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# POST-EARTHQUAKE ASSESSMENT OF BRIDGE COLLAPSE AND DESIGN PARAMETERS

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## ABSTRACT

Severe damage to Bull Creek Canyon Channel bridge and collapse of Mission-Gothic Undercrossing are investigated herein for assessment of the January 17, 1994 Northridge earthquake. To perform time-history analysis, both bridges are modeled as three-dimensional systems considering bridge columns, superstructures, and abutment conditions. For Bull Creek Canyon Channel bridge, a method based on ACI bridge design code is introduced to determine columns' shear failure. Insufficient spiral in columns above connections of the channel wall leads to bridge damage. Failure mechanism of Mission-Gothic Undercrossing is due to effects of flare on the structural system. Use of flare in bridge structure design may result in unexpectedly unfavorable effects on bridge performance during earthquakes. Suggestions based on analyses and post-earthquake observation are proposed to improve bridge performance.

## KEYWORDS

Bridge structures; finite element method; flared columns; ground motion; Northridge earthquake; reinforced concrete; shear capacity; spiral; stiffness; time-history analysis.

## INTRODUCTION

The January 17, 1994 Northridge earthquake caused extensive damage or collapse of about eight freeway bridges (Buckle, 1994). These bridges were designed and built from the mid-sixties to the mid-seventies. Their performance during the earthquake indicated that much damage to bridge structures could be avoided on the basis of current design standards. Bull Creek Canyon Channel bridge and Mission-Gothic Undercrossing are among those bridges damaged or collapsed. The former crosses a water channel and the channel wall is integrally connected with columns of one bent, while the latter features directional dependence of flared column stiffness and no stirrups to enclose flare reinforcement in the lower part of the flare. Both display a skew layout of bents and abutments. These unique characteristics resulted in bridge failures under the strong earthquake.

For Bull Creek Canyon Channel bridge, nonlinear time-history analysis is conducted to investigate the damage mechanism under three-dimensional ground motion. Abutment conditions and the effective fixity of channel wall to columns are taken into account. A method to determine the column's shear mode is developed based on ACI bridge design code (ACI 343R-88, 1988) and is combined with computer algorithm IAI-NEABS (Imbson & Associates, 1993) to evaluate the column's ultimate capacity. Column's shear failure at the bent fixed by the channel wall is observed to be due to insufficient spiral in this region. Further studies indicate that spiral in the potential plastic hinge zone is inadequate according to ACI codes. Mission-Gothic Undercrossing is also analyzed under three-directional ground motion. Effects of flare on structural systems are investigated. The role of flare is important to structural systems and should not be neglected. Suggestions based on damage assessment and post-earthquake field observation are proposed to improve bridge performance.

## BULL CREEK CANYON CHANNEL BRIDGE

### *Structural Description and Failure Behavior*

Bull Creek Canyon Channel bridge is located on State Route 118 San Fernando Freeway, Los Angeles, California. Its two parallel bridge structures carry east and west bound traffic over Bull Creek Canyon Channel. Both bridge structures have three spans. Layout and dimensions of the two bridge structures are shown in Fig. 1. Superstructure is a multi-cell, cast-in-place prestressed box girder with structural depth of 1.22 m. Monolithic drop-diaphragm abutments are laid on galvanized sheet metal and on neoprene strips on pile cap beam. Transverse movement at each abutment is prevented by four parallelogram-shaped shear keys. Length of columns at bent 2 varies from 6.71 m to 7.32 m measuring from the bottom soffit to the top of the pile cap. Effective length of columns at bent 3 is approximately 4.88 m due to connection of the channel wall about 2.44 m from the top of the pile cap. All the columns are octagonal in 1.219 m diameter with 36 to 64 #9 longitudinal reinforcement. The main reinforcement stretches 0.9 m into the cap beam and extends 1.1 m to 1.2 m into the pile cap with outward-bend L-hooks. Number 5 spiral is evenly distributed with 76 mm pitch along the 1.219 m top and bottom of columns and with 305 mm spacing over the remainder. Reinforcement details in columns of the right structure at bent 3 are also shown in Fig. 1. Abutment and bent footings are sustained by 406 mm to 610 mm diameter cast-in-drilled-hole (CIDH) concrete piles. Length of the piles is approximately 12.19 m at abutments and 9.14 m at bents.

