Seismic performance and retrofit of fixed and hinged RC bridge columns with short bar anchorage
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Abstract

A large number of bridges located in the Reno/Sparks area have been identified as having potentially inadequate detailing to resist earthquakes. A major deficiency in many columns of these bridges is short dowel bars in the footings and short lap splices at the base of the columns. Also, the columns are inadequately confined. To identify the extent of inadequacy of these bridge columns and to develop a method to retrofit them, six 0.4 scale model specimens were constructed and tested. The first two specimens represented "as-built" details for existing bridge columns; one with a fixed base detail and the other with a one-way hinge detail. The other four specimens were identical to the two as-built, but were retrofitted to address their deficiencies. Two of the retrofitted specimens represented the retrofitting of the specimens with a one-way hinge detail. One was tested with yielding anchor bolts, while the other was tested with a yielding base plate. The other two retrofitted specimens represented columns with a fixed base detail. One of these was tested in the strong direction of the column while the other was tested in the weak direction. All specimens were subjected to a nominally constant axial load and cyclic lateral loads in the strong direction. Test results of the as-built specimens showed that the prototype bridge columns are very susceptible to severe damage during strong earthquakes, and that they need to be strengthened. Results from testing the four retrofitted specimens showed that connecting the steel jacket to the base plate provided an effective retrofitting method and enabled the columns to reach their design strength under cyclic lateral loading. This retrofit also enhanced the shear strength and the ductility of the columns. This paper presents the results of testing the as-built specimens, the one-way retrofitted hinged base specimen with a yielding base plate, and the retrofitted fixed base specimen tested in the strong direction.

Introduction

Recent earthquakes in California and Japan have revealed the poor seismic
performance of bridges that were designed prior to the development of modern bridge design codes. Many bridges collapsed or were severely damaged during those earthquakes, generally as a consequence of inadequate design and brittle detailing. Substructures are considered to be the primary source of bridge damage during strong earthquakes. Failure of bridge columns may result from a number of deficiencies related to the design philosophy that was common before the 1971 San Fernando earthquake. Bridge columns designed using that philosophy generally lack the necessary ductility capacity and the shear strength to survive moderate to strong earthquakes. A very common detail for the failed bridge columns during recent earthquakes was column reinforcement that is lap spliced immediately above the foundation, and with a splice length that is insufficient to develop the bar strength under cyclic loads. In addition to that, the failed columns were also characterized by small amount of inadequately anchored, widely spaced transverse reinforcement that failed to confine the core concrete.

As a result of the poor behavior and the damage occurred to bridges during recent earthquakes, Nevada has started a process to evaluate and upgrade the existing highway bridges. Many reinforced concrete highway bridges in Northern Nevada have been identified as having inadequate detailing to resist earthquakes. A common deficiency in many columns of these bridges is short dowel bars in the footing and short lap splices at the base of the columns. Also, the columns are poorly confined and lack adequate shear strength. Therefore, the prerequisite elements for sufficient ductility and earthquake energy dissipation are missing in these columns. Some of the column bases are rigidly connected to the footings while others incorporate one-way hinge details. The purpose of the study summarized in this paper was to develop and test column-footing connection seismic strengthening details for both types of column bases.

Research Significance

Approximately thirty bridge structures located in the Reno/Sparks area have been targeted for seismic retrofit during the next few years. These bridges were designed in the late 1960's and were found to be very susceptible to severe damage during strong earthquakes. Results from this research provide the basis for design of retrofit measures to upgrade the performance of column-footing connections and also provides an effective means to enhance the shear strength and the ductility of these bridge columns.
Experimental Program

The experimental program included testing of six 0.4 scale model specimens. The first two specimens represented "as-built" details for existing bridge columns; one with a fixed base detail and the other with a one-way hinge detail. The other four specimens were identical to the two as-built specimens, but were retrofitted to address their deficiencies. A steel jacket was placed on each of the retrofitted columns to improve the shear strength and the concrete confinement. To ensure sufficient moment transfer, these steel jackets were connected to the footings by a base plate and bolts that were anchored to the footing. These bolts were essential because the tests of the as-built specimens showed that column dowels in the footings were too short and did not maintain the column-footing integrity under cyclic loads. The first two retrofitted specimens were identical to as-built hinged base column specimen but in one specimen the base plate was designed to remain elastic while in the other the base plate was designed to yield. The other two retrofitted specimens were identical to the as-built fixed specimen; one was tested in the strong direction, while the other was tested in the weak direction. The base plate for these two specimens was designed to yield while the anchor bolts remain elastic.

