CONTINUITY OF PRECAST BRIDGE U-GIRDERS CONNECTED TO A CAST-IN-PLACE SUBSTRUCTURE SUBJECTED TO SEISMIC LOADS

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ABSTRACT:

It is critical in high seismic regions to have full connectivity between the girders and the support columns to transfer the high seismic forces. Using precast girders for the superstructure allows for better construction tolerances and quicker construction methods. The purpose of this study is to develop and examine integral connection details between precast U-Girders and cast-in-place substructures subjected to longitudinal seismic loading. Analytical modeling and experimental cyclic testing of four, 40 percent precast U-girder specimens are used to investigate the behavior of the connections. This study primarily investigates the effect of post-tensioning on the connection performance. Analytical and experimental results show benefits to longitudinal post-tensioning in respect to joint detailing and system continuity which aren't considered in current design methods. Additionally, potential local joint cracking problems need to be addressed in joint design methods.

KEYWORDS: Precast, Seismic, Bridge, Post-Tensioning, Connection

1. INTRODUCTION

Bridge structures are an integral part of the nation's highway infrastructure and as the infrastructure continues to age, existing bridges will need to be widened, retrofitted, or replaced, and new bridges built. Often, these infrastructure improvements will occur in heavily congested areas where traffic delays and public safety are of major concerns. In regions of high seismicity, bridges typically have a cast-in-place concrete superstructure integrally connected with a cast-in-place substructure in order to transfer the high seismic moment and shear forces. This monolithic bridge construction provides good continuity for transfer of seismic forces; however, falsework over the traffic lanes is needed while the superstructure is cast which potentially can reduce bridge clearance over the traffic lanes, traffic rerouting disrupting traffic flow, and increased site construction time; all of which is dangerous for both construction workers and motorists. Using precast concrete girders for the superstructure eliminates the need for falsework over traffic lanes and also allows for accelerated construction time at the job site, thereby reducing traffic delays and the danger to construction workers and motorists. However, the lack of experimental data on precast girder integral connection behavior subjected to seismic forces has led designers and agencies to either over-design these types of connections or not use them at all (Holombo et al., 2000). The purpose of this study is to investigate the longitudinal seismic behavior of the integral connection between precast concrete girders and cast-in-place concrete and develop design guidelines based on analytical and experimental testing for the Nevada Department of Transportation (NDOT).

2. BACKGROUND

The first objective of this research was to gather and process information regarding precast girder connection details in order to build an appropriate experimental program. This was accomplished through a survey, literature search, and computer analyses.

2.1 Department of Transportation Survey

To gain insight on current practices, a survey of typical connection details and construction techniques for integral connection type bridges was sent to Department of Transportation (DOT) agencies in high seismic regions. DOT departments utilizing this connection most commonly used post-tensioning in the negative moment region and either/or extended strands and mild reinforcement mechanically or lap-spliced connected in the positive moment region. When post-tensioning was not used, more reinforcement in the deck was placed for additional negative moment capacity. The drawback of using strands is its tendency to slip under cyclic loads (Miller et al., 2004) thus requiring some sort of mechanical connection or relying on lower transfer stresses that are not always easy to determine with a degree of confidence.

2.2 Literature Search

Prior research at the University of California, San Diego (UCSD) (Holombo et al., 2000) investigated the continuity of a post-tensioned spliced precast girder system (using both bulb-tee and bathtub girders) subjected to longitudinal seismic forces using California of Transportation (Caltrans) seismic design criteria (Caltrans 2004). Experimental results showed the column was able to achieve the desired ductility level while the superstructure remained essentially elastic. It was also recommend that the column moment should be proportioned to the girders according to relative stiffness, or roughly two-thirds of the column moment should be resisted by the adjacent girders, and the other one-third resisted by the remaining girders. They also recommend extending the column reinforcement into the bent cap as far as possible to assist in joint transfer. Experimental tests at UCSD showed that a precast integral bridge system performs well using current design guidelines; however, since the superstructure remained elastic, it is hard to quantitatively assess the girder connection into the bent-cap. Therefore one of the objectives of the current study is to isolate the girder connection into the bent cap and evaluates the failure mechanisms.

