Shake table response of bridge columns

M. Saiidi¹, A. Itani¹, N. Johnson¹, J. Mortensen² & S. Ladkany³

¹Department of Civil Engineering, University of Nevada, Reno, USA
²Nevada Department of Transportation, Carson City, Nevada, USA
³Department of Civil Engineering, University of Nevada, Las Vegas, USA

Abstract

Two recent studies on the shake table response of large-scale bridge columns are summarized. The first was undertaken to evaluate the adequacy of a new performance-based design (PBD) method for confinement steel in circular columns and the other study was focused on retrofit of substandard octagonal bridge columns supported on pedestals. The test results demonstrated the effectiveness of the proposed PBD for a predetermined curvature ductility. The retrofit studies showed the effectiveness of fiber-reinforced plastic fabrics in eliminating pedestal damage and the improvements in response when the column shear demand was controlled.

1 Introduction

Bridge columns are typically the primary members that control the seismic response of bridge structures. They are designed to undergo large plastic deformations and dissipate energy during strong earthquakes and generally prevent damage to other bridge components. A great deal of information has been obtained at the University of Nevada, Reno (UNR), from testing of large-scale bridge column models subjected to shake table earthquake simulation and quasi-static cyclic loading. The focus of the studies on bridge columns has included new column behaviour, retrofit of substandard columns, and repair of severely damaged columns. This article summarizes the findings for an example study for each of the first two groups. Both projects utilized shake table simulation in addition to analytical studies. The study on new columns was concentrated on performance-based design. The retrofit studies were focused on octagonal columns supported on pedestals.
2 Performance-based design of confinement steel

Bridge columns are generally designed to be flexure dominated. The amount of confinement steel to provide certain level of ductility needs to be a function of axial load, longitudinal steel, geometric properties, and material properties. Existing code equations generally either ignore some of these variables or do not rationally include the parameters. Furthermore, the equations do not include a performance parameter, and hence are not suited for performance-based design.

A comparative study of different confinement steel design methods for circular columns has shown that using existing equations can lead to a relatively small ductility capacity under higher axial forces. This is demonstrated through an analytical study the results of which are shown in Fig. 1 for a typical round column [1].

In this figure P is the axial load, $f'_{c}$ is the concrete compressive strength, and $A_g$ is the gross column section area. The methods shown are based on the American Concrete Institute (ACI) [2], American Association of State and Transportation Officials (AASHTO) [3], first edition of the AASHTO Load and Resistance Factor Design (LRFD) [4], California Department of Transportation (Caltrans) [6], and the Applied Technology Council (ATC-32) [5]. The PBD curve shown in the figure is discussed subsequently. All the methods, except for the one by ACI include the axial force in the equation. Nonetheless, the curvature capacity drops significantly as the axial load increases.

A new simple performance-based design (PBD) method was developed for confinement steel design. The method incorporates the effects of axial load, vertical steel ratio, section geometry, and material properties to design the confinement steel for a target curvature ductility capacity. The effect of most of these parameters is implicit in a moment-curvature analysis of the column. Compared to the existing design codes, the method is somewhat more involved because it requires a moment-curvature analysis rather than a set of equations.
This does not appear to be an issue because moment curvature analysis is becoming common in earthquake design. Details of the method are presented by Saiidi and Mortensen [7]. To evaluate the proposed PBD method, two, quarter-scale column models were tested on one of the UNR shake tables.

2.1 Test columns

The columns were designed to be flexure dominated. The confinement steel in one of the models, SC-CAL, was designed based on the Caltrans provisions and in the other, SC-PBD, it was based on the proposed method. It was decided to test the columns under an axial load that was close to the upper range of the values shown in Fig. 1 because the curvature ductility is lower in that range, thus making high axial loads more critical. A nominal axial load index (defined as \( P/f_c A_g \)) of 0.25 was used.

The vertical steel consisted of 16-#4 bars and the steel ratio was 2.78 percent. The specified concrete compressive strength was 5 ksi (34.5 MPa) and the specified steel yield stress was 60 ksi (414 MPa). The target curvature ductility in SC-PBD was 10. The aspect ratio of the column was selected so that shear would not control the design. The spiral steel was controlled by confinement. Gage wire W2.9 was used for the spiral with a spacing of 1 in. (25 mm) and 1.6 in. (41 mm) in SC-PBD and SC-CAL, respectively.