In all specimens, the footing did not model the existing footing but was designed to be stronger than the column bases to preclude failure in the footings. The specimens model the prototype bridge columns between the footing and point of inflection in the column. The columns were loaded vertically with a nominally constant axial to simulate the dead load of the bridge superstructure and loaded laterally to simulate earthquake forces.

This paper presents the results of testing the as-built specimens, the one-way retrofitted hinged base specimen with a yielding base plate, and the retrofitted fixed base specimen tested in the strong direction.

Test Specimens

a) As-Built Specimens

The one-way hinged column specimen is shown in Fig. 1. The column pedestal represented the pedestals in the existing columns. At the junction between the footing and the pedestal, there was a one-way hinge region. During the construction, the hinge was formed by casting a 25" x 6" x 1.25" depressed key in the footing. The 1.25 in. dimension is scaled down from the specified size of 3.0 in. on the prototype. Then, two 0.5 in. thick expansion joint fillers (Styrofoam) were added on the sides of the key-way. To simulate the existing
Figure 1: As-Built Hinged Base Specimen Details

Figure 2: As-Built Fixed Base Specimen Details
dowel anchorage in the prototype bridges, nine # 5 deformed Grade 40 reinforcing bars were used in one row to connect the footing to the pedestal. The ratio of the actual to the required anchorage length was kept the same as that in the prototype column. Also, the steel ratio was the same as that in the prototype column. The column ties in hinged base specimen provided the same confinement steel ratio as that in the prototype. The column steel ratio at the hinge throat was 1.86 percent; the average steel ratio in the prototype hinge throat is 1.9 percent.

The fixed-base column specimen is shown in Fig. 2. To simulate the existing dowel anchorage in the prototype bridges, sixteen # 6 Grade 40 reinforcing bars were used to connect the column to the footing. Similar to the hinged specimen, the ratio of the actual to the required anchorage length was kept the same as that in the prototype column. Also, the steel ratio was the same as that in the prototype column. The column ties in the specimen provided the same confinement steel ratio as that in the prototype. The steel ratio at the column-footing connection was 2.77 percent which is similar to the average steel ratio of the prototype column.

b) Retrofitted Specimens

The retrofitted hinged specimen details were identical to that of the as-built hinged specimen except for the slight reduction in the column height due to expecting thickening of the footings. The reinforcement details for the column and pedestal were identical to that used for the as-built specimen.

A steel jacket was used to enhance the flexural and shear strength of the column specimen. The jacket consisted of two parts; an "oblong" jacket that had the same shape and height as the column, and a rectangular jacket to confine the column pedestal (Fig. 3). The jacket was fabricated from 1/4 in. thick A36 hot-rolled steel. A 1/2-in. gap was provided between the column and the jacket and was pressure-grouted with a high strength grout. Because the dowel anchorage in the footing was insufficient, the steel jacket was connected to the footing through a steel base plate with stiffeners, all A36 type. Also, to ensure a one-way hinge action, the base plate was only placed in the direction of moment transfer.

The base plate was anchored to the footing through one row of three 3/4-in. diameter A193 Grade B7 high strength bolts which were 14 in. embedded into the footing (Fig. 3). A high strength epoxy adhesive was used to anchor these bolts to the footing. A 1-in. thick high strength grout was used between the footing and base plate for leveling. The base plate capacity was determined to be consistent with what is required to develop the moment capacity of the connection with well-anchored column bars. The anchor bolts were designed to remain elastic.

The retrofitted fixed-base column specimen in the strong (RFS) was
Figure 3: Retrofitted Hinged Base Specimen Details (RHP)

Figure 4: Retrofitted Fixed Base Specimen Details (RFS)
similar to the as-built fixed base specimen except for the column height. The column had the same dimensions of the as-built specimen with an exception of the column height that was reduced by 2 in. to account for the footing overlay that is expected to be placed to strengthen the footing.