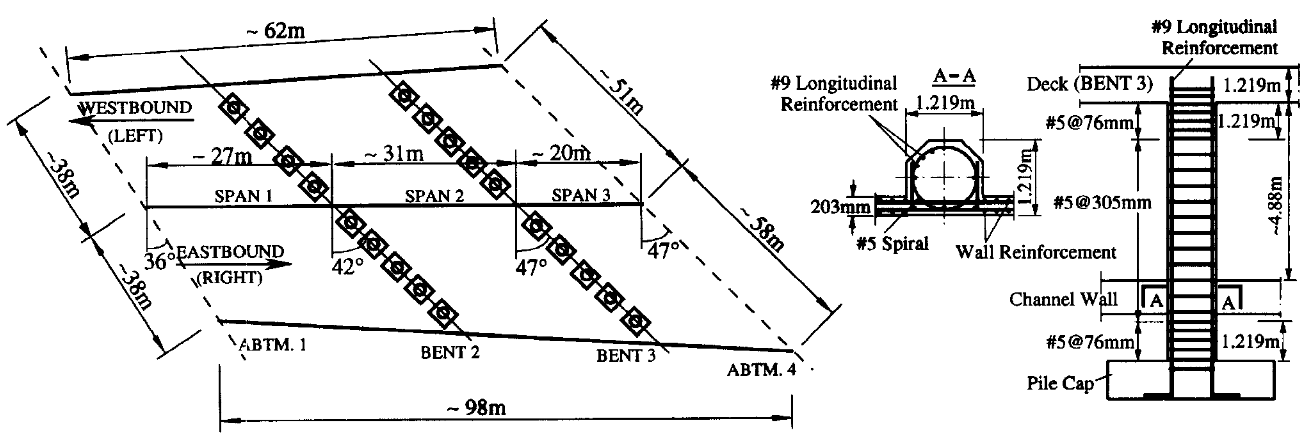


Fig. 1. Bull Creek Canyon Channel bridge and details of column reinforcement

Field observation after the earthquake indicates that the bridge suffered severe damage during the earthquake. Two southernmost columns of the right structure at bent 2 showed a rupture of spiral and buckling of longitudinal reinforcement just in the transitional area of spiral spacing. Column failure led the superstructure to drop. Columns at bent 3 for both structures failed above the channel wall. At this failure area, spiral spacing was 305 mm. Approximately 381 mm offset of approach pavement on the southeast side implied that the failure of shear keys allowed the structure to have transverse movement.

*Structural Model for Nonlinear Time-history Analysis*

Due to similar damage behavior of right and left structures, a three-dimensional lumped mass model is established for the right bridge structure. Superstructure is modeled as linear elements, while each column is divided into three nonlinear elements. Element at each end of the column considered as potential plastic hinge zone is 1.219 m long, with spiral spacing of 76 mm except that elements at the lower part of columns at bent 3 are confined by a spiral with 305 mm pitch. The middle element of columns features a spiral spacing of 305 mm. Columns are considered to be fixed with footings at bent 2 and channel wall at bent 3. Interaction between foundation and soil is not taken into account due to scant damage information. Seismic loading used herein was recorded at Sylmar County Hospital parking lot which is approximately 18 km away from the epicenter. A modification of seismic loading is made to consider the effects of the distance between the recording station and the bridge which is about 7.5 km from the epicenter (Campbell, 1981, Cheng and Lou, 1995). Detailed descriptions, such as properties of superstructure and column segments, as well as capacity of columns under axial load and biaxial bending, refer to the report by Cheng and Lou (1996).

