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ANALYSIS and RETROFIT of FIXED FLARED COLUMNS with GLASS FIBER-REINFORCED PLASTIC JACKETING

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ABSTRACT

The main objectives of this study were to develop and analyze a seismic retrofit method for flared columns that are fixed at both ends and consist of a structural flare. The columns of Bridge I-1556 located in Sparks, Nevada, were used in the study. A previous study of the columns with respect to the seismic requirements had shown that the confinement steel is inadequate and the shear capacity near the base of the columns is marginal. Both of these deficiencies were addressed by using a glass fiber-epoxy jacket. Because the columns are flared, a continuous wrap over the flared segments is not appropriate. Therefore, for the flared segments, a series of overlapping straps were recommended. The FHWA and ACI procedures were used in design of the composite jacket. To determine the effectiveness of the retrofit, the as-built and retrofitted columns were analyzed for earthquakes loading using a nonlinear response history analysis computer program called RC-Shake. This program accounts for stiffness and strength degradation of reinforced concrete elements under cyclic loads. The 1994 Sylmar-Northridge earthquake record was used. It was found that the retrofit reduced the displacement ductility demand by 50 percent under an earthquake with PGA of 1.2g.

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Chapter 1

Introduction

1.1 Introductory Remarks

The seismic design of reinforced concrete highway bridges requires a ductile response of the bridge columns. During an earthquake event, the bridge must be capable of large inelastic deformations without a significant reduction in capacity¹⁰. In reinforced concrete columns, the most critical type of failure to prevent is brittle failure at plastic hinges due to a lack of adequate confinement steel or inadequate shear capacity⁹. The prevention of shear failure is particularly important when large inelastic deformations occur because plastic hinging reduces the shear capacity of the column. The method used most often for enhancing ductility and shear strength of existing columns is steel jacketing. The use of advanced composite jacketing, typically called fiber-reinforced plastics (FRP), has become increasingly popular over the last 5 years for seismic retrofit of highway bridge columns.

During the January 17, 1994 Northridge earthquake, several highway bridge structures failed in non-ductile shear failure modes. One of these bridges was the SR-118 Mission/Gothic undercrossing that was supported on one-way flared columns. In California and Nevada, the use of one-way and two-way flared columns has been extensive in highway bridges. A series of studies involving an analytical and experimental assessment of seismic vulnerability of flared highway bridge columns and the development of retrofit methods was undertaken at the University of Nevada, Reno. This report considers Bridge I-1556 located in Sparks, Nevada which was previously identified in Ref. 18 as having a potential for shear failure and being deficient with respect to confinement steel.

1.2 Previous Work

Research pertaining to the seismic behavior of reinforced concrete highway bridge columns is quite extensive. Though flared columns are used extensively in bridges, only a few studies have considered their seismic performance. The discussion of previous work that follows is limited to the experimental studies that consider: (1) the seismic performance of flared bridge columns, and (2) the effects of FRP jacketing of reinforced concrete columns.

1.2.1 Flared Bridge Columns

Prior to the 1994 Northridge earthquake very little attention was paid to the seismic performance of flared columns. The shear failure of several columns with non-structural flares in the 1994 Northridge earthquake prompted research to determine their seismic response^{8,13}.

A preliminary analytical study of the SR-118 Mission/Gothic undercrossing was conducted by the National Institute of Standards and Technology (NIST)⁸ and the University of California, San Diego (UCSD)¹¹. The main longitudinal reinforcement in the columns was placed in a circular pattern over the height of the column. It was thought the column flares would spall off during severe lateral loading, thus not increasing the flexural capacity of the column. Both studies concluded that the flares contributed to the flexural strength of the columns, and that the supposed spalling of the flare would not occur even though the flares were “architectural.”

UCSD continued to investigate these columns with a study completed by Sanchez et al.¹³ that consisted of testing both as-built and retrofitted columns with architectural flares. The study suggested a retrofit method of decoupling the effect of the flared portion of the

column by cutting the flare at the soffit, leaving a gap between the flare and the soffit. This type of retrofit would not work in columns with structural flares in which the main bars follow the shape of the flares.

Webhe et al.¹⁷ investigated the seismic behavior of four highway bridges with flared columns located in northern Nevada. The one-way flared columns, in general, were constructed using two different reinforcement details. The first type of column is somewhat similar to those that failed during at the Mission/Gothic undercrossing in that they consist of longitudinal steel distributed in a prismatic circular pattern through the core of the column. However, unlike the columns in the Mission/Gothic undercrossing, these columns have heavily reinforced (longitudinal and transverse) flares. In the second type of columns, the primary longitudinal reinforcement follows the column flare without a core longitudinal reinforcement or spirals, and the transverse reinforcement consisted solely of lateral ties. A non-linear analysis was completed in this study that indicated the first type of column has sufficient shear capacity, and would respond well to seismic excitation. However, the second type of column clearly showed an insufficient shear capacity and confinement, indicating probable brittle failure in a seismic event.

The results of the analytical study led to phases two through four of research, which involved the testing of large-scale flared columns at the University of Nevada, Reno. In the second phase, Webhe et al. investigated two forty-percent scale as-built specimens subjected to quasi-static cyclic lateral loads. The results confirmed the need for the development of a retrofit system for the columns. Caywood et al.³ studied two additional forty-percent scale specimens that were retrofitted with steel jackets and tested under slow cyclic loads. This study showed the steel jackets were effective in retrofitting the test specimens.

Further testing of the flared columns under the response of more realistic dynamic loading was completed in the third and fourth phases of the UNR study. The third phase study was conducted by McElhaney et al.⁷, in which two thirty-percent scale specimens were tested. One specimen modeled the as-built columns while the other was retrofitted with a steel jacket. The purpose of the study was to verify the results of the first two studies and quantify the effects of the dynamic loading as compared to the quasi-static loading. The third study showed generally good correlation with the first two studies, while some variation existed due to the dynamic loading. The fourth study considered two FRP retrofitted specimens subjected to shake table loading⁶. This study showed that the FRP retrofit was as effective as steel jacketing in improving the seismic response.

1.2.2 FRP Jacketing of Highway Bridge Columns

The use of advanced composites for seismic retrofit has become an increasingly popular area of research over the last several years. Many studies have considered the effects of FRP jacketing on columns for confinement, flexural behavior, shear strength, and lap splice reinforcement. The research varies widely with respect to the type of composites and configurations used to install the jacketing. This section outlines some of the relevant research to this project relating to highway bridge columns retrofitted with advanced composites.

UCSD was one of the first research institutions to delve into the retrofit of reinforced concrete columns with advanced composites¹⁴. Seible and Priestley tested four rectangular cantilever columns. The specimens consisted of two retrofitted with a steel jacket and the other two with a glass fiber-epoxy jacket. The jackets were installed in the plastic hinge

region at the base of the column. The specimens were tested both in the weak and strong directions to determine the level of flexural increase experienced by the retrofit. The test results showed that the advanced composite and steel jackets performed very well, providing a displacement ductility that exceeded 8. The glass fiber-epoxy jackets showed slightly better confinement than the steel jackets, and were found to be a suitable retrofit method for reinforced concrete columns.

In a study at the University of Arizona, Jin et al.⁴ created a glass fiber jacket system that was used to retrofit reinforced concrete columns. The experimental study consisted of ten 0.2-scale, typical pre-1971 reinforced concrete columns, with five circular columns and five rectangular columns. Four as-built specimens and six retrofitted specimens with glass fiber composite jackets were tested. The as-built specimens had serious deficiencies in shear strength and lap splice detail. The results showed that the composite jackets greatly increased the column ductility capacity and shear resistance and prevented bond failure.

In another project, Saadatmanesh et al.¹² used a parametric study to present a method of using fiber-reinforced straps to increase the strength and ductility of reinforced concrete columns. The goal was to improve the seismic performance without increasing the flexural strength. The parametric study indicated that this method of strapping the column was effective in both increasing the shear capacity and in improving the column ductility.

Seible et al.¹⁵ considered both steel jacketing and carbon fiber tow jackets to retrofit prismatic reinforced concrete bridge columns. Four specimens were tested representing one as-built, one steel jacketed, and two carbon fiber jacketed specimens. The carbon-fiber jacketed specimens exceeded the measured displacement ductility and ultimate flexural

strength of the steel-jacketed specimens. The carbon-fiber specimens reached a ductility of nearly 10 at failure, while the steel-jacketed column reached a ductility of 8.