For RFS specimen (Retrofitted fixed base specimen in the strong direction), an “oblong” steel jacket was used to enhance the flexural and shear strength of the column specimen (Fig. 4). The jacket had the same shape and height as the column. No horizontal stiffeners were placed on the flat side of the column jacket. The jacket was fabricated from 1/4 in. thick A36 hot-rolled steel. A 1/2- in. gap was provided between the column and the jacket and was pressure-grouted with a high strength grout. Because the dowel anchorage in the footing were insufficient, the steel jacket was connected to the footing through a steel base plate with stiffeners. Two similar 3/4” thick, A36 steel base plates were welded to the steel jacket. The base plate was anchored to the footing with two rows of 3/4” diameter ASTM-A307 bolts on each side of the column strong direction. The bolts were embedded 14 inches into the footing to preclude bond failure. The base plate capacity was determined to be consistent with what is required to develop the moment capacity of the connection with well-anchored dowel bars. Also, the size and the number of anchor bolts were chosen to prevent their yielding. To limit the size of weld connecting the jacket to the base plate, five 5/8” thick, A36 stiffeners were used.

**Test Procedure**

The test procedures were identical for all test specimens. The first step was to apply an axial load through the threaded rods to the predetermined load which simulated the dead load stresses. A nominally constant axial load of 90 kips was applied to the as-built and retrofitted hinged-base specimens, while an axial load of 80 kips was applied to the as-built and retrofitted fixed-base specimens. For the retrofitted specimens, and after applying the total axial load, the anchor bolts were tightened. Cyclic lateral loads were applied using displacement control. The displacement was applied slowly. All specimens were subjected to several cycles of increasing column drift; however, no particular earthquake was simulated during testing.

**Test Results**

**As-Built Hinged-Base Column**

The as-built hinged-base specimen was subjected to sixteen cycles of drift reversals at seven amplitudes. There were minor cracks on the tapered side of
Figure 5: Load-Deflection Response for the As-Built Hinged Specimen

Figure 6: Load-Deflection Response for RHP Specimen
the column at six and twelve inches above the column pedestal. These were the only cracks observed on the column above the pedestal. However, cracks were observed in the pedestal on the tapered side of the specimen. The cracks led to a concrete spall over a 5.5 in. long by 10.0 in. wide area. One vertical splitting crack parallel to the dowels was observed on each narrow side of the pedestal. Failure of the column was caused by complete loss of cover concrete in the lap-splice region due to large displacement reversals.

Figure 5 shows the load-deflection response for the as-built hinged column specimen. The curves indicate significant strength degradation even at 1% drift. The slope of lines connecting the peak loads in the loops indicates a strength degradation of 43 percent and 21 percent in the positive and negative direction, respectively. Usually, a part of the apparent strength degradation is due to the P-Δ effect. However, the axial load application mechanism in this test made P-Δ effect negligible. The strong pinching of the hysteresis loops indicates a considerable reduction in the energy absorption capacity of the hinge (Fig. 7). One of the main reasons for the stiffness degradation was the gradual deterioration of bond between the concrete and the steel dowels passing through the hinge throat. The bond failure occurred due to the short anchorage length of the bars. In addition, the severe cracking of concrete on the column pedestal resulted in further stiffness and strength degradation. Generally, the hinge was not able to reach the strength and ductility capacity to the level of a hinge with adequate steel anchorage.

**Retrofitted Hinged Base with Yielding Plate (RHP)**

Specimen RHP was subjected to twenty cycles of drift reversals at nine amplitudes. The column was able to maintain its strength up to $\mu_\Delta = 6$. During the test, the base plate developed large tensile stresses and yielded in tension. Also, uplifting and bending of the base plate were very pronounced. Bearing of the base plate against the grout and the footing did not cause any damage to the concrete. Most of the base plate deformations took place between the toe of the stiffeners and the anchor bolt line. However, after the weld between the stiffeners and the base plate failed, the base plate deformations extended to the area under the stiffeners. This led to the fracture of welds between the base plate and the pedestal steel jacket. On the other hand, the anchor bolts connecting the base plate to the footing developed small stresses and did not yield. Generally, retrofitting the as-built column with a steel jacket and yielding base plate increased the capacity of the column to the level of a similar column with well-anchored bars.