*Observation of Numerical Results*

Nonlinear time-history analysis of the right bridge structure is performed by IAI-NEABS on Sun Sparc station. Yield pattern of the right Bull Creek Channel bridge is presented in Fig. 2. Column B3A first yields at both ends (3.86 s). Yielding first appears at the top of column B3B (3.86 s) and then at the bottom (3.88 s). After 0.16 s, the remaining columns of bent 3 yield. Most of the columns at bent 3 begin to yield at the top and bottom simultaneously. Column B2E finally yields at the top only.

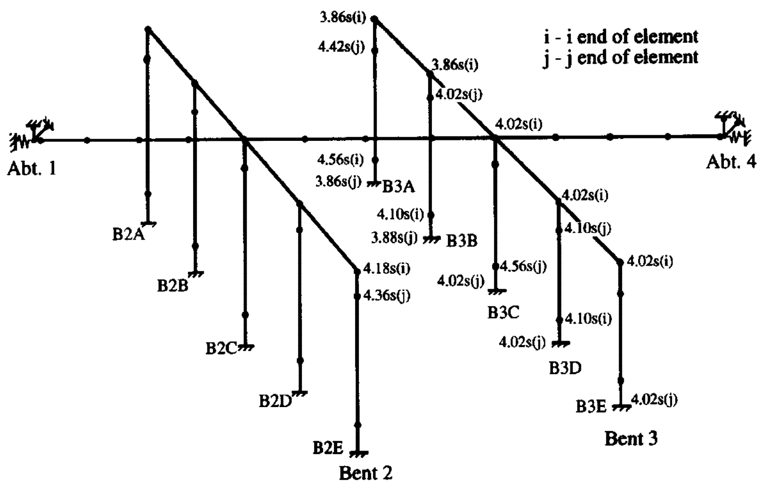


Fig. 2. Yielding pattern of Bull Creek Canyon Channel bridge's right structure

Total shear-friction reinforcement of each shear key is 8 #6 (4,542 $mm^2$) and the strength of reinforcement is 414 MPa. Using equation R11.7.3 of the ACI building code (ACI 318-89, 1992), shear strength of the four shear keys is 12,540 kN, which is much less than maximum shear force of 136,259 kN (4.20 s) at abutment 1 and 171,236 kN (4.20 s) at abutment 4, respectively (Cheng and Lou, 1996). Shear keys at abutment 4 fail at 3.18 s, and after 0.16 s, shear keys at abutment 1 also fail. Shear block does not fail during the time-history analysis. In addition, computer results indicate that each column displays almost the same displacement along its bent direction. At the same time, displacement perpendicular to the bent increases from column B2A to B2E at bent 2, and decreases from column B3A to B3E at bent 3. This suggests that under the earthquake, the bridge structure not only moves in the horizontal plane but also rotates itself.

*Shear Failure Criterion of Reinforced Concrete Column*

At bent 3, column elements at the bottom should fail prior to those at the top because inadequate spiral in these bottom elements makes columns unable to resist strong shear forces above the connections of columns and channel wall. However, analytical results show that columns at bent 3 yield at both ends simultaneously. This phenomenon may be explained by the assumption of computer algorithm IAI-NEABS. IAI-NEABS assumes that the ultimate shear capacity of columns is always greater than shear force. Thus shear failure of columns is not expected to happen (Imbsen and Penzien, 1984). This assumption is reasonable for most reinforced concrete members have sufficient shear capacity to avoid the occurrence of shear failure. Because the spiral ratio at the top of columns is different from that at the connection of column and the channel wall, an additional method is needed to use with computer results to determine the column's shear or flexural failure mode. Based upon ACI bridge design code (ACI-343R, 1988), the column's shear capacity V can be expressed as

$$V = 0.17\left(1 + 0.073\frac{N_o}{A_g}\right)\sqrt{f'_c}\, b_w\, d + \frac{A_v f_{ys}\, d}{s} \tag{1}$$

where $N_o$ = compressive axial force, N; $A_g$ = gross area of column cross-section, $1.359 \times 10^6$ mm$^2$; $f'_c$ = compressive strength of concrete, 41.37 MPa; $b_w$ = diameter of circular section, 1,219 mm; $d$ = distance from extreme compressive fiber to centroid of tension reinforcement, 946 mm; $A_v$ = area of spiral within spacing, 400 mm; $f_{ys}$ = spiral yield strength, 303 MPa; $s$ = spiral spacing, mm. Therefore, Eq. (1) is the function of axial load $N_o$ which varies with the change of time.

