-span freeway viaduct supported on 94 flared columns was the focus of the study. The flare geometry is the same in all the columns, but the total column height varies. All the columns have a two-way hinge detail at the base and are monolithically connected to a solid diaphragm built into a multi-cell box girder superstructure. At the widest section, the longitudinal steel ratio is 1, 1.4, and 1.8 percent in different columns. Typical piers in the bridge have three or four columns. A push-over analysis of the more critical piers was conducted and the shear demand for the development of full plastic hinges was determined. Because the flares are gradual, the potential location of plastic hinge could not be established without an analysis. The moment curvature relationship was determined at multiple sections along the column height and, based on the demand moment diagram, the plastic hinge location was established. Plastic hinging took place over a relatively large portion of the

[1]Professor, Dept. of Civil Engineering, University of Nevada, Reno, NV 89557

[2]Asst. Professor, Dept. Of Civil Engineering, S. Dakota State University, Brookings, SD 57007

[3]Assoc. Professor, Dept. of Civil Engineering, University of Nevada, Reno, NV 89557

[4]Staff Engineer, CH2M Hill, Seattle, WA

columns because of the gradual flares and the fact that the columns were pinned at one end. At the plastic hinge, it was found that shear demand and capacity (without strength reduction factor) were nearly the same in some of the columns, indicating the need for retrofit to provide a margin for shear capacity.

Two 0.4-scale specimens representing the as-built columns with 1 and 1.8 percent steel (specimens LS and HS, respectively) were constructed (Fig. 1). Note that, unlike typical pre-1971 bridge columns, there was a large amount of lateral steel in the columns. The transverse steel amount met the current code requirement for confinement. The specimens were built in an upside-down position and were subjected to quasi-static cyclic displacements of increasing amplitudes in the strong direction (Fig. 2). Based on the observed response of the as-built columns, it was decided that the primary objective of the retrofit would need to be a reduction in tie bar shear stresses and shear cracks.

Two more specimens similar to LS and HS were built. Specimen R-LS (retrofitted LS) had a multi-segment oval steel jacket separated from the column with a layer of Styrofoam to minimize the flexural enhancement provided by the jacket. Test results showed that tie stresses reduced considerably. However, the jacket shifted the plastic hinge closer to the end. Because this shift could create problems at the connection region, a gap was placed in the steel jacket in the next specimen, R-HS (Fig. 5) at 762 mm above the footing to control the plastic hinge location. No Styrofoam sheets were used between the jacket and the column in R-HS. The jacket in both specimens was designed to resist shear. A factor of safety of two was applied to the shear corresponding to the plastic hinging of the column. The shear capacity was assumed to be provided by concrete, transverse steel, and the steel jacket. The FHWA (Federal Highway Administration) provisions in the retrofit manual were used (FHWA 1995). The required jacket thickness for shear was 1.5 mm, and it also satisfied the confinement requirement. To facilitate handling, installation, and welding, the thickness was increased to 4.8 mm. A large number of strain gages, load cells, and LVDTs' (linear variable differential transformers) were installed in each specimen to measure longitudinal and transverse bar strains, curvatures, axial load variation, and the jacket strains. Specimens R-LS and R-HS were subjected to the same displacement history as that of the as-built specimens.

TEST RESULTS AND ANALYSIS

Results of the tests on the as-built specimens indicated that flexure dominated the mode of failure in both. The displacement ductility at failure was 8 for LS and 6 for HS. Both specimens exhibited good energy dissipation capacity, and strength degradation was small. However, the ties yielded in both specimens, and there was extensive shear cracks at 45 to 60 degrees relative to the column axis. Figure 3 shows specimen HS at the end of the test. The failure load was controlled by the buckling of the longitudinal bars and low-cycle fatigue failure of these bars. Note the extensive shear cracking and the relatively large plastic hinge length. The extent of tie bar yielding was limited in LS but was substantial in HS. Figure 4 shows the maximum perimeter tie bars strains at different displacement ductilities in HS.