In the flared column studies at UNR, two thirty-percent one-way flared columns were retrofitted with FRP jackets consisting of one glass-fiber, epoxy-jacketed specimen and one carbon-fiber epoxy-jacketed specimen⁶. This work is the only case where both non-prismatic reinforced concrete bridge columns and advanced composite jackets were considered. This study showed that the advanced composite jackets increased the shear capacity of the column and improved its seismic response.

1.3 Objective and Scope

The primary objective of this study was to develop and analyze a seismic retrofit system for bridge columns with moment connections at both ends of the column, which incorporated structural flares. The seismic retrofit detail considered was a glass fiber-epoxy jacket to improve confinement and shear strength of the columns.

An analytical study was completed on a full-scale prototype one-way flared reinforced concrete bridge column used in Bridge I-1556 in Sparks, Nevada. Both as-built and retrofitted columns were subjected to a series of earthquakes to determine the effectiveness of the retrofit.

Chapter 2

Description of Bridge and Seismic Retrofit

2.1 Introduction

The purpose of this chapter is to present: (1) description of the bridge being studied and the deficiencies of the columns, and (2) design of FRP jacket for the bridge columns to address the deficiencies.

2.2 Bridge I-1556

2.2.1 Details of Bridge I-1556

Bridge I-1556 is a two-span plate-girder bridge located in Sparks, Nevada. It is an overcrossing located on I-80 at the Sparks Boulevard Interchange. The bridge was constructed in 1989 according to the 1983 AASHTO Standard Specifications¹. The superstructure consists of eight plate girders supporting a 241-mm thick concrete slab deck. The girders are identical and equally spaced at 3.96 m. The bridge is skewed at 11°.

Seat-type abutments at both ends support the bridge with one four-column bentcap in between. The columns are flared and are rigidly connected to both the footings and bentcap with a full moment connection in both the longitudinal and transverse directions. Figure 2-1 shows a view of one of the columns. Each column is supported by a group of four piles constructed of precast pre-tensioned concrete with a design load capacity of 1960 kN (441 kips) per pile.

The reinforcement detail is identical for all four columns. The longitudinal reinforcement ratio varies along the height of the column from 1.05 to 1.6 percent. The

longitudinal reinforcement follows the parabolic shape of the column elevation resulting in different cross sections and longitudinal steel ratios. The number of the longitudinal bars also varies with the height of the column.

Figures 2-2 and 2-3 show the column elevation and reinforcement details, respectively. Note that the lower 1.83 m (6 ft.) of the columns has constant cross section and is not flared. The main longitudinal reinforcement consists of 20 - $\phi 36$ mm (#11) bars that extend throughout the column height. In addition 12 - $\phi 23$ mm (#7) bars are placed along the upper flared segment of the column from 3.65 m to 6.7 m from the footing. The transverse reinforcement consists of lateral ties and crossties of $\phi 16$ mm (#5) bars at 102 mm vertical spacing.

The specified material properties for the steel and concrete are shown in Table 2-1. For the analysis in this study, the concrete strength was increased to reflect estimated increase in strength over time since the placing of the concrete. The concrete compressive strength of the prototype column was 27.6 MPa (4.0 ksi) in 1989. The estimated increase in strength over the ten years since placing of the concrete is 32.4 MPa (4.7 ksi).

A previous evaluation of the columns of Bridge I-1556 showed that plastic hinges would form at column ends because the columns are fixed at both ends¹⁸. The study also revealed that in flared columns with hinged base the plastic hinge would form at some distance from the top end of the column. With respect to the lateral steel, the study showed that the column ties in Bridge I-1556 are inadequate in terms of (a) confinement and (b) shear strength at the plastic hinge at the bottom of the columns. In the study Ref. 6, a method to apply FRP jackets on flared columns, was developed. In the next section both the

confinement and shear problems of the columns in Bridge I-1556 are discussed and the FRP design is described.

2.3 Design of FRP Jacket

The use of FRP jackets to retrofit highway bridge columns is becoming more accepted due to the high strength, light weight, and relative ease of application. The typical wrap consists of a fiber-reinforced plastic (FRP), which combines a fabric-reinforcing material with a bonding agent. The fabric-reinforcing material provides the composite wrap with its primary strength, while the bonding agent holds the reinforcing material together. The reinforcing material consists of uni-directional fibers which are arranged into a laminate by overlapping layers of reinforcing material in any direction to achieve the required strength.

The most common reasons for seismic retrofit of bridge columns using FRP jackets are to improve the shear capacity and confinement. The deficiencies of the columns and the retrofit design with respect to both of these issues are discussed in this section.

2.3.1 Design of FRP Jacket for Shear Strength Enhancement

The jacket was designed through consideration of the capacity and demand at the predicted plastic hinge location. The nominal shear capacity of the column (V_n) was calculated by including contribution from concrete (V_c), and the transverse steel (V_s), using the CALTRANS method². This method was chosen because it accounts for the reduced concrete strength capacity of the plastic hinge. The concrete shear strength includes the

effects of compressive axial stress, displacement ductility demand, and level of column core confinement. The shear demand was calculated based on moment capacity and location of the plastic hinges.

The jacket thickness was designed for the difference between the column shear capacity and demand. The jacket design shear strength is outlined in Eq. 2-1.

$$V_{Design} = \Omega_0 \cdot V_{Demand} - \phi \cdot (V_c + V_s) \quad (2-1)$$

This equation considers both an over-strength safety factor (Ω_0) and a strength reduction factor (ϕ) to account for the uncertainties of seismic loading effects and accurate estimation of shear capacity. The octagonal shape of the column was approximated as a circular column to account for the contribution of the jacket in a conservative manner. The Federal Highway Administration (FHWA) composite jacket design procedures were used to find the required thickness of the jacket¹⁶. The FHWA design equation was modified to only include passive confinement effects of the jacket. Equations (2-2) and (2-3) were used to derive the thickness of the jacket based on the jacket design shear, assuming a circular column relationship.

$$V_{Design} = \frac{\pi}{2} \cdot ((t_p \cdot E_p) \cdot \epsilon_p) \cdot D \cdot \cot(\theta) \quad (2-2)$$

$$t_j = \frac{2}{\pi} \cdot \left(\frac{V_{Design}}{(E_p \cdot \epsilon_p \cdot D \cdot \cot(\theta))} \right) \quad (2-3)$$

Where: t_p = Thickness of FRP jacket (passive component)

E_p = Young's modulus of FRP jacket (passive component)

ε_p = Design strain of FRP jacket (passive component)

D = Effective depth of FRP jacket

θ = Design shear crack failure inclination (assumed to be 45 degrees)

The FHWA method models the jacket as hoop reinforcement with the jacket area equivalent to the thickness of the jacket with a spacing height of unity (1 mm or 1 in). The thickness of the jacket (t_j) is calculated using Eqs. (2-1) through (2-3), given the plastic hinge location and composite wrap properties (Table 2-3). The calculated thickness must be rounded up to the typical minimal thickness of the installed jacket including epoxy therefore providing additional strength beyond that of the original design.

The jacket was designed to provide additional shear capacity near the footing at the predicted plastic hinge region. An overstrength factor of Ω_0 from 1.5 to 2 has been used. In this study a value of 1.8 was used. The strength reduction factor was assumed to be $\phi = 1$. The material properties assumed for the jacket are shown in Table 2-2.

The glass fiber-epoxy has a typical layer thickness of 1.27 mm. Based on Eq. (2-1) through (2-3) a thickness of 12.5 mm was found which required ten layers of the glass fiber-epoxy wrap. Reference 18 showed that the inflection point of the columns under the ultimate condition will be at 1.83 m (6 ft.) from the footing. The jacket was designed to reinforce the prismatic portion of the column from the footing to 1.83 m (6 ft.) above the footing in two sections of 0.91 m (3 ft.). In the prismatic portion of the column, the jacket can be wrapped continuously around the perimeter of the column.

2.3.2 Design of FRP Jacket for Confinement

Several confinement steel design procedures are available for reinforced concrete columns. Four sets of design provisions specified in the following codes were used to design the confinement steel. These codes were: the American Concrete Institute (ACI), the American Association of State Highway and Transportation Officials Load and Resistance Factor Design (AASHTO-LRFD), the California Department of Transportation (Caltrans), and the Applied Technology Council panel 32 (ATC-32). The tie bar configurations and the size were kept the same as those in the actual columns (Fig. 2-3). The codes were used to determine the required spacing and to evaluate the extent of deficiencies of the column ties with respect to confinement requirements.

Table 2-3 lists the required tie spacing for different codes at different cross sections that are shown in Fig. 2-3. The ties running in the narrow direction of the column controlled the design for Sec. 1-1. The spacing for Sec. 2-2 and 3-3 was controlled by the ties in the wide direction of the column section. It can be seen that the AASHTO code requires the smallest spacing (the highest amount of confinement steel), while the requirements for the other three codes are similar. The actual spacing of the column ties is 100 mm (4 in.).