2.3 Connection Parameter Study

To better understand the behavior of the connection parameters discussed in the previous two sections, strength and ductility characteristics of the parameters were analyzed using the cross-sectional software program XTRACT, 2002. The post-tensioning detail is very advantageous because it allows the section to have a high negative moment capacity without having to increase reinforcement amounts in the deck. However, a drawback of post-tensioning in continuous systems is the introduction of positive secondary moments in the joint region. High secondary moments are not desired due to the limited space in the joint positive moment region to place reinforcement. These secondary moments can be minimized and controlled by the designer through proper tendon configurations. Not only is post-tensioning advantageous for negative moment capacity, it also increases positive moment capacity, so if second order moments are small, reinforcement requirements can be minimal in the joint positive moment region. In the positive moment region of the joint, the easiest connection is lap-splicing mild-reinforcement through the bent-cap. As mentioned above, the use of strands in this region requires a mechanical connection generating more work and congestion in an already highly congested area. Also, from a ductility standpoint, mild-reinforcement is more desirable than strands, which can be important in seismic design.

From this investigation, the most important parameter was the amount of post-tensioning used in the girders. Therefore, four precast U-girders were ultimately selected for experimental investigation using varying amounts of post-tensioning in the negative moment region of the girder connection and mild-reinforcement spliced through the bent-cap in the positive moment connection region.

3. EXPERIMENTAL TESTING PROGRAM

The experimental program consisted of testing four, 40% scale bridge specimens to investigate the ability of the integral connection to transfer seismic forces in the longitudinal direction between the substructure and superstructure. A 40% scale was chosen based on the availability of rebar and the smallest dimension size we were willing to construct and test. After an inelastic dynamic analysis using SAP2000, 2007, it was determined that the 40% scale factor chosen could be tested with actuators at each end of the bridge to simulate the seismic loading. Figure 1 shows the test configuration adopted for this study. Notice that mass was added on both spans; this was necessary to achieve the correct scaled dead load and secondary moments in the region near the bent-cap. Since recommendations for contributory superstructure (Holombo et al., 2000) stiffness to resist the column moment were made from the researchers at University of California at San Diego, and due to budget and equipment restraints, only one girder on each side of the bent cap was modeled. As mentioned above, the main objective of the experimental program was to isolate the girder connection and look at the failure mechanisms with the primary parameter being the amount of girder post-tensioning. To achieve this objective, the first three specimens, UGHP, UGLP, and UGNP were designed to have varying amounts of post-tensioning and inelastic behavior in the superstructure at the girder/bent-cap interface while allowing limited inelastic behavior in the column. For comparative purposes, the column demand was targeted at 75% of the ultimate column moment. This ensured more direct comparisons between each of the first three tests. After initial experimental results from UGHP and UGLP, it was decided the fourth specimen, UGHPM, would have the same girder connection details as UGHP, but a reduced column size. This resulted in having all the inelastic behavior in the column while the superstructure remained elastic (similar to the tests at UCSD (Homolbo et al., 2000). Essentially this test verified the connection details as it would be built in the field.

Each specimen was subjected to reverse cyclic loading consistent with guidelines given in the *Recommendations for Seismic Performance Testing of Bridge Piers* (FHWA, 2004). The cycles were run in force control until ¾ of the yield displacement, thereafter, the cycles were run in displacement control until failure. Figure 2 shows the loading protocol for specimen UGHP. Protocols for the other specimens follow the same trend as shown in Figure 2.

4. SPECIMEN DESIGN

The specimens were designed using the American Association of State Highway and Transportation Officials (AASHTO, 1998). If design guidelines other than AASTHO were used, the proper reference is cited.