As expected the measured load deflection response for the retrofitted specimens showed good energy dissipation. There was little strength degradation. The envelope of load-displacement responses for HS and R-HS are shown in Fig. 6. Note that the initial stiffnesses were nearly the same. This was contrary to the expectation because the steel jacket and the grout in the enlarged oval section would normally increase the stiffness. The concrete compressive strength on the day of testing for HS and R-HS was 51.7 MPa and 42.3 MPa, respectively. Assuming that concrete stiffness is proportional to the square root of the compressive strength, the stiffness of R-HS without the steel jacket would be 10 percent lower than that of HS. The addition of the steel jacket and the grout would increase the stiffness by 10 to 20 percent some of which would be offset by the reduced concrete strength. Therefore, a slight net increase in the initial stiffness was still expected, but did not materialize in R-HS. The initial stiffness of R-LS was also very close to that of LS. However, the concrete strength in R-LS was 35 percent lower than that in LS accounting for close to 20 percent reduction of stiffness. The retrofit in R-LS made up for the lower concrete stiffness thus leading to the same initial stiffness as that of LS.

Figure 6 also indicates that the strength continued to increase after yielding in R-HS, whereas the strength of HS remained constant. A similar trend was observed in R-LS and LS. The retrofit did not increase the displacement ductility beyond that of the as-built specimens. Specimens R-LS and R-HS failed at ductility of 8 and 6, respectively. These limits were the same as those measured for the as-built specimens. However, the objective of the retrofit, which was to reduce stresses in the lateral steel, was accomplished. Figure 3 shows the reduction in the maximum tie bar strains at different ductilities as a result of retrofit. At the low ductility level of 2, the strains in the as-built and retrofitted specimens were small and were nearly the same because shear cracks had not been yet formed and concrete was resisting most of the shear. At ductility of 4, the maximum strain in the retrofitted specimens was 57 percent of that in the as-built column. The tie bars in the retrofitted column reached a maximum of 75 percent of the yield strain at ductility of 6, whereas the ties in HS were well past the yield point. A more detailed analysis of the columns and the effectiveness of the retrofit is presented in (Caywood et al. 1999).

A lateral load-deflection analysis of the retrofitted columns with a gap in the steel jacket was conducted to determine if ductility could be improved by adjusting the location of the gap. In this model, deflections due to flexure, shear, and bond slip were included. The effectiveness of the jacket as vertical reinforcement was also studied. Because only partial composite action was expected between the jacket and the column, a lower and an upper bound analysis was performed. The actual response of R-HS indicated that the jacket was 47 percent effective compared to a fully composite jacket. Several analyses were followed assuming partially effective jacket using different gap locations. It was found that the ductility capacity was not sensitive to the gap location and jacket with a gap in the range of 380 to 760 mm above the footing would lead to a displacement ductility of approximately 6.5. The optimized gap location was found to be at 500 mm from the footing in the specimen corresponding to 1.25 m distance from the soffit in the actual bridge.

CONCLUSIONS

The plastic hinge location in reinforced concrete columns with structural flares has to be determined from a moment-curvature analysis of multiple sections along the column heights before the location of plastic hinge can established. In the columns studied in this research plastic hinging took place over a relatively large portion of the columns because of the gradual flares and the fact that the columns were pinned at one end. Because of the relatively large amount of lateral steel in the columns, the ductility capacity of the as-built specimens was relatively high. However, the high strain in the tie bars indicated little margin against shear failure. The addition of steel jackets reduced the strains substantially and was effective. However, no improvement in the ductility capacity was observed because the as-built columns were relatively ductile even without the jacket. The addition of a continuous steel jacket shifted the plastic hinge closer to the ends causing some concerns for connections. The introduction of a gap in the steel jacket to force the plastic hinge in a predetermined location was effective and did not adversely affect the ductility capacity. Attempts to optimize the gap location revealed that the ductility capacity was insensitive to the gap location. Therefore, the criterion to determine an appropriate gap location would be to place the gap sufficiently away from the connection to avoid spread of plastic hinge to the joint.