According to ACI, only Sec. 1-1 would require retrofit. The Caltrans and ATC-32 indicate that the existing confinement steel is sufficient and no retrofit is necessary. Using the AASHTO requirements and the design properties for glass FRP shown in Table 2-2, the number of wraps were determined. The effectiveness factor of the composites for the column was assumed to be 0.75 because the column cross section is non-circular. The confinement provided by the existing ties was taken into account. It was found that the columns would require 13 wraps in Sec. 1-1 (plastic hinge at the top of the column) and 5

wraps elsewhere. Based on the ACI code, the number of wraps for Sec. 1-1 would be only 4. The reason for the difference between the AASHTO and ACI results is in the coefficient of a lower bound confinement steel requirement in both codes. The ratio of the tie bar area and spacing according to the AASHTO version of this equation is:

$$A_{sh}/s = 0.12 h_c (f'_c/f_y) \quad (2-4)$$

Where h_c = core section dimension perpendicular to the ties

This equation is an older version of the ACI code formula. In the current ACI code, the coefficient has been reduced to 0.09, thus resulting in a 25 percent reduction. Because a good portion of the required confinement is already provided by the existing ties, the 25 percent difference in the total confinement requirement translates into a large difference in the number of composite wraps that are required for the two codes.

Considering that the ACI results are similar to those of Caltrans and ATC-32 in addition to the fact that these codes are more up to date, it is reasonable to conclude that the appropriate number of wraps to provide confinement in Sec. 1-1 is four and that no wraps are needed elsewhere. As was shown in Sec. 2.3.1, however, Sec. 3-3 needs to be strengthened by 10 wraps due to its low shear capacity.

2.3.3 Combined FRP Jacket Design

Based on Sec. 2.3.1 and 2.3.2, it is recommended that 4 layers of glass fiber composites be placed over the entire height of the column plus an additional 6 layers placed over the lower 1.83 m (6 ft.) of the columns. All the layers in the lower 1.83 m (6ft.) can be

wrapped continuously. The installation on the rest of the column height should be in the form of partial straps similar to those used in Ref. 6 for use on the columns of Bridge 1250 (the Airport Viaduct) in Reno, Nevada. Figure 2-4 shows the composite jacket. Note that a 50-mm (2-in.) gap is recommended at both column ends.

The shear capacity of the column with and without the retrofit jacket is shown in Fig. 2-5. In this graph the shear capacity of the 4-layer composite jacket over the upper 4.9-m of the column is included, even though confinement and not shear controlled the FRP design. Nonetheless, the presence of the 4-layer jacket will enhance the shear capacity. It can be seen in the figure that the FRP jacket increased the margin against shear failure considerably.

Chapter 3

Analytical Procedure and Results

3.1 Introduction

The as-built and retrofitted columns described in Chapter 2 were subjected to the Sylmar Hospital earthquake to determine the effectiveness of the retrofit. This chapter describes the analytical procedure and presents the results.

3.2 Analysis Procedure

One column was used in the analysis assuming that its ends were fixed against rotation. The analysis was done in the strong direction of the columns. The actual piers have four columns. Under lateral loads, column axial forces vary because of the overturning effects. The increase in the axial load increases the moment capacity and the associated column shear. This effect was account for in analyzing both the retrofitted and as-built columns.

The analysis consisted of two parts both taking into account the nonlinearity of the columns. First a static pushover analysis was completed to find the basic load-displacement relationship for each column. In the second analysis, the basic load-displacement properties were used in a nonlinear dynamic of the columns subjected to ground motions. This section describes each analysis and the results.

3.2.1 Pushover Analysis

Computer program RCMC was used to determine the moment curvature relationships for different cross sections¹⁹. From the pushover analysis the dynamic response of the column was calculated treating the column as a single-degree-of-freedom (SDOF) system. The confined concrete properties were used in the analyses assuming Mander's model. Also included was the increase in the flexural strength due to the transverse strength of the FRP jacket. This was done despite the fact that there is a small gap at the end of the composite jacket and that, theoretically, the plastic moment should form in the gap region. In reality, however, the plastic hinge is distributed over a certain length that extends into the composite jacket. Past studies have shown that the transverse capacity of the composite, even though it is relatively small, contributes to the flexural strength of columns⁶. Primarily the epoxy matrix controls the strength properties of the FRP sheets in the transverse direction. Table 2.2 lists the values used in the analysis. The axial load was taken equal to 6750 kN (1515 kips) based on a previous push-over analysis of the bent¹⁸. The results from RCMC were idealized by a series of bilinear relationships to facilitate the push-over analysis. Figure 3-1 shows the RCMC results and the idealized curves.

The analysis of the as-built column was conducted taking into account the moment and curvature variations along the column height. The initial stiffness and the load associated with plastic hinge formation at both ends of the column were calculated. Based on past shake-table performance of flared column models, it was assumed that the as-built column strength degradation starts at a displacement ductility of 5. In absence of an established method to estimate the descending slope, it was assumed that this slope is the average of the initial and post-yielding stiffness of the column. The load-displacement

relationship for the as-built column is shown in Fig. 3-2. The coordinates of different key points and the stiffnesses for different branches are listed in Table 3.1.

The load-displacement relationship for the retrofitted column was constructed by first assuming that the FRP jacket would increase the initial stiffness of the column by approximately 30 percent. This figure was based on past experience in FRP jacketed columns for which the increase in the stiffness has been reported to be in the range of 20 to 50 percent. The “yield” moment associated with point 1 in Fig. 3.1 was found assuming plastic hinges at both column ends and using the results from RCMC analysis. To determine the displacement at the start of strength degradation, the measured ductility capacity for FRP jacketed flared columns reported in Ref. 6 was used. It was assumed that the displacement ductility for this point is 7.5. Following a procedure similar to that used for the as-built column, the stiffness of the descending branch was assumed to be the average of the initial and post-yielding stiffnesses. The load-deflection relationship and the numerical values for the curve are shown in Fig. 3.1 and Table 3.1, respectively.

3.2.2 Dynamic Analysis

The computer program RC-Shake was used to analyze the equivalent SDOF system representing each column subjected to ground motions. An in-depth discussion of the parameters of RC-Shake can be found in Ref. 5. The 1994 Northridge earthquake as measured at the Sylmar Hospital was applied to the SDOF system by adjusting the peak ground acceleration (PGA) of the motion original motion from 100 to 250 percent. The measured PGA was 0.6 g. This meant the PGA used in the analysis was 0.6g to 1.5g. The viscous damping was assumed to be 5 percent. The mass was the tributary dead load for

each column. The computer program incorporates a modified version of the Q-Hyst model to account for the variation of the force and displacement. The model takes into account stiffness and strength degradation.

Figures 3-3 to 3-6 show the displacement histories of the top of the as-built and retrofitted columns for different earthquake amplitudes. The maximum displacements, ductilities, and drift ratios are listed in Table 3-2. Because the yield displacements of the two columns were nearly the same (Table 3-1), the effect of retrofit on all three parameters listed in Table 3-2 was nearly the same. It can be seen that under 1xSylmar earthquake neither column yielded. However, the peak displacement of the retrofitted column was approximately 30 percent less than that of the as-built column. The as-built column reached a ductility of 1.3 under the 1.5xSylmar whereas the retrofitted column still remained elastic. Under 2xSylmar the ductility demand in the retrofitted column was only one-half of the as-built column. The displacement history in Fig. 3-6 shows that when the earthquake amplitude was increased to 2.5xSylmar (PGA= 1.5g), the as-built column became unstable whereas the retrofitted column reached a moderate ductility demand of 2.5.

The hysteresis curves for the four earthquake runs are shown in Figs. 3-7 to 3-10. The linear behavior of both columns under 1xSylmar can be seen in Fig. 3-7. Figure 3-9 shows the considerable difference in energy dissipation demand for the two columns through the large area within the hysteresis loops for the as-built column versus the relatively small area for the retrofitted one. Finally, the unstable behavior of the as-built column under 2.5xSylmar can be observed in Fig. 3-10.

The aforementioned results clearly demonstrate the effectiveness of the retrofit jackets. The composite jacket improved the shear strength and confinement and was able to withstand an earthquake with a PGA of 1.5g with only a moderate ductility demand.