Comparison of column shear capacity based on Eq. (1) and shear forces due to the earthquake indicates that columns at bent 2 do not fail in shear except column B2E. For column B3A at bent 3, shear failure first occurs at the bottom (3.68 s), where spiral spacing is 305 mm. Then the top of column fails in shear since the spiral spacing in this region is 76 mm. Yielding appears later at 3.86 s at both ends. This implies that the column fails before plastic hinge develops. Table 1 points out that all the columns at bent 3 first display shear failure at the bottom. Shear failure at the top occurs later than that at the bottom due to more spiral provided in the top plastic hinge zone. Therefore, the analytical failure mechanism is similar to that observed after the earthquake (Priestly et al., 1994) and gives a satisfactory description of the bridge structure damage.

Table 1. Summary of failure modes

| Mode of Failure | | Columns | | | | | |
|---|---|---|---|---|---|---|---|
| | | B2E | B3A | B3B | B3C | B3D | B3E |
| Top | Shear | 4.26 s | 3.72 s | 4.22 s | 4.04 s | 4.10 s | 3.90 s |
| | Flexure | 4.14 s | 3.86 s | 3.86 s | 4.02 s | 4.02 s | 4.02 s |
| Bottom | Shear | — | 3.68 s | 3.86 s | 3.88 s | 3.88 s | 3.86 s |
| | Flexure | — | 3.86 s | 3.88 s | 4.02 s | 4.02 s | 4.02 s |

*Bridge Performance Improvement*

The aforementioned analytical results show that columns at bent 3 fail in shear before they reach their flexural capacities. This implies that spiral in the potential plastic hinge zone is insufficient. To evaluate columns' shear capacity, the following equations of ACI bridge design code (ACI-343R, 1988) are used here to determine the need for spiral in the potential plastic hinge zone.

$$\rho_s = 0.12\frac{f'_c}{f_{ys}} \tag{2}$$

$$\rho_s = 0.45\left(\frac{A_g}{A_{co}} - 1\right)\frac{f_c'}{f_{ys}} \tag{3}$$

where $\rho_s$ = ratio of volume of spiral to total volume of core; $A_{co}$ = core area, $9.81 \times 10^5$ mm$^2$. Other notations are the same as in Eq. (1). Due to the use of #5 spiral in the original design, the smaller spacing comes from Eq. (3) and is approximately 45 mm, much less than the design spacing of 76 mm. The original design provided inadequate spiral in the potential plastic hinge zone.

Table 2. Summary of failure modes ( 45 mm spiral spacing)

| Mode of Failure | | Columns | | | | | |
|---|---|---|---|---|---|---|---|
| | | B2E | B3A | B3B | B3C | B3D | B3E |
| Top | Shear | — | 4.10 s | 4.20 s | 4.04 s | 4.06 s | 4.10 s |
| | Flexure | — | 3.86 s | 3.86 s | 4.02 s | 4.02 s | 4.04 s |
| Bottom | Shear | — | 4.12 s | 4.12 s | 4.04 s | 4.06 s | 4.10 s |
| | Flexure | — | 3.86 s | 3.88 s | 4.02 s | 4.02 s | 4.04 s |

To investigate whether column spiral provided by ACI 343R-88 is sufficient to prevent the occurrence of shear failure prior to flexural yielding, nonlinear time-history analysis of the right bridge is performed again changing spiral spacing in the potential plastic hinge zone from 76 mm to 45 mm. Spiral spacing in the middle column decreases from 305 mm to 76 mm. Columns' bottom elements at bent 3 are strengthened with spiral of 45 mm pitch. Table 2 summarizes shear failure and flexural yielding for each column. Flexural yielding clearly happens prior to shear failure at the both column ends. Closely spaced spiral in the potential plastic hinge zone ensures that after yielding, columns can still exhibit ductile behavior and dissipate kinetic energy. Thus the brittle failure of bridge columns can be avoided.