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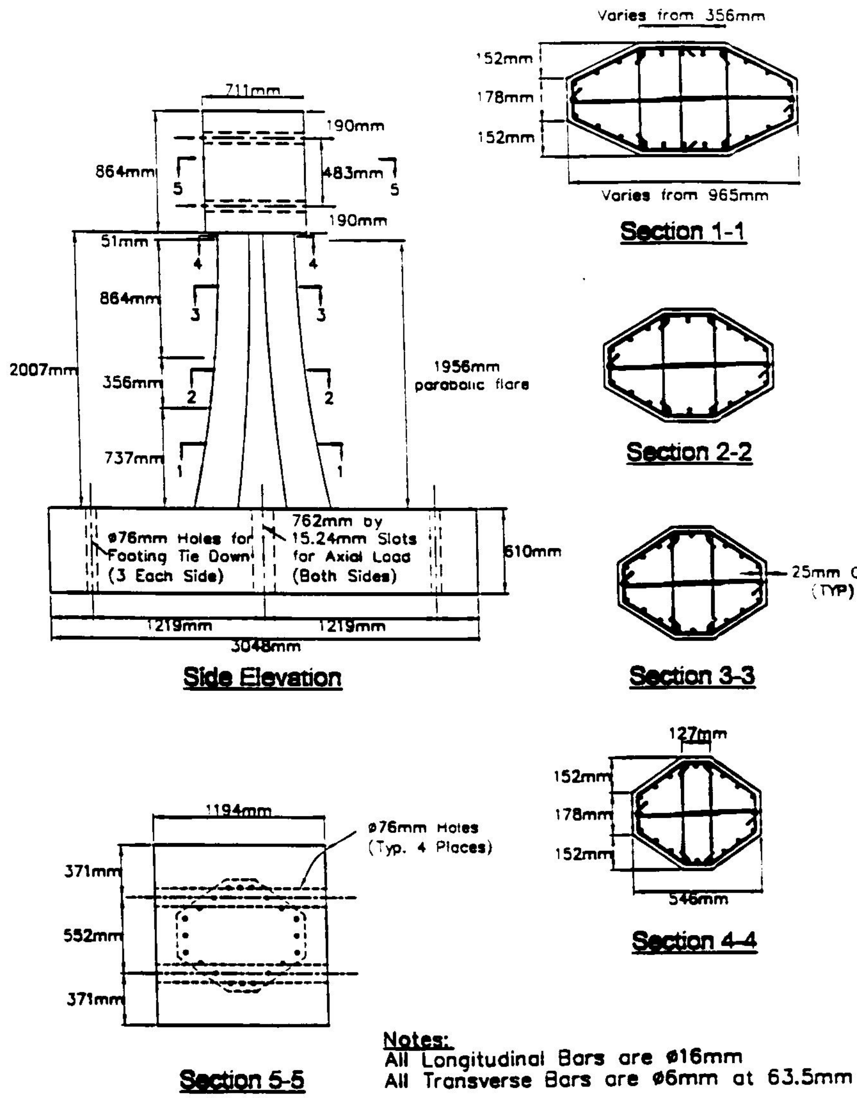