Chapter 4

Summary and Conclusions

4.1 Summary

The main objectives of this study were to develop and analyze a seismic retrofit system for the columns of Bridge F1556 located in Sparks, Nevada. The bridge pier consists of four one-way flared columns with moment connections at both ends of the column. A previous evaluation of the bridge columns with respect to the seismic requirements had shown that the confinement steel is inadequate and the shear capacity near the base of the columns is marginal. Both of these deficiencies were addressed in developing a retrofit strategy. The seismic retrofit detail considered in this study was a glass fiber-epoxy jacket. Because the columns are flared, a continuous wrap over the flared segments is not appropriate, as it will lead to an undesirable increase in the flexural capacity of the columns. Therefore, for the flared segments, a series of overlapping straps was recommended. The FHWA and ACI procedures were used in design. The composite properties were based on the values reported by Fyfe and Associates for their glass FRP wraps. For the prismatic part of the column, a continuous wrap is appropriate. It was found that ten layers are necessary over the prismatic part of the columns and four layers elsewhere.

To determine the effectiveness of the retrofit, the as-built and retrofitted columns were analyzed for earthquakes loading using a nonlinear response history analysis computer program. The moment-curvature relationships for the columns were determined by using a moment-curvature analysis program called RCMC. Program RC-Shake was used to determine the response histories. This program accounts for stiffness and strength

degradation of reinforced concrete elements under cyclic loads. The 1994 Sylmar-Northridge earthquake record was used. The peak amplitude of the ground acceleration (PGA) was varied from 0.6g (1xSylmar) to 1.5g (2.5xSylmar). It was found that the retrofit reduced the displacement ductility demand by 50 percent under an earthquake with PGA of 1.2g. Under the 2.5xSylmar record (PGA= 1.5g), the as-built column became unstable whereas the retrofitted column remained stable and had a moderate ductility demand of 2.5.

4.2 Conclusions

- The analytical study of the columns of Bridge I-1556 showed that the columns were vulnerable with respect to shear in the lower part of the columns when subjected to loads in the plane of the pier. The columns were also deficient with respect to the current confinement steel requirements.
- Four layers of glass fiber reinforced plastic wraps are needed over the entire height and additional six layers are needed in the lower part of the columns. These numbers are based on fibers with properties shown in Table 2-2.
- The FHWA shear design and ACI confinement steel design provisions were found to be appropriate for the columns.
- The nonlinear dynamic analysis of the as-built and retrofitted columns showed substantial improvement in the seismic behavior of the columns.

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Tables

Table 2 - 1 Bridge I-1556 Material Properties

	Specified	Estimated Actual
Concrete Compressive Strength	27.6 MPa (4 ksi)	32.4 MPa (4.7 ksi)
Steel Yield Stress	414 MPa (60 ksi)	414 MPa (60 ksi)

Table 2 - 2 Composite Jacket Properties

	DESIGN	SPECIFIED
Tensile Modulus	20,700 MPa (3,000 ksi)	27,600 MPa (4,000 ksi)
Ultimate Strain	0.006	0.02
Ultimate Tensile Strength	124 MPa (18 ksi)	552 MPa (80 ksi)
Strength at 90 degrees	34 MPa (5 ksi)	34 MPa (5 ksi)
Laminate Thickness	13 mm (0.51 in.)	13 mm (0.51 in.)
Modulus at 90 degrees	2,600 MPa (375 ksi)	2,600 MPa (375 ksi)

Table 2 - 3 Confinement Steel Spacing Requirements, mm (in.)

	ACI	AASHTO	ATC-32	CALTRANS
Section 1-1	88 (3.5)	66 (2.6)	102 (4.0)	115 (4.5)
Section 2-2	105 (4.1)	79 (3.1)	101 (4.0)	111 (4.4)
Section 3-3	105 (4.1)	79 (3.1)	101 (4.0)	111 (4.4)

Table 3 - 1 Load-Displacement Properties

	As-Built	Retrofitted
Displacement at point 1 mm (in)	21.3 (0.84)	20.8 (0.82)
Force at point 1 KN (kips)	2887.0 (649.1)	3665.2 (824.0)
Stiffness for Branch 0-1 KN/mm (kips/in)	135.5 (774.0)	176.2 (1006.0)
Displacement at point 2 mm (in)	106.5 (4.2)	156.0 (6.1)
Stiffness for Branch 1-2	3.4 (19.3)	4.4 (25.2)
Stiffness for the Descending Branch	-69.5 (-396.6)	-90.3 (-515.7)

Table 3 - 2 Maximum Displacements and Ductilities

	Peak Displacement, mm (in)			Displacement Ductility			Drift Ratio %		
	As-built	Retrofit	% Reduction	As-built	Retrofit	% Reduction	As-built	Retrofit	% Reduction
1.0 x Sylmar	17.4 (0.7)	12.4 (0.5)	29	0.8	0.6	27	0.3	0.2	29
1.5 x Sylmar	33.2 (1.3)	18.6 (0.7)	44	1.6	0.9	43	0.5	0.3	44
2.0 x Sylmar	56.5 (2.2)	26.3 (1.0)	53	2.7	1.3	52	0.8	0.4	53
2.5 x Sylmar	147.5 (5.8)	52.2 (2.1)	65	6.9	2.5	64	2.2	0.8	65