A large number of strain gages were installed on the longitudinal and transverse bars at the critical sections to monitor the behaviour under different levels of earthquakes. In addition, displacement transducers were placed at different heights to measure curvature and to allow for assessment of the actual moment-curvature relationship. Other instrumentations included an accelerometer and a displacement transducer at the top of the column. The test setup is shown in Fig. 2.

![Figure 2: Test setup.](image-url)
The table motion simulated a synthetic earthquake that matched the ATC-32 spectrum for medium soil (Type D). The synthetic record was chosen over others because of the high displacement ductility demand that it could place on the columns without exceeding the limitations of the shake table. The ATC-32 record contains a large number of high-amplitude pulses that allow for the evaluation of the specimens under multiple inelastic cycles during each run. The amplitude of the simulated earthquakes was increased in subsequent runs to observe the column performance under a range of limit states.

2.2 Evaluation of test models and the proposed PBD

The performance of the two columns with different confinement steel, one designed based on the current Caltrans procedure and the other based on the proposed PBD method, was distinctly different from each other particularly under moderate and strong earthquake simulations. The nominal peak acceleration during Run 10 was 0.66g and the earthquake would be considered to be a moderate to strong motion. In both columns the cover concrete in the plastic hinge spalled. However, the extent of spalling in SC-PBD was considerably less (Fig. 3). Specimen SC-CAL failed during run 12 with a target peak input acceleration of 0.99g, whereas Specimen SC-PBD failed during the 13th run with a peak acceleration of 1.1g. The measured curvature ductility capacity (defined as the ratio of the maximum curvature and the effective measured yield curvature) was 8 for SC-CAL and 9.5 for SC-PBD, which was close to the target value of 10. When the proposed PBD was applied to the column discussed in the beginning of Sec. 2, it was found that a uniform level of ductility was accomplished (Fig. 1) regardless of the level of axial force.

Figure 3: SC-CAL (left) versus SC-PBD (right).

3 Retrofit of columns supported on pedestals

As part of a comprehensive study to develop retrofit methods for a 24-span freeway bridge in Las Vegas, Nevada, the retrofit of single columns with octagonal shape supported on pedestals was studied. Three, quarter-scale
models where constructed and tested on the shake table, the first representing the as-built condition while the other two were retrofitted. The models were tested in the strong direction subjected to the 1994 Northridge earthquake record at Sylmar. The intensity of input earthquake was gradually increased until failure.

### 3.1 As-built model

Figure 4 shows the elevation and the reinforcement for the most critical column. This model was referred to as OLVA (for octagonal Las Vegas column, as-built). Horizontal reinforcement in the pedestal is not shown for clarity. The total height of the model was 2.99m. The column was rigidly connected to the pedestal, but the pedestal connection to the footing was a one-way hinge with dowels centered along the length of the pedestal base. The amount of lateral steel in both the column and the pedestal was severely deficient based on current earthquake resistant design standards.

### 3.2 Retrofitted columns

Testing of OLVA revealed that the pedestal is particularly vulnerable to earthquake forces (See Sec. 3.3) but the overall displacement ductility capacity of the column is reasonable. Two retrofitted columns, OLVR-1 and OLVR-2 were tested to evaluate the retrofit effectiveness. Through past research and the testing of the as-built model it was learned that one-way hinges lack sufficient energy dissipation capacity. Therefore, the pedestals in both retrofitted models were enlarged to minimize yielding at the base and to force plastic hinging to the top of the pedestal in the column. Figure 5 shows the as-built and retrofit scheme for the prototype. To address the pedestal weakness, unidirectional glass fiber reinforced plastic (GFRP) fabrics were installed on the pedestal, with fibers running in the horizontal direction. The design of GFRP was based on the horizontal force that was estimated using a strut-and-tie model of the column.

![Figure 4: As-built reinforcement detail.](attachment:image.png)
Earthquake Resistant Engineering Structures IV

Figure 5: As-built (left) and retrofitted (right) hinge details.