## MISSION-GOTHIC UNDERCROSSING

### *Structural Description and Failure Behavior*

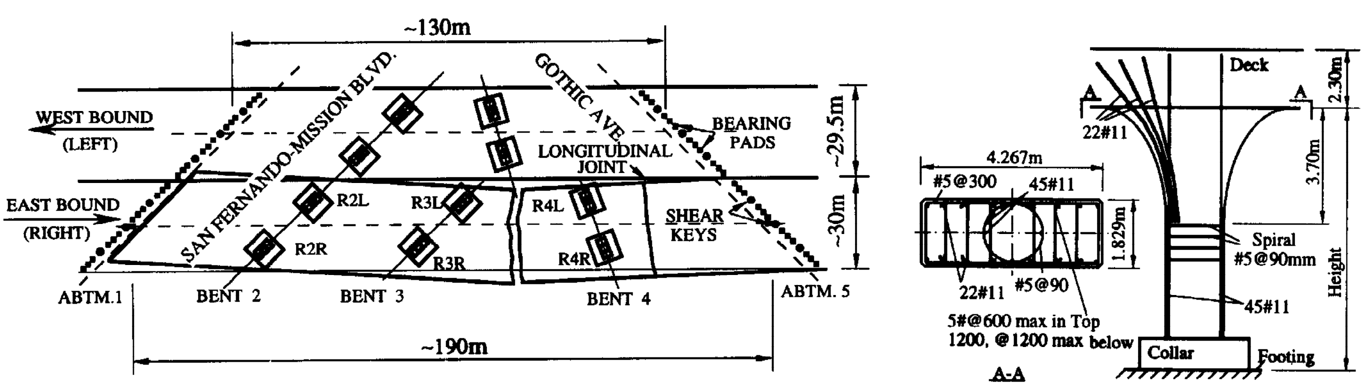


Fig. 3. Mission-Gothic Undercrossing and column reinforcement details

Mission-Gothic Undercrossing lies close to the northwest of Bull Creek Canyon Channel bridge. Bridge abutments were designed to be parallel to San Fernando-Mission Boulevard and Gothic Avenue. Therefore, a significant difference in length of the two parallel bridge structures results from skew of abutments. Dimensions of bridge structures are shown in Fig. 3. Superstructure is a multi-cell, cast-in-place post-stressed girder with a structural depth of 2.3 m, and sits on twelve elastomeric bearing pads at abutments.

Three square shear keys at each bridge end are embedded in the abutment seat, and the superstructure can move 152 mm in the bridge's longitudinal direction. Reinforcement details of each flared column are the same and are shown in Fig. 3. Note that there is no stirrup to enclose flare reinforcement along the lower part 1.22 m of flare. Detailed descriptions are reported by Cheng and Lou (1996, 1995).

The right bridge structure collapsed completely between span 3 and 4. Severe damage to span 1 and 2 was observed. Abutment 1 remained seated, showing little longitudinal movement, but transverse movement was about 330 mm. At bent 2, the right column failed and slid on the footing due to damage in the reinforced concrete collar, while the left column collapsed, dropping the flare to one side of column. Columns at both bent 3 and 4 failed at the lower part of flare. Since flares were not completely damaged, the bent was able to rest on them. The end of the diaphragm at abutment 5 was unseated. The left bridge structure did not collapse, but it suffered extensive damage. Most columns failed in the region of flare. Abutment 1 moved approximately 76 mm in the longitudinal direction and 51 mm in the transverse direction. Transverse movement of 254 mm was observed at abutment 5. Diaphragms at both ends remained seated.