Figures

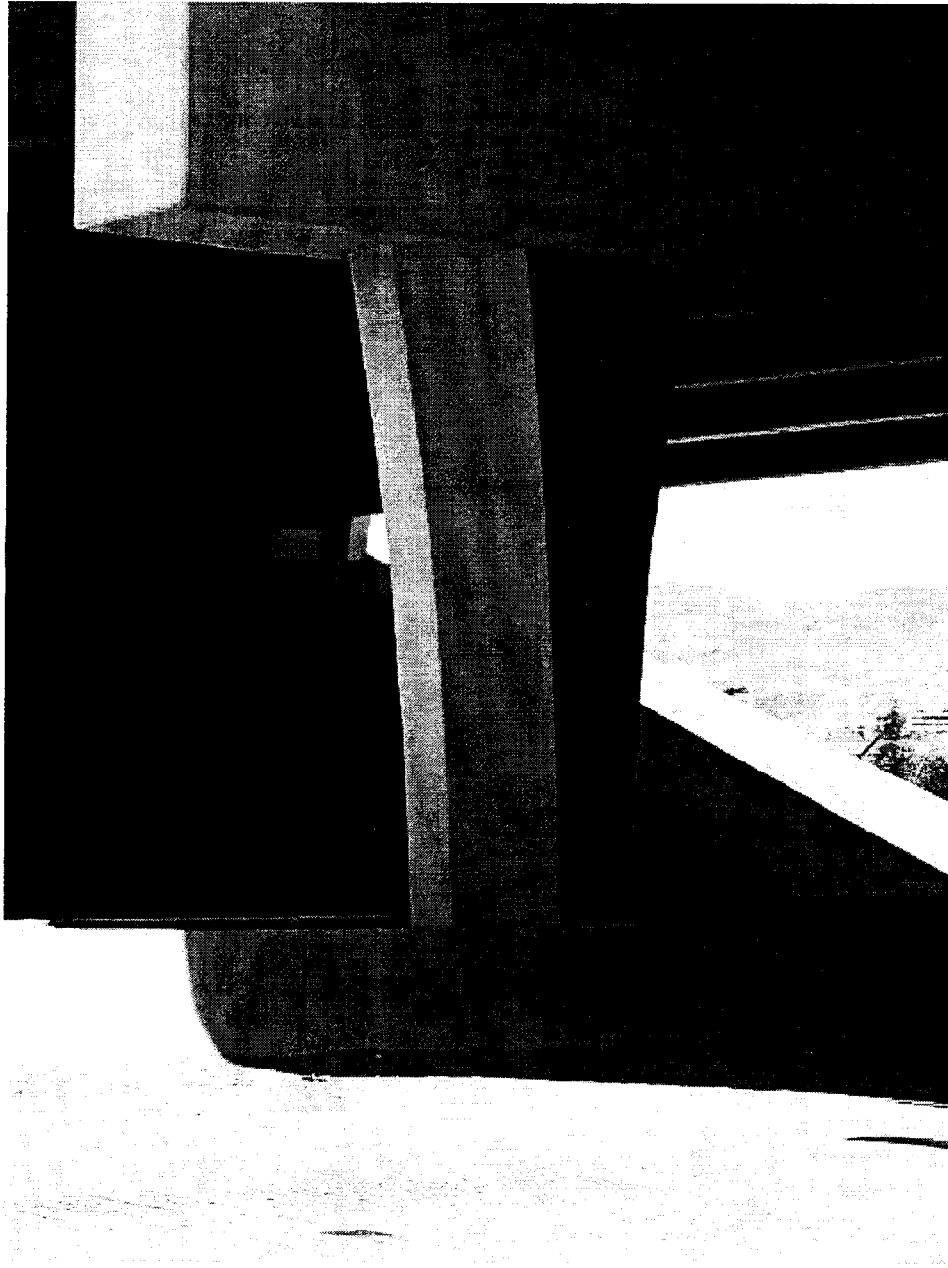


Figure 2 - 1 A View of the Edge Column in Bridge I-1556

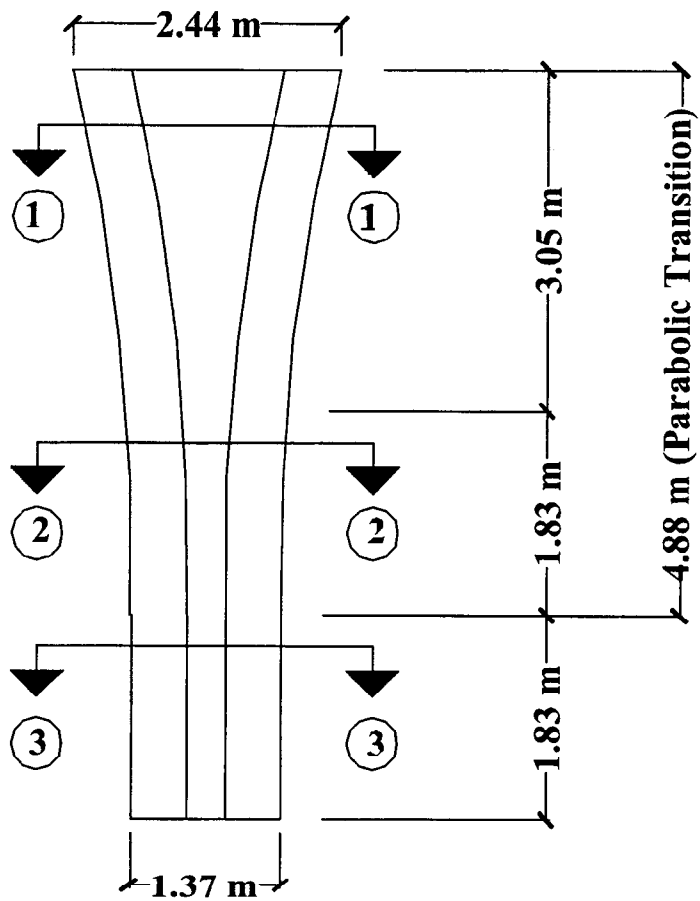
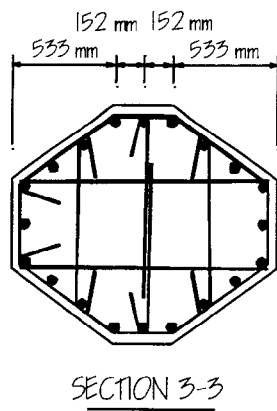
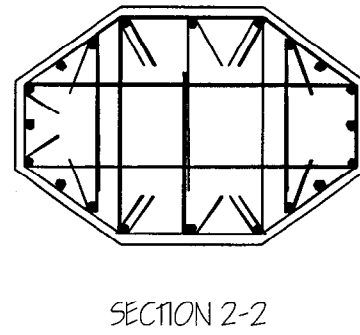
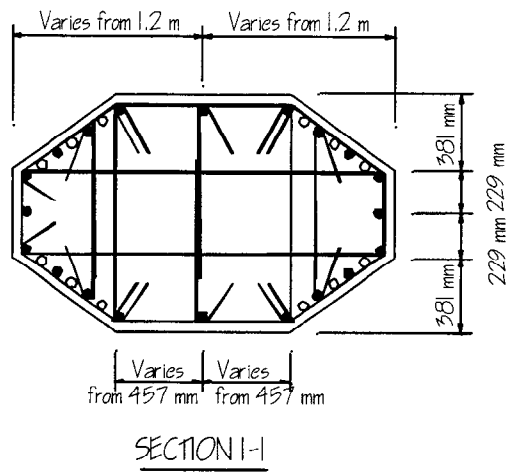


Figure 2 - 2 Elevation of Columns in Bridge I-1556



- Notes: 1. All main (vertical) bars marked $\bullet$ are # 11
 All main (vertical) bars marked $\circ$ are # 7
 2. All ties and cross ties are # 5 @ 4"