Fig. 1 - Details of as-built specimen HS

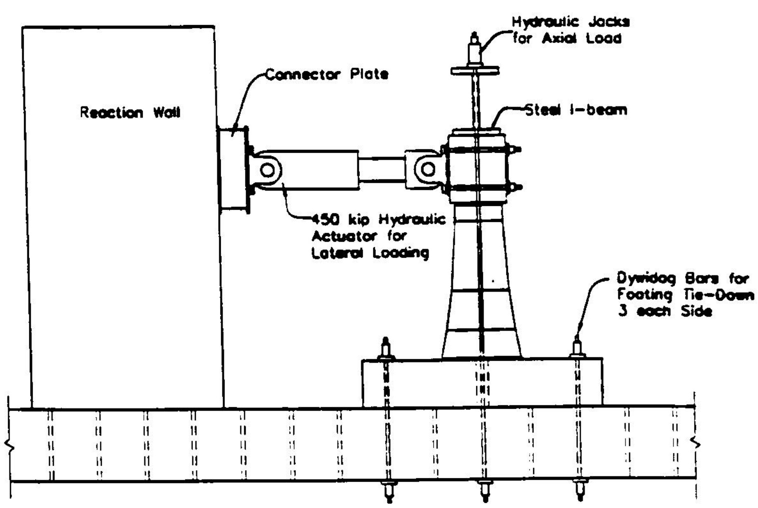

Fig. 2- Test set up

Fig. 3 -Specimen HS at the end of the test

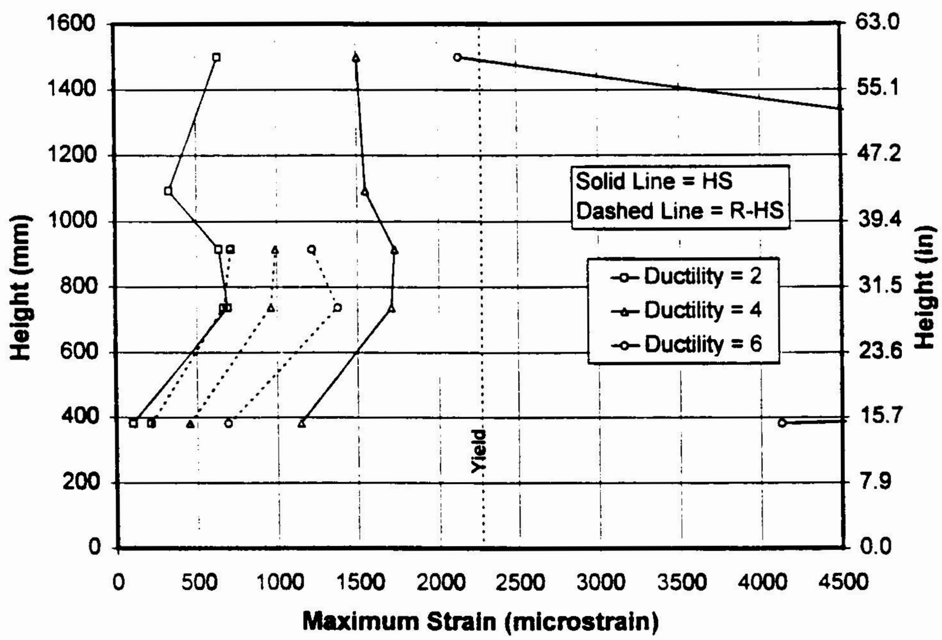

Fig. 4 -Perimeter tie bar strains at different displacement ductilities in HS and RHS