4.1 Girder Design

For the girder design, scaled versions of NDOT girders were used with the connection details mentioned above. The first specimen UGHP was the closest representation to the NDOT girders with a post-tensioning level of $0.15f'_cA_g$. UGLP contained 25% less post-tensioning ($0.11f'_cA_g$) than UGHP, and UGNP containing no post-tensioning. Again each girder was designed for a column demand moment of 75% of the ultimate demand. The girder for UGHPM had the exact same details as UGHP. Girder sections are shown in Figure 3(a) for specimens UGHP, UGLP, and UGHPM, and Figure 3(b) for specimen UGNP.

4.2 Column Design

As mentioned previously, for specimens UGHP, UGLP, and UGNP, the column (shown in Figure 4(a)) was designed in order to limit the column inelastic response while plastic hinges formed at the girder bent-cap interface. However, in the fourth specimen, a new column was designed representative of a column that would be used in actual design. In other words, the column would contain the inelastic behavior while the superstructure remained essentially elastic. To do this, the column was designed for the girder capacity divided by a factor of 1.3. The 1.3 factor is required by AASHTO, 1998, to ensure the girders will remain essentially elastic. Figure 4(b) shows the column used for the fourth specimen (UGHPM).

4.3 Bentcap Design

The bent cap for specimens UGHP, UGLP, and UGNP was designed using Caltrans Seismic Design Criteria, (Caltrans, 2004), since AASHTO, 1998, does not provide a clear design procedure for joint design where seismic forces are transferred between the substructure and superstructure. The details contained in the Caltrans specifications are similar to those given in the book *Seismic Design and Retrofit of Bridges* (Priestley et al., 1996) and Prestressed Concrete Institute (PCI, 2003). This method was used in the experimental study done at the University of California, San Diego and the bent cap performed adequately. Caltrans uses the idea of external joint stirrups to transfer the column tension into the superstructure. It is important to note that since the column was over-designed for specimens UGHP, UGLP, and UGNP, the amount of joint stirrups provided was based on the column moment demand required to produce failure in the superstructure, not the nominal capacity of the columns required by Caltrans. The bent cap reinforcement for specimen UGHPM was designed based on preliminary results from the first three specimens and was detailed for the minimum reinforcement required by AASHTO in D-regions, which is less the requreiment using the Caltrans method. External joint stirrups for each specimen are shown in Figure 5. The diagram on the left shows an elevation view of the external joint stirrup set, while the diagram on the right shows a top view of the joint depicting the number of stirrup sets. It should be noted that modifications to the Caltrans method are proposed by Sri Sritharan, 2005. Sritharan's research suggested that the model proposed by Priestley et al., 1996 was conservative when the joint was prestressed and unconservative when the joint contained no prestress. According to his research, some joint stirrups are needed for the partially prestressed case and no joint stirrups are needed for the fully prestressed case, only a nominal amount for crack control. His conclusions were based on a series of experimental tests investigating each the cases presented above. It should be mentioned that these models work well for solid sections that are directly in line with the column. However, these 2-D representations may not necessarily be adequate for non-solid sections such as a U-girder or when the girder is not directly in line with the column.

