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Effect of Displacement Histories on Lightly Confined Reinforced Concrete Bridge Columns

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Effect of Displacement Histories on Lightly Confined Reinforced Concrete Bridge Columns

Submitted in partial fulfillment of requirements for Graduation with Honors to the Department of Mathematics, Engineering, and Computer Science, Carroll College, Helena, Montana

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Abstract

This research involved the study of seismic behavior of older reinforced concrete bridge columns. The column reinforcement was modeled on columns used in the state of Washington prior to the mid 1970s. These columns were characterized by small quantities of hoop steel, poor confinement, and non-ductile behavior. The goal of the research was to determine the effects of cyclic load history on the resistance of such columns to lateral loading. Four specimens have been tested in a previous phase of the work, but the displacement histories used did not differ enough to determine the relative importance of two possible characterizing parameters: maximum displacement and cumulative plastic displacement. The current phase of the work involves constructing and testing two additional columns, subjected to very different load histories, to clarify the role of each parameter. Further analysis will be needed to determine the influence of each parameter on the column resistance.
Acknowledgements

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The author would also like to thank University of Washington graduate student Tyler Ranf for his hard work, tireless energy, and amazing leadership throughout the project.

Thank you also to Dr. John Scharf, my academic advisor at Carroll College, for all of his guidance and input regarding this research experience, to Dr. Cheryl Conover for her patience and technical writing expertise, and to Larryn Krause for all her time spent reading and editing this paper.

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1 Introduction

1.1 History of Earthquakes in Washington State

Due to the subduction of the Juan de Fuca plate beneath the North American plate (Figure 1.1), Washington State is considered a highly seismic area (Noson, Qamar, and Thorsen, 1988). Strike-slip faults are shallow while subduction faults are deeper and prone to produce earthquakes larger in magnitude and longer in duration. Therefore, it is necessary to determine if subduction earthquakes with long durations, which might produce a large number of loading cycles, would cause more damage to older bridge columns than an earthquake from a strike-slip fault.

Figure 1.1 Subduction of Juan de Fuca Plate (Noson, Qamar, and Thorsen, 1988)
1.2 The Project

Jared Nelson and Zachary Price, former University of Washington graduate students, investigated the influence of displacement history in their master theses (Nelson 2000); one objective of this current research was to add at least two more data points to the existing data set to see if a clear trend could be determined between cyclic displacements histories and the damage accumulation in a column. Two prototype columns were built and tested at the University of Washington.

1.3 Objectives

This paper will explain how the columns were constructed, how the data acquisition system was setup, what tests were performed, and what initial conclusions can be drawn from the expanded data set.
2 Prototype Column

2.1 Design History

A standard pre-mid-1970s Washington State highway bridge column was selected as the prototype. Such a column was chosen because a large number of bridges were built before the mid-1970s at which time the design requirements changed (Figure 2.1). The design requirement changes specified that bridge columns use more steel and a higher strength of concrete. This figure also shows that older bridges were more likely to be damaged by the 2001 Nisqually Earthquake.
Figure 2.1 Bar Graph of Number of Bridges Built in each Decade (Nelson, 2000)
2.2 Design Considerations

The column was characterized by three main components: hoop spacing, amount of transverse steel, and the strength of the concrete. Currently the Washington State Department of Transportation (WSDOT) design specifications do not allow hoop reinforcement, but instead, require spiral reinforcement of grade 60 steel with a pitch of no more than three inches. WSDOT also requires that the longitudinal reinforcement be of grade 60 steel. Prior to the mid-1970s the WSDOT design code specified that grade 40 reinforcement could be used with a hoop spacing of no more than one foot be used. Comparing the two design specifications leads to the conclusion that the amount of steel used in a present day column would be dramatically increased from the amount of steel used prior to 1975. Furthermore, in 1980, WSDOT specified that the strength of the concrete be at least 4000 psi, whereas prior to the mid-1970s a specification of 3600-4000 psi was given (Nelson, 2000).
3 Test Procedure

The test procedure, as well as the column specifications, had already been established during previous tests (Nelson, 2000). One of the main elements of this test procedure was scaling down the test column from the prototype column by a factor of three. This needed to be done in order to fit the column into the testing apparatus; also it is impractical to build three full-size columns in the lab.