Figure 2 - 3 Reinforcement Detail of Typical Column on I-1556

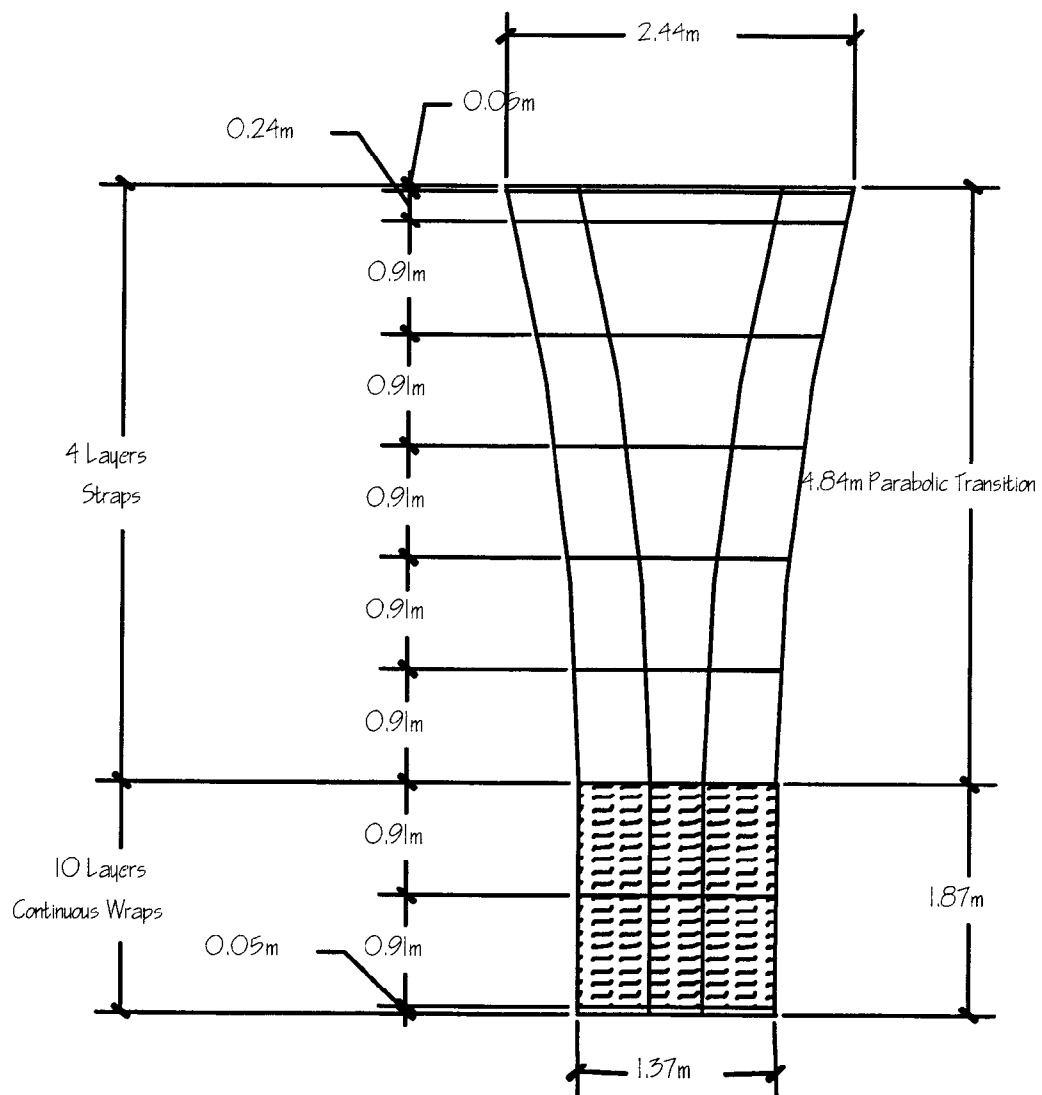


Figure 2 - 4 Glass Fiber-Epoxy Jacket Elevation

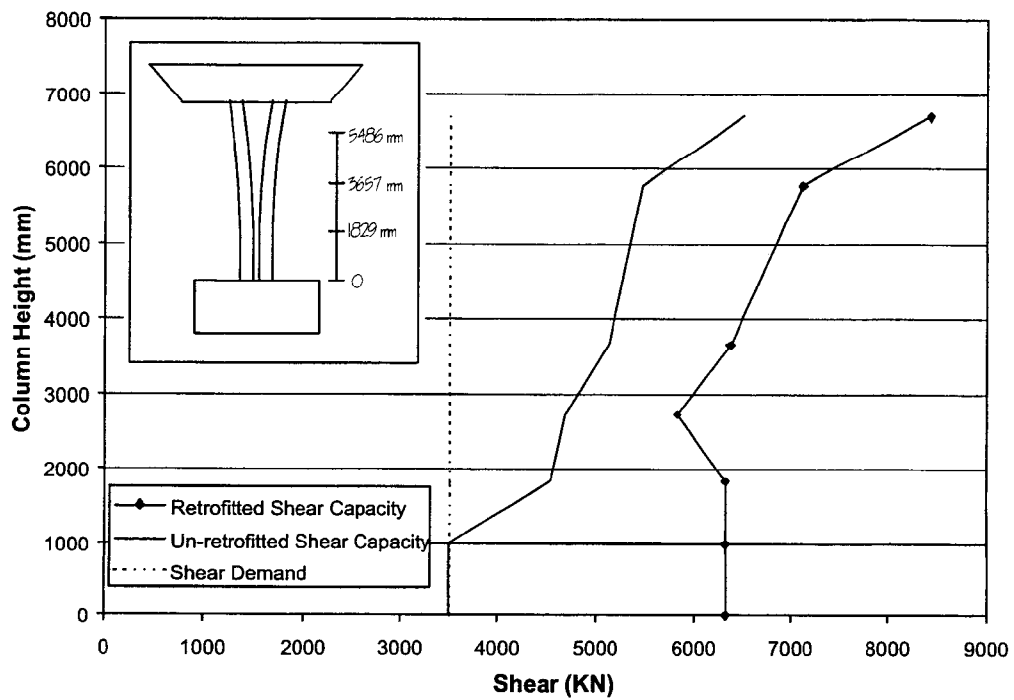


Figure 2-5 Shear Demand and Capacities for the As-Built and Retrofitted Columns

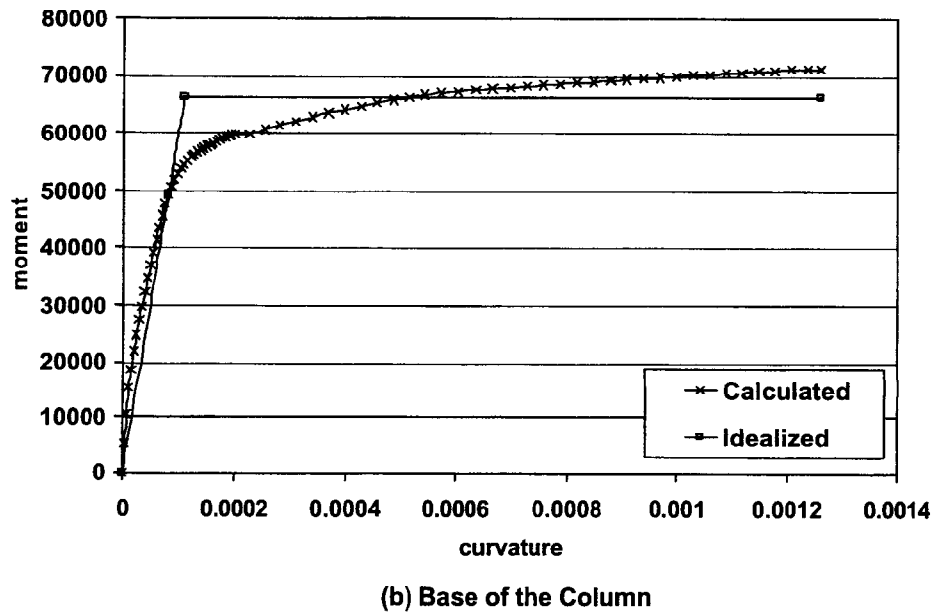
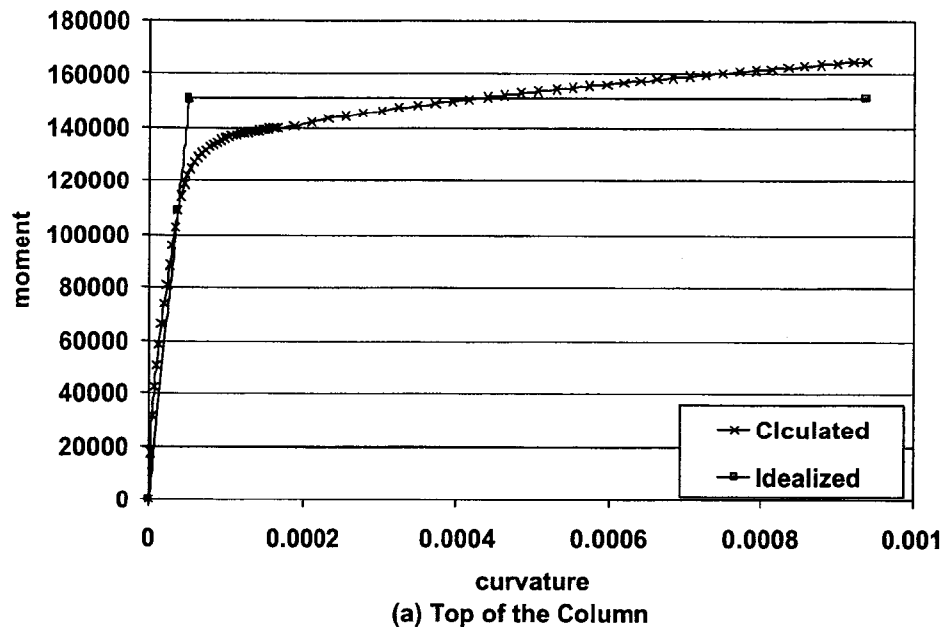