5. EXPERIMENTAL RESULTS

5.1. UGHP, UGLP, UGNP

Hysteresis curves for UGHP, UGLP, and UGNP are shown are shown in Figure 6. Based on analytical work, it was predicted that plastic hinges in the girders would form on both sides of the joint with the negative moment region of the girder reaching its rotation capacity. Strain gauge data indicated significant reinforcement yielding in both the positive and negative moment regions of the girder for UGHP, UGLP, and UGNP. However, in each test, a large crack developed at the top of the joint that limited the lateral load capacity of each specimen. Due to load cycling, the crack extended horizontally across the entire width of the joint in each test. After the crack formed, the girder forces were redistributed and the girder positive moment region lost rotation capacity. This crack formed at $1.75\Delta_y$ for UGHP and started in the girder flange and extended into the joint. Notice from the hysteresis plot that the lateral load dropped significantly during the second cycle of the final load stage. For UGLP, the crack formed at $2.5\Delta_y$ and started in the joint. The hysteresis plot shows a more gradual reduction in lateral load capacity for UGLP than UGHP, suggesting a more ductile response which was expected. For UGNP, the joint crack occurred during load stage $2.0\Delta_y$ and again was initiated in the joint. The failure was more ductile, similar to UGLP. For each test, the subsequent load cycle resulted in a lateral load drop of about 10%. Each specimen continued to carry appreciable lateral load until ultimately concrete crushing and longitudinal reinforcement buckling occurred in the girder positive moment region. Cracks in the joint first started to appear around Δ_y for UGHP, $0.75\Delta_y$ for UGLP, and $0.60\ \Delta_y$ for UGNP. UGNP displayed more joint cracking than UGLP, and UGLP displayed more joint cracking than UGHP. External joint stirrup strain gauge data for specimens UGHP and UGLP were below yield with values for UGLP 40% higher than UGHP. UGNP external joint stirrups showed joint stirrups were just starting to yield with values 30% higher than UGLP. In addition, the joint in specimen UGNP displayed the most visible damage while UGHP displayed the least. Pushover curves obtained with SAP2000, 2007 also agree well with the experimental results indicating the ability to achieve nearly the full strength of the connections.

5.2. UGHPM

The hysteresis curve for UGHPM is shown in Figure 6. This test was to verify the connection details in the case when the failure occurs in the column, as is the conventional bridge design method. As in the previous tests, the girder connection details into the bentcap were adequate to transfer the high seismic forces between the substructure and superstructure. Specimen behavior through load stage $2\Delta_y$ was as expected from preliminary analysis predictions; the column cover started to spall and inelastic behavior was essentially isolated to the column. However, during the next load stage, $3\Delta_y$, a large horizontal joint crack again developed as it did during the previous specimens and limited the lateral load carrying capacity of the specimen. At this point, damage no longer occurred in the column and similar girder force redistribution that occurred in the previous specimens occurred in UGHPM. Also similar to the previous speicmens, a 10% decrease in lateral load capacity occurred during the next load stage, $4\Delta_y$. During this load stage, concrete cover spalling in the girder positive moment region started to occur. Further increases in lateral displacement were not possible after this load stage because the displacement capacity of the test set-up was reached.

5.3 Comparisons

To compare the connection ductility of test results, a plot of percent maximum lateral load vs. displacement ductility is shown in Figure 7 for cycles 1 and 2 for each load stage starting at a displacement ductility of 1. The main observation from this figure is that each connection detail performs adequately and would work for design. Specimen UGLP shows the best ductility performance when comparing UGHP, UGLP, and UGNP; while UGHPM is the most ductile overall. From a design standpoint however, connection ductility is not of great concern since both Caltrans, 2004 and AASHTO, 1998 essentially require the superstructure to remain elastic, therefore ending up with designs similar to UGHPM. Therefore, for design, UGHP would be more desirable since less joint damage was observed, suggesting that higher post-tensioning has a beneficial effect for joint design. This finding is consistent with Sritharan, 2005, suggesting that current design guidelines for joint design are over-conservative for joints with post-tensioning. With that said, there is also a local problem with the joint reinforcement due to the large horizontal crack that developed during each test. More specifically, the joint crack initiated on the beam compression area on the positive seismic moment side of the joint; since the joint crack occurred at different levels of lateral load, the reinforcement in this region is inadequate in terms of crack control.