Displacement ductility of 1 (Fig. 6) corresponds to the first yield of the base plate. In both directions of loading, and after the column had reached the maximum capacity, there was a sudden reduction in the column strength to 48.5
Figure 7: Load-Deflection Envelopes for As-Built and RHP Specimens

Figure 8: Load-Deflection Response for the As-Built Fixed Specimen
kips in the push direction, and 49.7 kips in the pull direction (Fig. 6). This is because the welds connecting the stiffeners to the base plate and to the steel jacket started to crack. After that sudden reduction in column strength, there was no significant strength degradation until the end of the test. Early cracking of the welds between the stiffeners and the base plate on the tapered side of the column prevented the column in the push direction to reach the ductility level reached in the pull direction (Fig. 6). Although the shape of hysteresis loops indicated small energy absorption, strength and ductility capacity were maintained up to drift ratios of approximately 6 percent. Figure 7 shows a comparison of the envelopes of the hysteresis curves for the as-built and retrofitted specimens. Note the improved strength and ductility provided by the yielding base plate. More details about the behavior of the retrofitted specimens are presented in Ref. 3.

**As-Built Fixed-Base Column**

The behavior of the as-built fixed-base column under cycling loading was poor. Stiffness and strength degradation were considerable during the test (Fig. 8). Shear and flexural cracks were observed on the column. The inclined shear cracks led to spalling of concrete cover over an area at the mid-height of the column. Also, some circumferential and vertical cracks were observed on the circular part of the column. Bond failure was observed between the dowels and the surrounding concrete. The bond failure led to 10.0 in. long (lap splice length) spalling of concrete cover in the lap-splice region on the straight side of the column. On the tapered side, column bars were not efficiently lap spliced with the straight dowels (starter bars). Therefore, smaller forces were transmitted from the column bars to the dowels. The lower forces led to less damage on the tapered side.

Figure 8 shows the load-deflection response for the as-built specimen. The curves indicate significant strength degradation. The column was not able to reach the strength and ductility capacity to the level with adequate steel anchorage. The slope of lines connecting the peak loads in the load deflection loops indicates a strength degradation of 46 percent and 50 percent in the positive and negative direction, respectively.

The stiffness degradation and the pinching of the hysteresis loops were mainly due to the gradual deterioration of bond between concrete and dowels at the location of short lap splices. Also, the shear cracks observed on the column, and the lack of confinement from the transverse steel had some contribution to the low energy dissipation. Failure of the column was caused by complete loss of cover concrete in the lap-splice region due to large displacement reversals. More details about the behavior of the as-built specimens are presented in Ref. 1.
Figure 9: Load-Deflection Response for RFS Specimen

Figure 10: Load-Deflection Envelopes for As-Built and RFS Specimens
Retrofitted Fixed Base Column in the Strong Direction (RFS)

Specimen RFS was subjected to twenty cycles of drift reversals at ten amplitudes which is the same displacement history for the as-built specimen. However, higher drifts were reached during the test due to the good performance of the retrofitted column. During the test, the column was able to sustain large lateral displacements without any strength degradation (Fig. 9).

During the test, the base plate developed large tensile strains and yielded. Also, uplifting and bending of the base plate were very noticeable. On each side of the plate, the outer row of anchor bolts remained elastic, while the inner row of bolts developed larger tensile strains with a minor strain hardening but did not fracture. This is because the uplifting of base plate at the location of the inner row was greater than that at the outer row. Bearing of the base plate against the grout and the footing did not cause any damage or crushing in the footing. Weld cracking was only noticed between the stiffeners and the base plate but none was observed between the base plate and the steel jacket. Generally, the behavior of the column was greatly improved due to the retrofitting method used. The steel jacket connected to the base plate provided an effective retrofitting method and enabled the calculated flexural strength with well-anchored bars to be achieved. Also, yielding of the base plate considerably increased the ductility of the column and showed very stable behavior during the test. This type of behavior would be very desirable during strong earthquakes since a base plate acting as a semi-rigid connection can dissipate energy and serve as a seismic isolator and hence, can reduce seismic response. On the last cycle (Fig. 9), a displacement ductility of 8 was reached without any strength degradation. Due to the capacity limit of the actuator, the test had to be stopped. It appeared that higher displacement ductility levels could be reached.