Figure 3 - 1 Moment-Curvature Relationships for the ends of the Jacketed Column

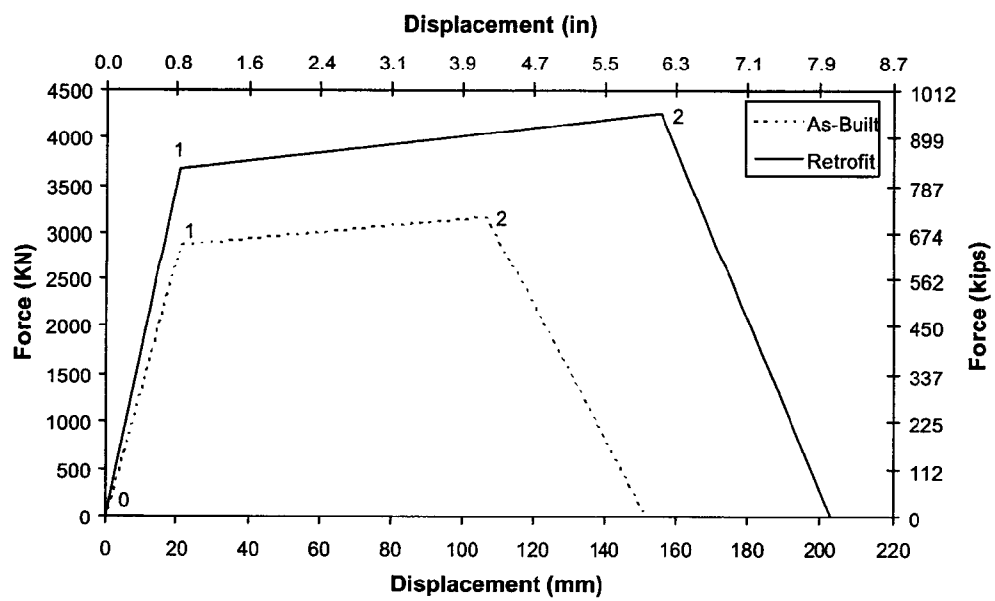


Fig. 3-2 Force vs. Displacement Envelopes for As-Built and Retrofitted Columns

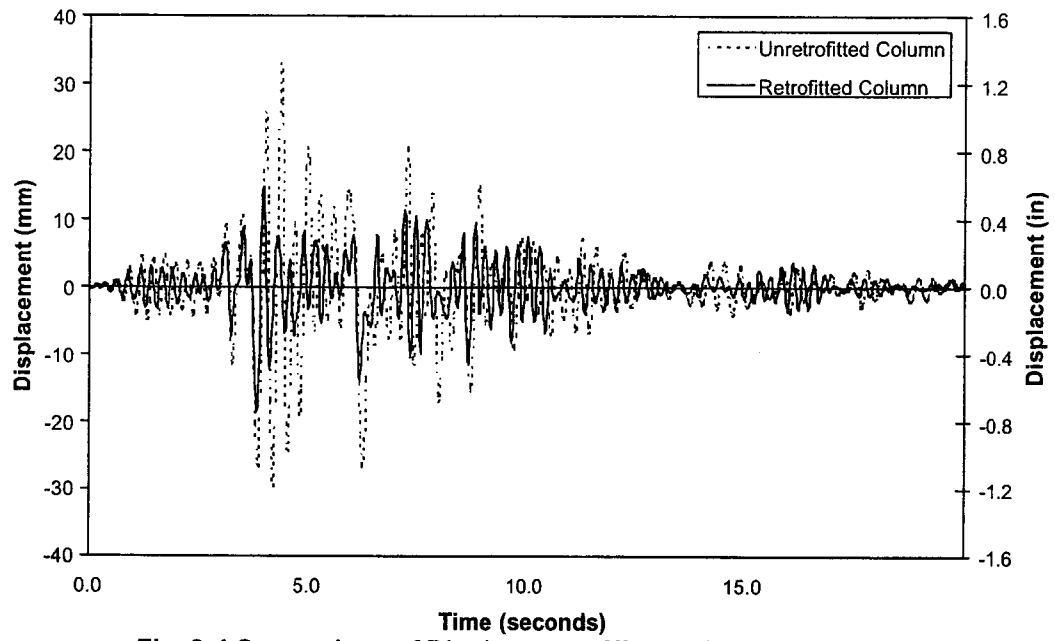


Fig. 3-4 Comparison of Displacement History for 1.5 x Sylmar Event

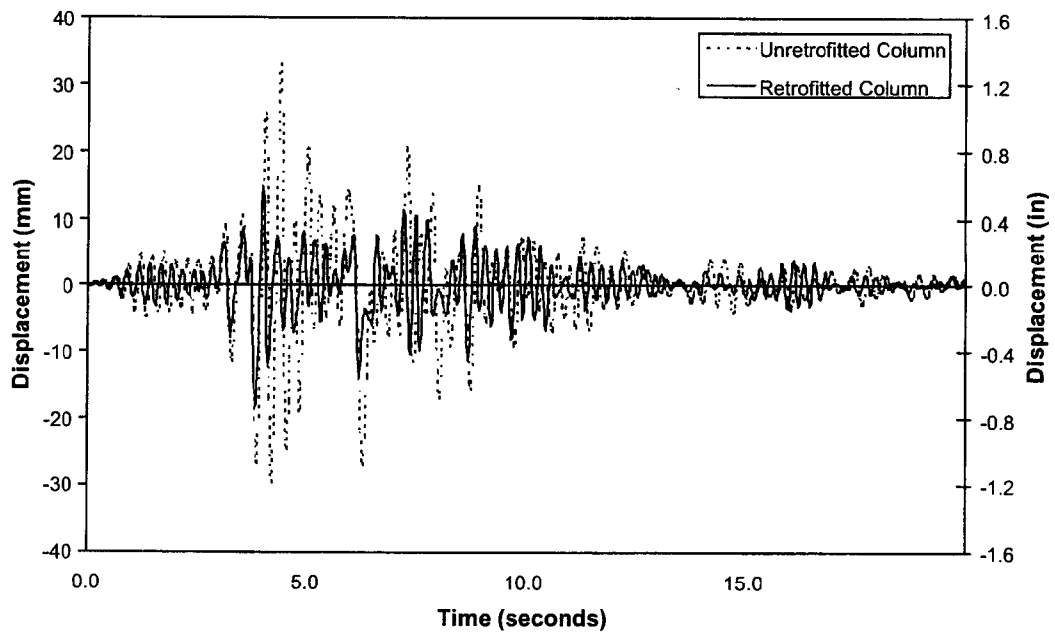


Fig. 3-4 Comparison of Displacement History for 1.5 x Sylmar Event

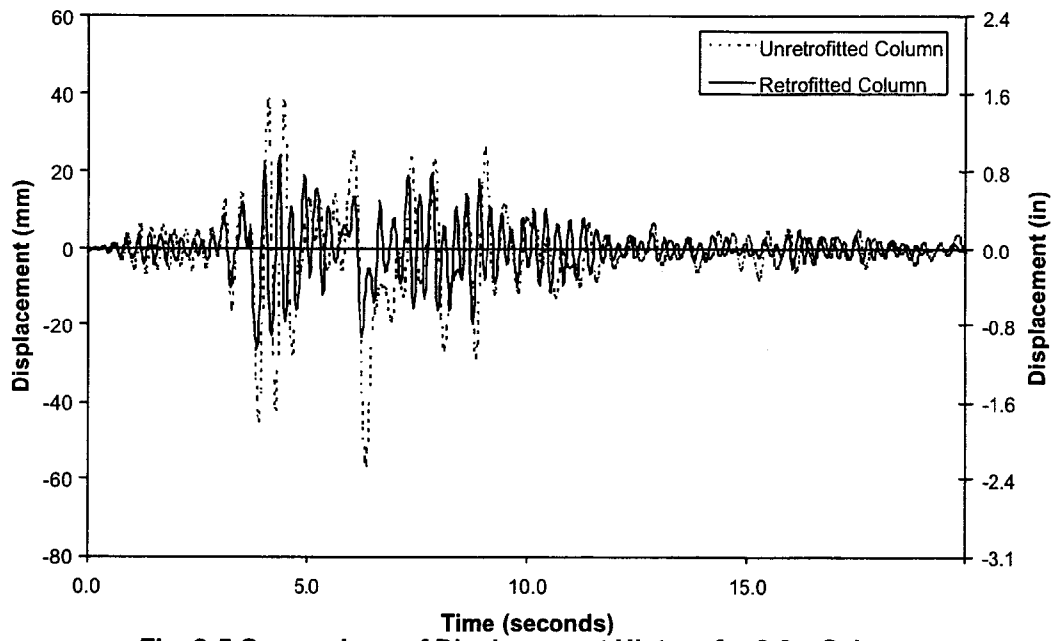


Fig. 3-5 Comparison of Displacement History for 2.0 x Sylmar Event

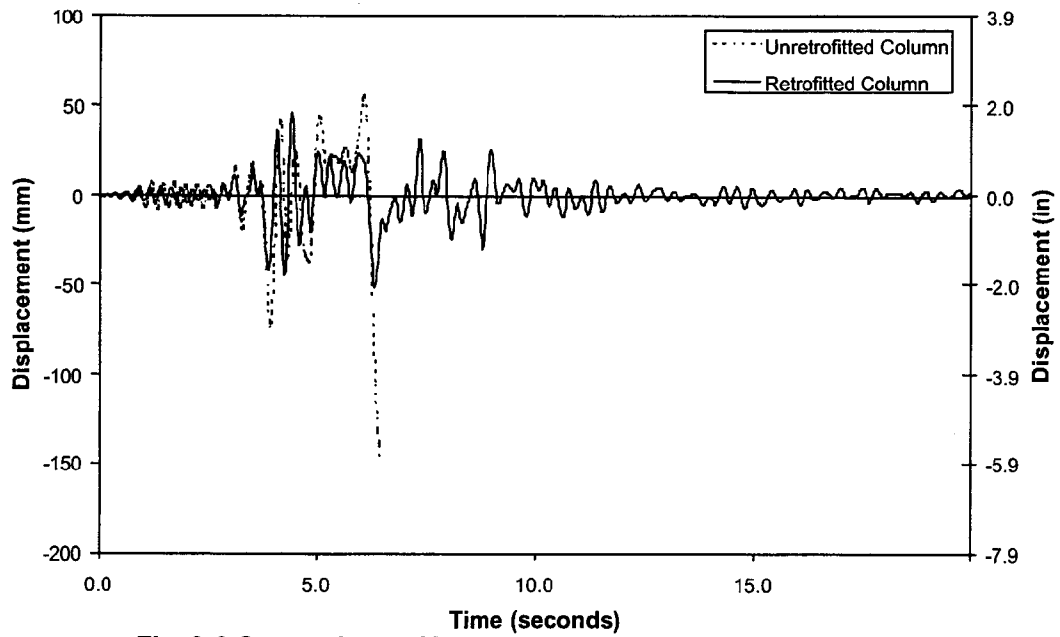