6. CONCLUSIONS

This paper presented research relating to experimental work on seismic testing of precast U-girders integrally connected to cast-in-place bent caps. The research primarily investigates the effect of longitudinal post-tensioning on connection behavior. Based on work conducted to date, the following conclusions can be made:

1. Girder connection details with or without post-tensioning for negative moment capacity and spliced mild-reinforcement in the positive moment region are effective details for design,
2. Current joint design methods are conservative for joints with post-tensioning applied, and do not specifically address problems with local crack control,
3. The greatest connection ductility of the first three tests was exhibited by specimen UGLP, which could be useful in dissipating seismic energy.

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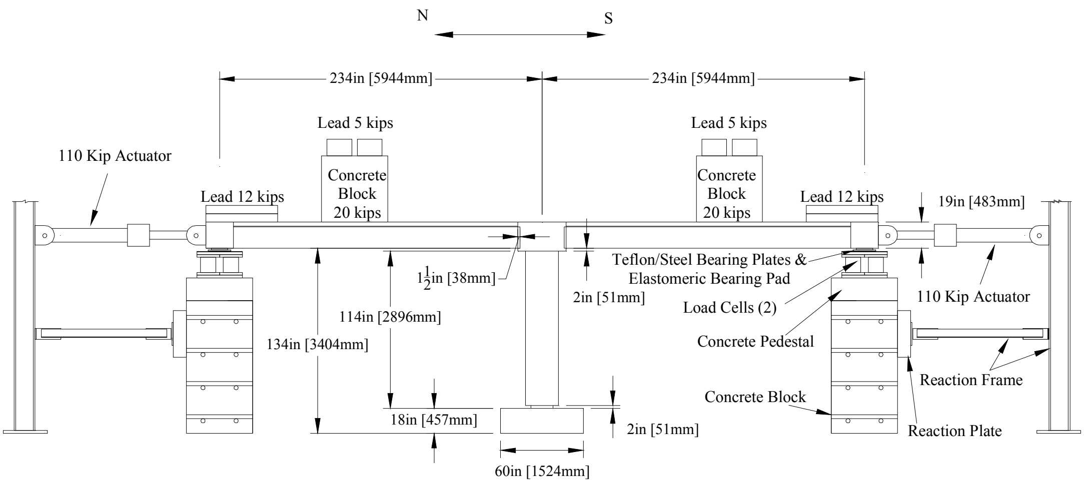