3.1 Construction Stages

3.1.1 Column Forms

The columns were constructed in three stages. First, timber forms were built for the columns' footings. Footings were used to anchor the columns to the strong floor. A Sonotube—which is a thick cardboard tube—was ordered to form a kicker (Figure 3.1). The kicker was used to anchor another sonotube that was the same height as the test column. Column forms were built for the hammerhead; the hammerhead's sole purpose was to attach the top of the column to a hydraulic actuator.
3.1.2 Steel Cages and Concrete Placement

Next, steel cages were built for the footing. The longitudinal reinforcement for the column was attached to the footing cages using a wooden template and tie wire. These cages were then set into the footing forms, which were oiled to prevent the concrete from adhering to the sides of the forms. The cages were secured by tie wire and chairs and were ready for the first concrete pour. After the first pour, the hammerhead forms were attached to the footing forms using wooden bracing. Then hoop steel was attached to the column longitudinal reinforcement, and the hammerhead steel cages were placed inside the forms and braced with chairs. After the column form and hammerhead were added to the footing, the entire column form stood over eight feet tall. A hopper attached to the high-bay crane was filled with concrete and then raised above the hammerhead and emptied into the column.

Figure 3.1 Column Forms and Column Reinforcement
3.2 Testing of Columns

For both concrete placements, cylinders were made for testing the strength at various ages of the concrete. It was necessary to vibrate the concrete during placement to ensure that no air pockets developed which would decrease the strength of the concrete. The cylinders were tested at 3, 7, 14, and 28 days after the first and second pours. After 28 days the concrete had reached a strength of at least 6000 psi; therefore, the columns had reached a strength at which they could be tested. The test set-up is shown in the diagram below (Figure 3.2). A hydraulic actuator connected to a data acquisition program collected the data. Also, potentiometers measured the displacement of the column at various heights (Figure 3.3).
Figure 3.2: Test Assembly (Nelson, 2000)
Figure 3.3 Transitional Displacement Potentiometer Layout (Nelson, 2000)
Before the column and testing apparatus were connected and the final set-up began, hydrostone was floated underneath the footing and on top of the hammerhead to ensure that the column was perfectly level with the lab floor.

High-strength Williams rods were used to anchor the footing to the floor; these were tightened down to the strong floor using a jack. Williams rods were also used to anchor down a lateral beam on top of the hammerhead. The lateral beam and Williams rods supported a jack and a load cell. The jack and load cell were used to create a vertical force of 160 kips on the column which simulated the weight (or dead load) of a bridge. The load cell was also connected to the data acquisition system to monitor and maintain the load. A spherical bearing was placed between the lateral beam and the hammerhead to ensure that the force on the column was always vertical.

Before the column could be tested, the cyclic loading histories had to be determined. Tyler Ranf, a graduate student at the University of Washington, determined the histories based on a multiple of the cracking moment of the concrete, $M_{cr}$, and the yield displacement, $\Delta_y$. Table 1 shows how the displacements were computed and the corresponding drift ratios.
Table 1. (Nelson, 2000)

<table>
<thead>
<tr>
<th>Displacement Level (inches)</th>
<th>Drift Ratio (percent)</th>
<th>Significance of Displacement Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.04</td>
<td>0.07</td>
<td>0.5 $M_{cr}$</td>
</tr>
<tr>
<td>0.16</td>
<td>0.27</td>
<td>$0.5(M_{cr} + M_y)$</td>
</tr>
<tr>
<td>0.29</td>
<td>0.48</td>
<td>1.25 $\Delta_y$</td>
</tr>
<tr>
<td>0.46</td>
<td>0.77</td>
<td>2 $\Delta_y$</td>
</tr>
<tr>
<td>0.69</td>
<td>1.15</td>
<td>3 $\Delta_y$</td>
</tr>
<tr>
<td>1.14</td>
<td>1.90</td>
<td>5 $\Delta_y$</td>
</tr>
<tr>
<td>1.83</td>
<td>3.05</td>
<td>8 $\Delta_y$</td>
</tr>
<tr>
<td>2.29</td>
<td>3.82</td>
<td>10 $\Delta_y$</td>
</tr>
<tr>
<td>2.74</td>
<td>4.57</td>
<td>12 $\Delta_y$</td>
</tr>
<tr>
<td>3.43</td>
<td>5.72</td>
<td>15 $\Delta_y$</td>
</tr>
</tbody>
</table>