*Structural Model for Nonlinear Time-history Analysis*

Analysis of bridge collapse behavior is focused on the right bridge structure because the left one did not collapse. A three-dimensional model is established for computer program IAI-NEABS. The superstructure is divided into linear elements, and columns are modeled as nonlinear ones. The upper part of column in the flare region is evenly separated into three segments, while the lower part is considered as one element. Due to heavy flare, an equivalent cross-section is introduced to treat each segment of the column as a straight-line element. Effects of elastomeric bearing at each abutment are taken into account. Relative displacement in the transverse direction is prevented by shear keys. Based on the details of bridge structure design, columns are assumed to connect footings with hinges. Interaction between soil and foundation is not considered due to scant damage information. Detailed descriptions of properties of superstructure and column's segments, stiffness of elastomeric bearing pads, yielding surface of reinforced concrete column under axial load and biaxial bending, and the like, are reported by Cheng and Lou (1996, 1995) elsewhere. Since Mission-Gothic Undercrossing is near Bull Creek Canyon Channel bridge, modified ground motion for Bull Creek Canyon Channel bridge is adopted here for nonlinear time-history analysis.

*Observation of Numerical Results*

Figure 4 displays the yielding pattern of the right bridge structure under three-dimensional ground motion. The structure yields from column R3L and R3R in the lower part of the flare at 3.38 s. After 0.06 s, column R4L and R4R yield. Column R2L and R2R reach yielding at 3.40 s and 3.48 s, respectively. All the columns yield within 0.7 s after 3.38 s, which suggests the bridge collapsed in a sudden manner. Shear keys at both abutments are expected to fail at 3.3 s. Failure mechanism of Mission-Gothic Undercrossing conforms to that observed after the earthquake (Priestly et al., 1994). Consideration of three-dimensional ground motion may give a more accurate description of collapse behavior and reveal unfavorable column's internal forces (Cheng and Lou, 1995).

Post-earthquake field observation indicates that the directional dependence of flared column stiffness is one cause of the bridge collapse. In fact, flare in the upper part of columns was originally designed for architectural aesthetics (Buckle, 1994). Therefore, reinforcement details in flare were not intended to increase column strength. After an earthquake, flare should be expected to fail, and could be repairable. The determination of flare reinforcement is based on experience because no efficient method is available to analyze flared column behavior. During the Northridge earthquake, damage to columns in the lower part of flare suggests that flare plays a role in column strength and that the failure mechanism of columns is affected by the existence of flare.

To determine the effects of flare on the performance of bridge structure, a three-dimensional model (identical with the former except that there are no flares in the upper part of columns) is used for analysis. Yielding pattern of the right bridge structure without flares is presented in Fig. 4. Yielding begins at the top of columns. All the columns enter yielding stages from the top, and yielding extends toward the middle. The yielding pattern here is totally different from that with flare in the upper part of columns. Thus the effect of flare cannot be neglected in prediction of bridge failure mechanism even though flare was originally considered as architectural aesthetics and not purposely designed to strengthen column capacity.