Figure 1: Experimental Test Set-Up

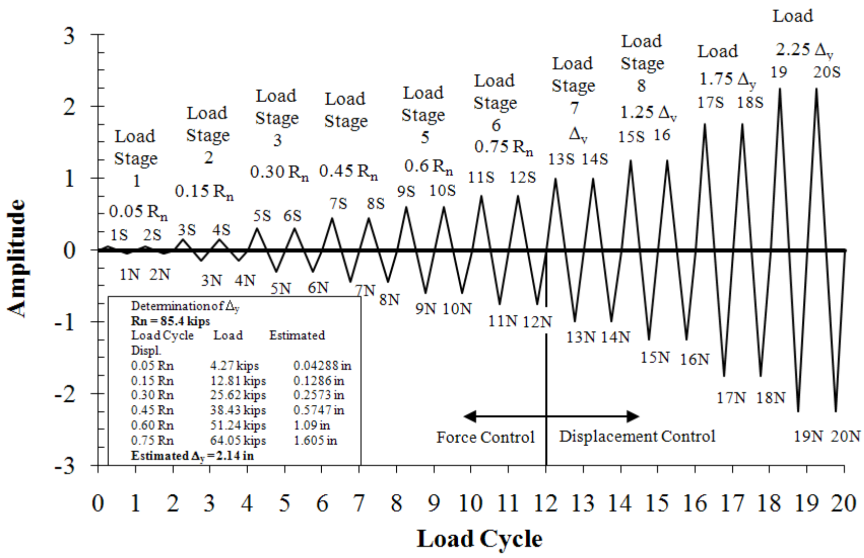

Figure 2: UGHP Reverse Cyclic Lateral Loading Protocol

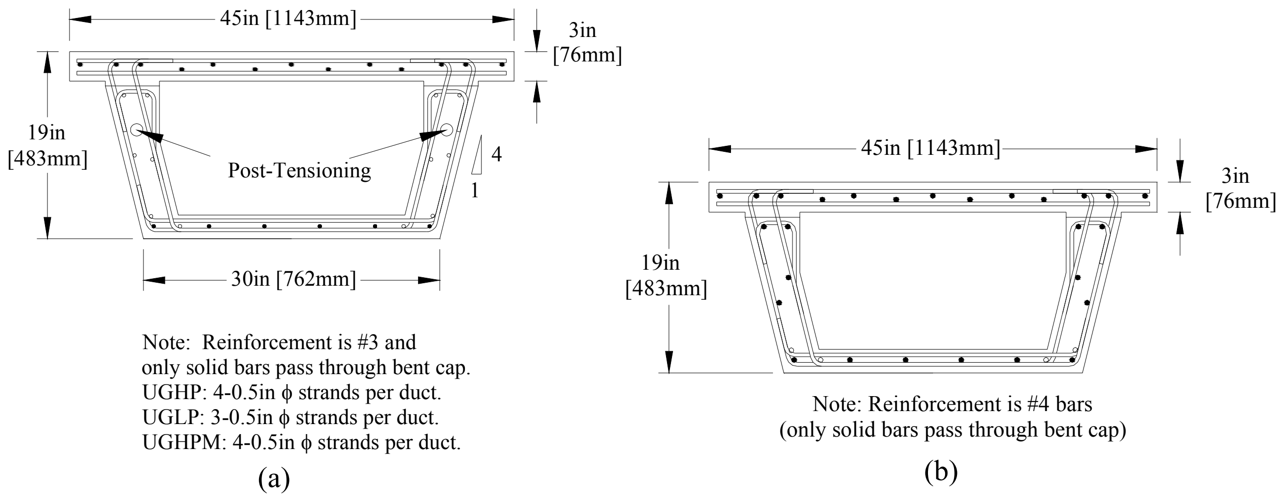