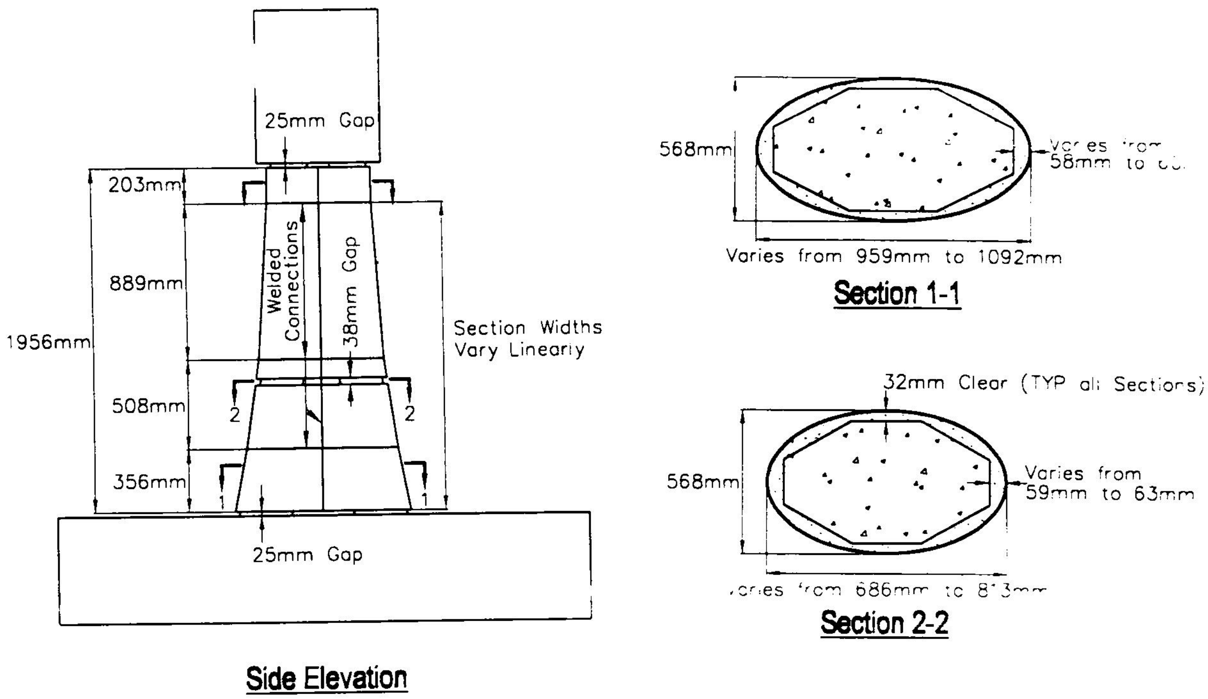

Fig. 5- Retrofit detail for specimen R-HS

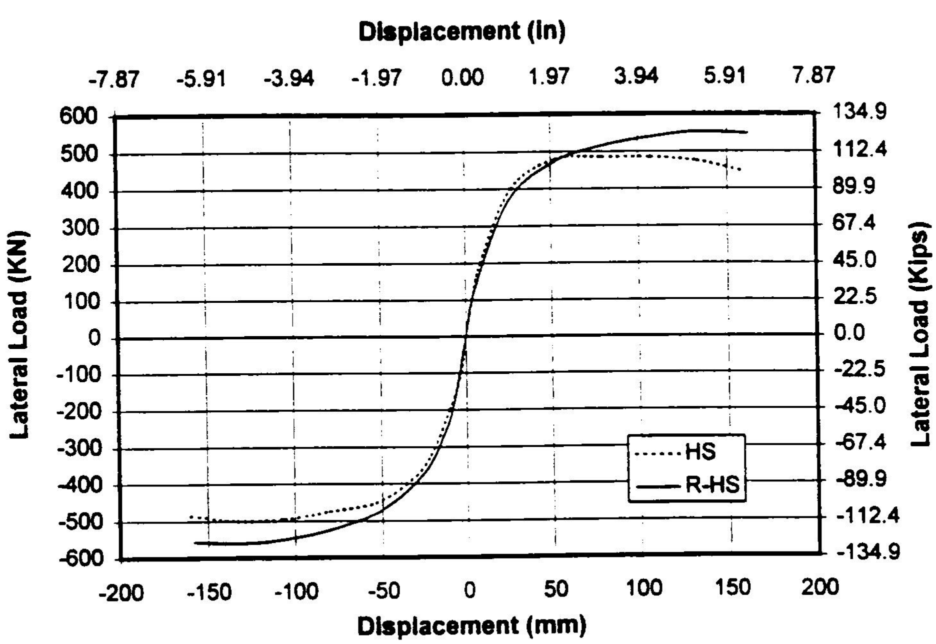

Fig. 6- Comparison of the response envelopes for HS and R-HS