Fig. 3-6 Comparison of Displacement History for 2.5 x Sylmar Event

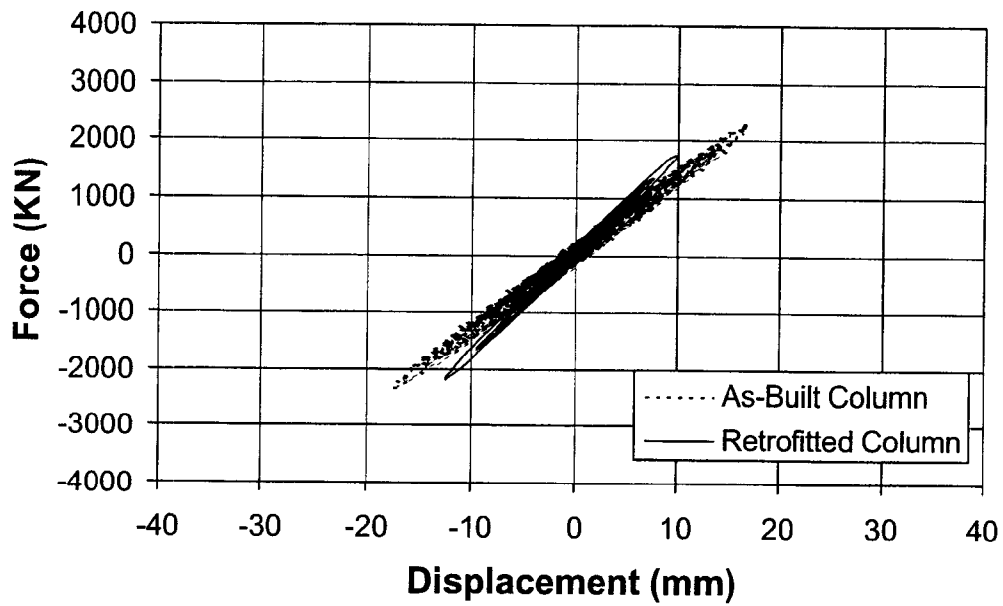


Figure 3 - 7 Comparison of Hysteresis Curves
for 1.0 x Sylmar

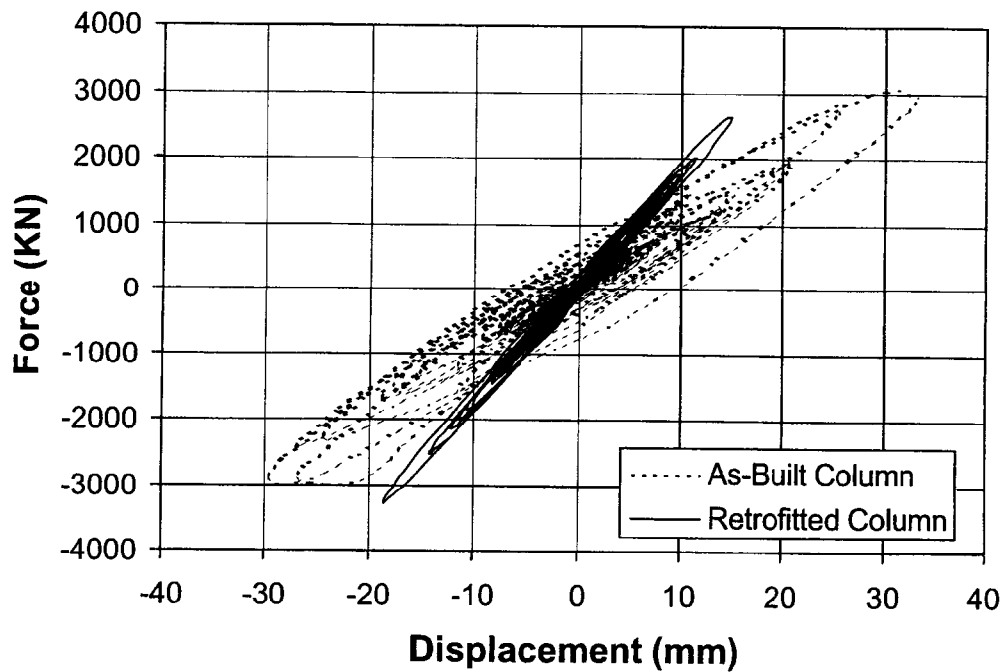


Figure 3 - 8 Comparison of Hysteresis Curves
for 1.5 x Sylmar

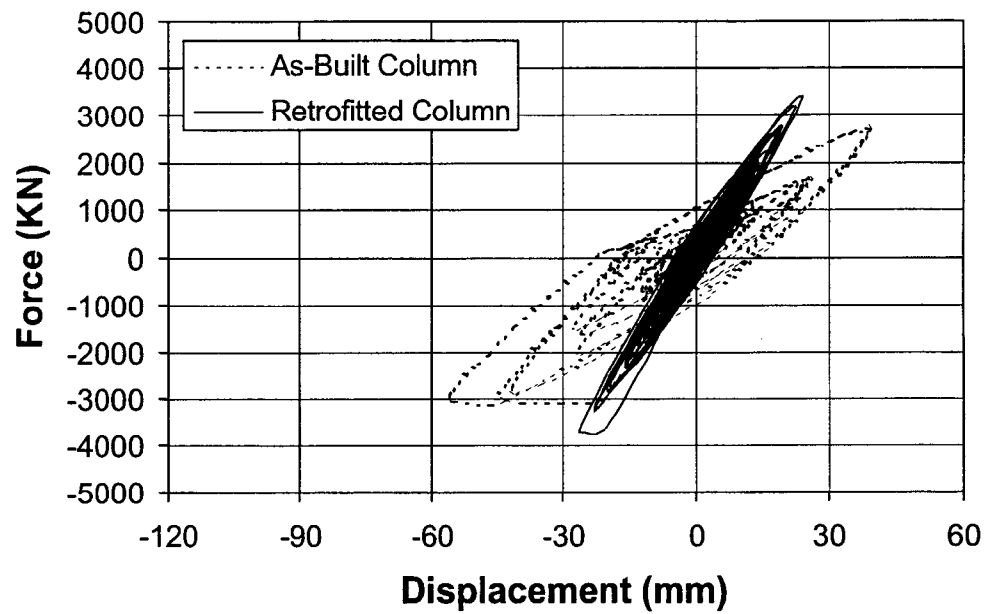


Figure 3 - 9 Comparison of Hysteresis Curves
for 2.0 x Sylmar

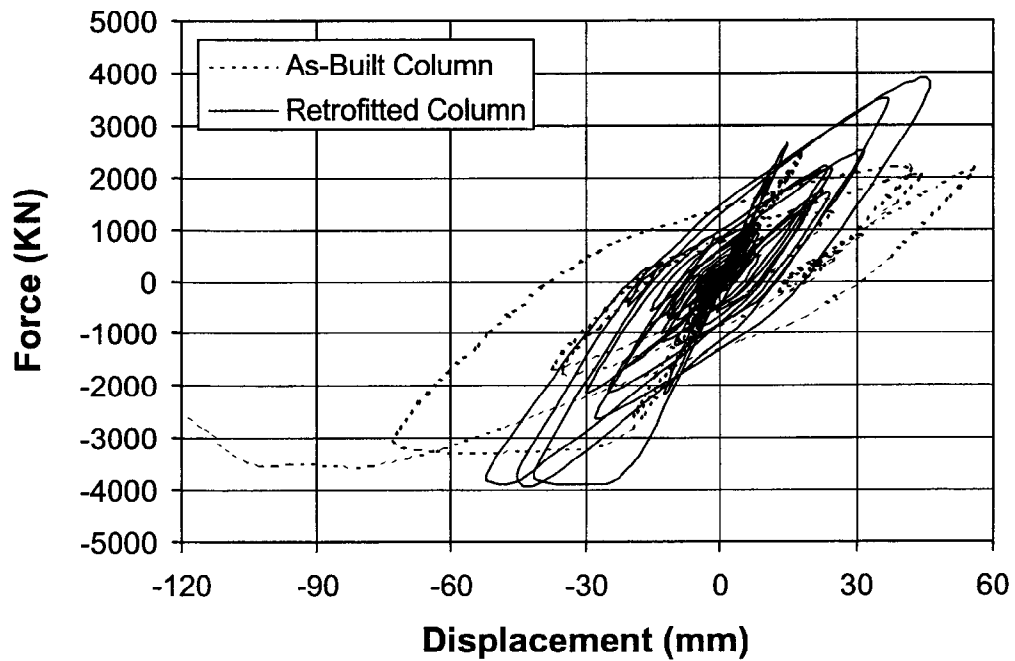


Figure 3 - 10 Comparison of Hysteresis Curves
for 2.5 x Sylmar

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