Figure 3: a) Girder section UGHP, UGLP, UGHPM; (b) UGNP

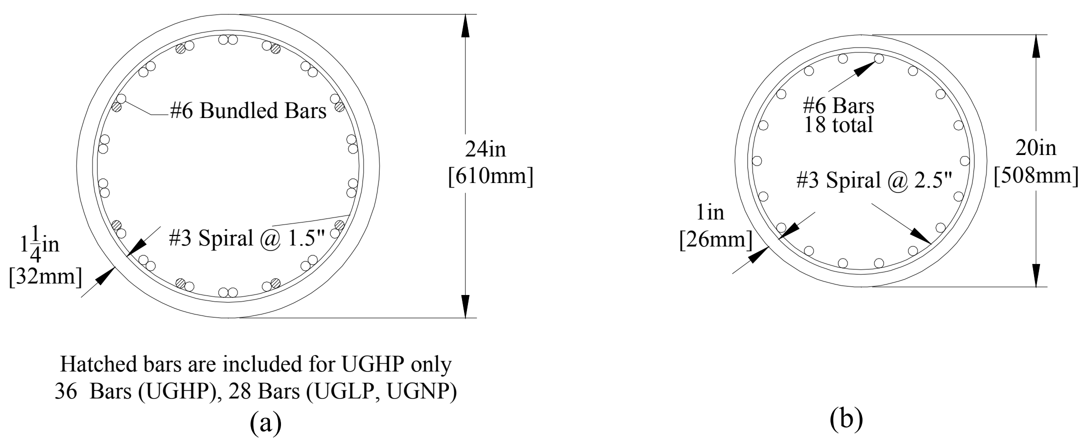

Figure 4: (a) Column section UGHP, UGLP, UGNP; (b) UGHPM

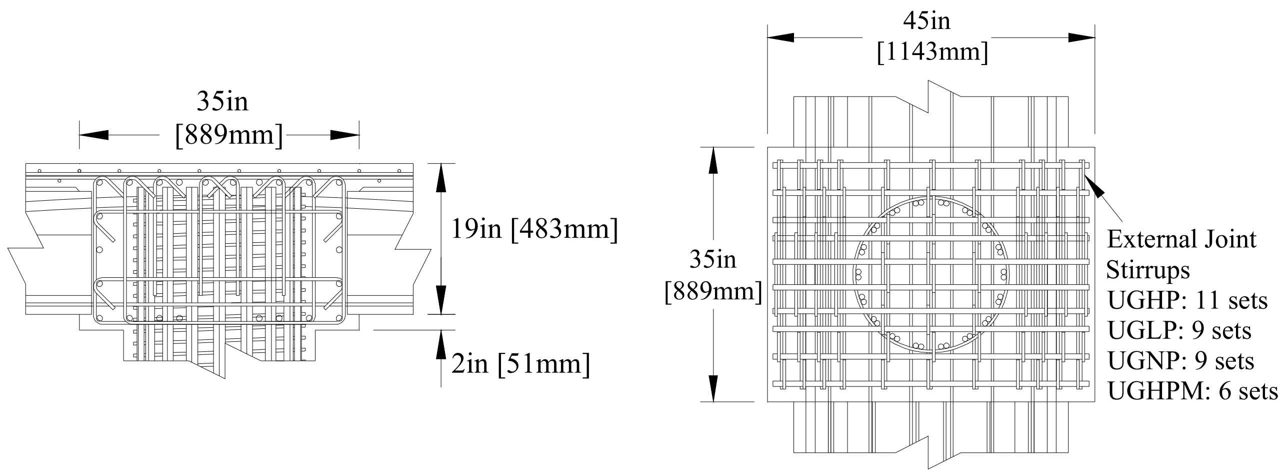