Shake table testing of OLVR-1 showed that the retrofit of pedestal was successful. However, shifting of the plastic hinge to the top of the pedestal increased the shear demand. The shear demand was reduced in OLVR-2 by severing some of the longitudinal bars at the base of the column and removing concrete. The hollow bars in Fig. 6 show the severed bars. The objective of the retrofit was to improve the ductility without placing a jacket on the column.

3.3 Measured Performance

Figure 7 shows the pedestal damage in OLVA after it was subjected to a simulated earthquake with the maximum acceleration of 1.2g. Vertical cracking of the pedestal began under 0.45g, and the crack continued to widen to the point that the column separated from the pedestal during the last run. At a displacement ductility of 3, the crack was sufficiently wide to the point that a bridge inspector would have closed the bridge to traffic. An important point to note is that pedestals are typically buried below grade and their damage can go undetected making the bridge potentially vulnerable despite its sound appearance above ground. The overall load-deflection envelope of the measured hysteresis curves appeared to be satisfactory with a displacement ductility capacity of nearly 6.9 (Fig. 8).

Figure 6: Severed bars in OLVR-2.

The pedestals in both retrofitted models performed well in that the yielding of pedestal base hinge was limited, there was no damage in the pedestal, and the strains in the GFRP jacket were relatively small. The retrofit in OLVR-1 was successful in shifting the plastic hinge to the column immediately above the top of the pedestal. OLVR-1 did not show any sign of major distress. After the run
with a peak table acceleration of 1.5g many flexural and some shear cracks were evident (Fig. 9). The specimen did not show any sign of strength deterioration until it was subjected to the Sylmar record with peak acceleration amplified to 1.7g, when it failed in shear after considerable yielding of the plastic hinge. Figure 8 shows that the column lateral load capacity increased substantially compared to OLVA, but the displacement ductility capacity dropped to 4.4 due to shear failure of the column along a diagonal line.

Given that the displacement ductility capacity of OLVR-1 was moderate and no pedestal damage was seen, it was decided in OLVR-2 to reduce the shear demand and determine if the ductility capacity can be improved without placing a jacket on the column. The method to reduce the shear demand was described in Sec. 3.2.

The response of OLVR-2 showed improvement over OLVR-1, but the crack pattern in both models was generally similar (Fig. 10). The severing of the some of the bars at the base of the column reduced the lateral load capacity (Fig. 8) by 23% compared to OLVR-1, and the displacement ductility capacity was increased to 5.3, or a 20% increase. OLVR-2 failed in shear under maximum input earthquake amplitude of 1.7g. The crack formation pattern was similar to that of OLVR-1. Despite the improvement in ductility capacity and no pedestal damage, a further refinement of the retrofit is being considered by either severing more bars at the base of the column or by placing a minimum number of FRP wraps on the column to improve its shear strength.

Figure 7: Pedestal damage in OLVA.
Earthquake Resistant Engineering Structures IV

Figure 8: Measured response envelopes.

Figure 9: Flexural and shear cracks in OLVR1 after earthquake with 1.5g.
Earthquake Resistant Engineering Structures IV

Figure 10: Flexural and shear cracks in OLVR2 after earthquake with 1.5g.

4 Conclusions

Research on the seismic response of reinforced concrete bridge columns in the past few decades has led to substantial improvement in design and performance of bridges. New bridge columns can now be designed with a higher level of confidence than before because of new research data. However, many issues remain unresolved and untested even for new construction. With respect to upgrading substandard bridge columns, retrofit techniques for routine cases have been developed through research. These methods do not readily apply to many unusual columns. This article examined the need to improve the confinement steel design in circular columns to achieve a uniform and desired level of ductility and presented a study to retrofit columns supported on pedestal. Through analysis and shake table testing of large-scale models, the paper demonstrated that the proposed performance-based design method for confinement steel was effective in accomplishing its objective. The retrofit of the columns with pedestal led to eliminating pedestal damage, improved energy dissipation, and improved ductility capacity. Further refinement of the retrofit might be necessary to eliminate shear failure.

References

Earthquake Resistant Engineering Structures IV