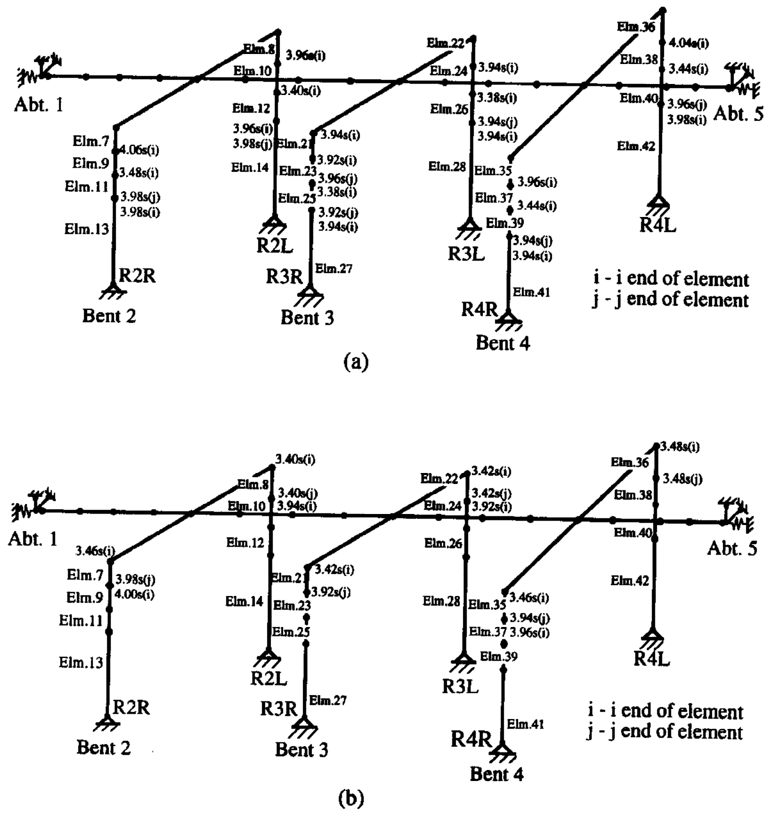


Fig. 4. Yielding patterns: (a) column with flare; (b) column without flare

Given heavy flare, the volume of flare at each column is approximately 6.116 $m^3$ and the volume of the column itself is approximately 19.831 $m^3$. Thus a flared column augments about 31 % of its mass. It is well known that under ground motion, the increase of structural mass results in an enlargement of external forces. Corresponding internal forces also become larger. Computer results show that flared columns suffer larger moments in both principal axes of cross-section than columns without flare (Cheng and Lou, 1996). Because of the existence of flare and little increase of column strength in the flare region, the bridge structure is subjected to stronger seismic force, and is more likely to be damaged or to collapse. This explains why a structure with flared columns reaches yield earlier than one without flare at the upper part of columns (see Fig. 4). Thus the use of flare in bridge structure design is not suggested; it may lead to unexpectedly unfavorable effects on bridge performance during an earthquake.

Nonlinear finite element analysis of flared columns (Cheng and Lou, 1995) offers an additional finding. Flared columns are parallel to their corresponding bent. Thus columns at bent 2 and 3 have an approximately 45-degree angle between the strong axis of the flared column's cross-section and the longitudinal direction of the bridge structure; the strong axis of the flared columns at bent 4 is approximately perpendicular to the bridge's longitudinal direction. Since transverse ground motion is the dominant cause of bridge collapse (Cheng and Lou, 1996, 1995), columns at bent 2 and 3 are mainly subjected to 45-degree lateral force while columns at bent 4 are under lateral force along the strong axis of their cross-section. Nonlinear finite element analysis indicates that a 13 % reduction of column capacity is observed if lateral force is at a 45-degree angle to the strong axis of the flared column's cross-section. Therefore, capacity of flared columns under lateral force is dependent on the orientation of the column's strong axis.

## CONCLUSIONS

Post-earthquake assessment of these two bridges gives a better comprehension of their collapse or damage behavior. Bull Creek Canyon Channel bridge is severely damaged at bent 3 because of insufficient spiral in columns above connections of the channel wall. Further studies point out that spiral in the potential plastic hinge zone is also inadequate. If spiral spacing in the potential plastic hinge zone follows ACI bridge design code, flexural yielding of columns will appear prior to shear failure and ductile behavior of columns can be expected. This proves that ACI code provisions for spiral spacing in the potential plastic hinge zone effectively prevent column brittle failure.

Mission-Gothic Undercrossing collapsed due to a skew of bents and abutments, directional dependence of flared column stiffness and no stirrups in the lower part of flare to enclose flare reinforcement. Flare plays an important role in structural failure mechanism. The existence of flare increases internal forces in bridge columns and changes the location of plastic hinges from the top of columns to the middle. Moreover, a structure with flare reaches yielding stage earlier than one without flare. Based on analytical solutions, the use of flare in the upper part of columns is not suggested. If flare has to be used for reasons of architectural aesthetics, circular flare is recommended so that column capacity is independent of skew bents. In addition, supplementary spiral is needed in the lower part of flare to enclose flare reinforcement and prevent propagation of concrete spalling.

## ACKNOWLEDGMENT

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