The displacement histories that were used for the first four columns are shown on the following page (Figure 3.4). The results from using these displacements were compiled to create a data set. Using this data set it was difficult to pinpoint where spalling and bar buckling occurred. It was decided that instead of increasing the displacement every 3 cycles, as shown in Figure 3.4(a), the displacement would be increased every half cycle. This will make it easier to determine exactly where and at what displacement significant spalling and bar buckling occurred.
Figure 3.4 Displacement Histories for the First Four Columns (Nelson, 2000)
4 Set-up of Data Acquisition

4.1 Previous Data Acquisition Set-Up

The first four columns that were tested previously included strain gauges inside the column (Figure 4.1) as well as rotational and shear potentiometers (Figure 4.2). The previous set-up was not duplicated because the data that the instruments measured was not vital to the objectives of this research. The test set-up of the translational displacement potentiometers for the last two columns was identical to that used for the previous four columns.
Figure 4.1 Strain Gauge Layout (Nelson, 2000)
4.2 Data Acquisition System

The translational displacement potentiometer set-up (Figure 3.2) includes the reference column on which the transitional displacement potentiometers were mounted. An independent reference column was necessary during testing to ensure that the transitional displacement potentiometers measured only the displacement of the column. Displacements were measured at heights of 10, 20, 30, and 60 inches from the column's
base. These heights were selected because most of the spalling and bar buckling occurs towards the base of the column. Two transitional displacement potentiometers were mounted just above the floor to measure the displacement of the footing. If the footing moved at all, these potentiometers would measure the displacement of the footing and a correction value could be calculated to determine the actual displacement of the column.

Potentiometers measure displacement by means of an internal circuit, which measures the resistance created by the displacement of the column. The resistance is converted into a voltage and is entered into the data acquisition system, which is run on a computer with compatible software. The data acquisition system software used by the University of Washington is LabView. The equations used to convert the voltage to an actual displacement is:

\[
\text{Displacement} = (V_o - V_i) * C_f
\]  
(Eq. 4.1)

where \( V_o \) is the output voltage, \( V_i \) is the initial voltage, and \( C_f \) is the calibration factor.

Each potentiometer was calibrated before the tests were run, so each potentiometer had a particular calibration factor.

A spreadsheet was set up with all of the calibration factors. Potentiometers were in one column with the corresponding calibration factor in the next column. The data could then be exported into the spreadsheet and the correction factors could be applied to the data. LabView recorded not only the displacements, but also the voltage outputs and initial voltages which allows for the computations to be verified.
5 Results

The results of the first four columns are shown in blue (Figure 9). A graph of displacement versus cumulative plastic displacement shows where first yield, significant spalling, bar buckling, and loss of axial load occurred. Analysis of the first four columns shows a trend between columns S3, C3.7-R, and C2.3-F, which lost the capability to support any axial load. These three columns all failed in flexure-shear, whereas column number C4.7-F failed primarily from flexure. Columns S1 and S15 also failed in flexure-shear; however, the data has yet to be analyzed, so it is undetermined when loss of axial load occurred. Column S1 and S15 are shown in fig. 9 in red.

Figure 9. Displacement-Energy Curves
6 Discussion and Conclusion

Results of this research could lead to more effective structural assessments of Washington State Bridge columns which would decrease the number of bridges that could be damaged in a major earthquake, as well as the potential injury or loss of life during an earthquake. Graduate student Tyler Ranf is extending the research and continuing previous work by calibrating damage models to better estimate the effects of an earthquake on the columns.

While I was involved in this research during the summer of 2003, one of my tasks was to set up the data acquisition system. I spent several days calibrating the potentiometers, recording the calibration factors, and connecting the potentiometers to the computer to record the test data in LabView.
References