The steel jacket developed very small tensile and compressive strains and did not yield. This is because the short dowels connecting the column to the footing were eliminated from the retrofitted specimen (to account for the addition of concrete to the top of the footing), and hence, no significant expansion of the concrete cover at the location of the short lap splice region occurred. Figure 10 shows a comparison of the envelopes of the hysteresis curves for the as-built and retrofitted fixed base specimen tested in the strong direction. Note the improved strength and ductility provided by the yielding base plate.

Development of Seismic Design Recommendations

Based on the analyses of the test results presented and the observations made during the experiments, seismic design recommendations were formulated. These recommendations provide the basis for design of retrofit details to
upgrade the performance of columns and column-footing connections of deficient bridges located in the Reno/Sparks area. The columns will be able to sustain sufficient numbers of inelastic cyclic displacements and can develop displacement ductilities in excess of approximately four and six for the one-way hinged base columns and the fixed base columns, respectively. Also, the retrofitting method presented will enable the columns to reach their design strength under cyclic lateral loading. The following are the summary of the recommendations:

- The retrofit measure should be designed such that the strength of the connection is governed by the yielding of base plate.
- The thickness of the steel jacket should be determined to inhibit shear failure and lap splice failure at the base of the column, with shear being the primary consideration.
- Anchor bolts should be designed to remain elastic.
- The base plate thickness should be determined assuming that the plate is fixed at the bolt line and at the toe of stiffeners, and that the plastic moment ($M_p$) of the plate is developed at both ends.
- Effect of column axial load on the base plate and the anchor bolts can be neglected.
- Compression on the concrete and grout needs to be checked to ensure that concrete compression failure does not occur.
- The stiffeners should be designed to remain elastic and to resist the simultaneous effect of moment and shear at the connection.
- Weld connecting the column to the base plate should have the capacity to resist the simultaneous effect of moment and shear at the connection.

Conclusions

The following conclusions are based on the study presented in this paper:

1. The highway bridges in general and those in Northern Nevada in specific that incorporate short dowel bars in the footings and short lap splices at the base of the columns are very susceptible to severe damages during strong earthquakes and need to be strengthened.

2. Bar slippage or anchorage failure causes severe strength and stiffness deterioration in the columns and reduces energy absorption substantially. It also results in substantial increase in column rigid-body displacement or rotation which under severe earthquakes can lead to complete collapse of the column.

3. Columns retrofitted with a steel jacket connected to a base plate provided an effective retrofitting method and enabled the calculated flexural strength
of columns with well-anchored bars to be achieved. This retrofit also enhanced the shear strength and the ductility of the columns.

4. Yielding of the base plate led to a very stable response and was a considerably better source of energy dissipation than yielding of anchor bolts.

5. For a yielding base plate, the plate thickness should be determined assuming that the plate is fixed at the bolt line and at the toe of stiffeners, and that the plastic moment capacity of the plate is developed at both ends.

6. The primary role of steel jacket in the retrofitted specimens was to improve the column shear strength and to provide moment transfer to the footing through stiffeners and anchor bolts. This is unlike the role of jacket for columns with sufficiently anchored bars in the footing. In these columns, the steel jacket plays two roles; one is to provide confinement and hence improve the plastic hinge performance, and the other is to improve the column shear strength.

7. Fracture of weld connecting the steel jacket to the base plate and also bearing failure of concrete can cause a significant reduction in column lateral capacity and can lead to a strong pinching in the column hysteretic response and should be avoided.

8. In general, one-way hinged base columns have lower energy absorption capacity than fixed base columns, even after they are retrofitted.

References