Figure 5: Joint Reinforcement

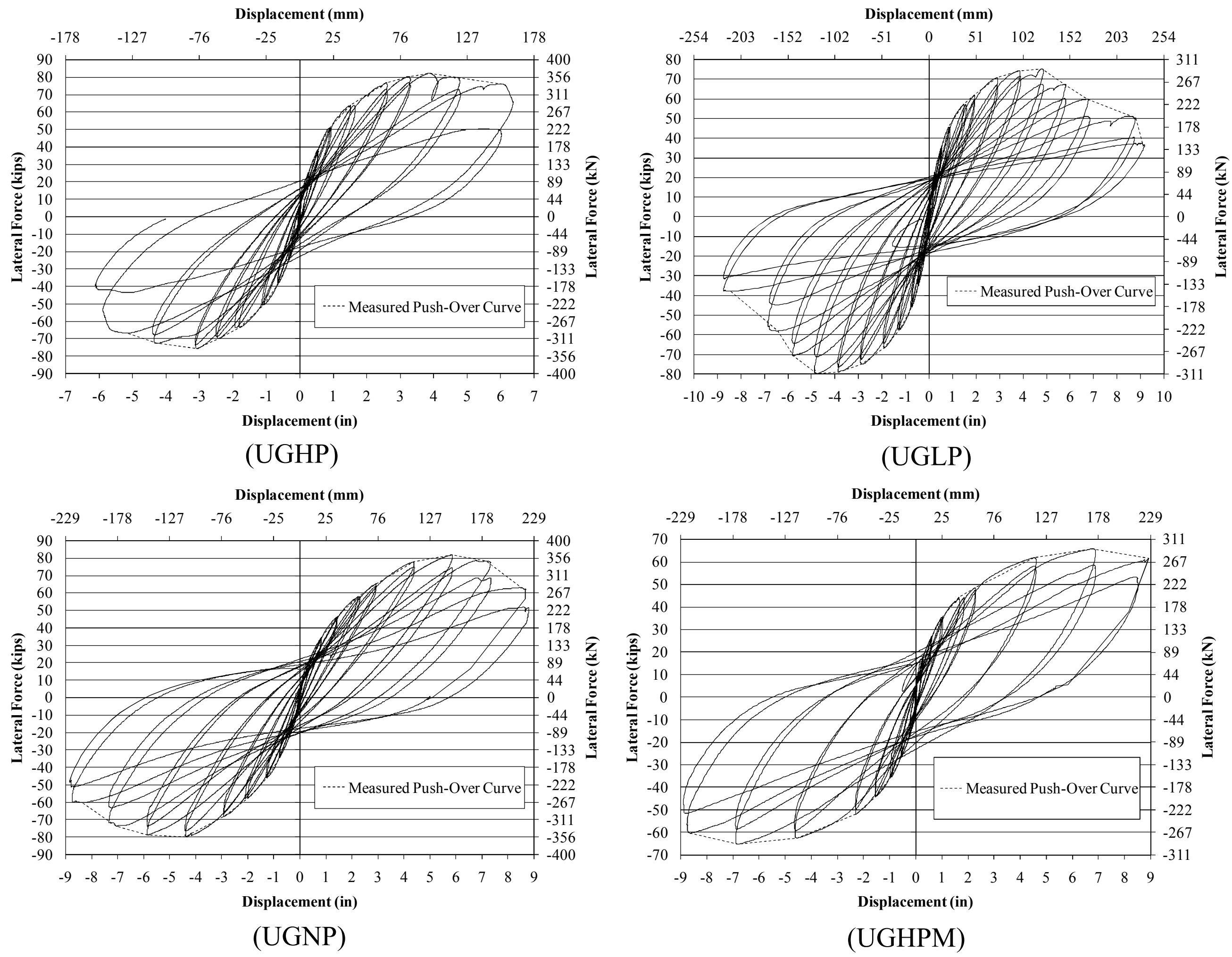

Figure 6: Specimen Hysteresis Curves

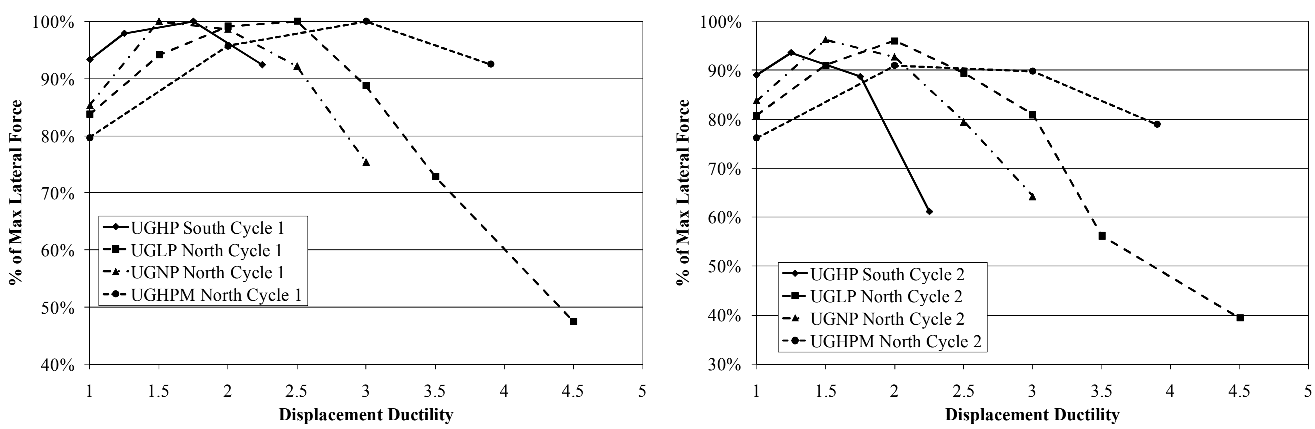

Figure 7: Connection Ductility Comparison