LARGE SCALE TESTING OF A LOW DAMAGE SUBSTRUCTURE CONNECTION IN A PRECAST CONCRETE BRIDGE

LIU, R. & PALERMO, A.

Department of Civil and Natural Resources Engineering, University of Canterbury, NZ

SUMMARY

At the University of Canterbury, a 1/3 scale, two span, fully precast concrete bridge using a low damage, cantilever, hammerhead, PRESSS/Dissipative Controlled Rocking pier was tested under transverse cyclic loading. This experiment was part of a two phase project under a parent program entitled Accelerated Bridge Construction and Design (ABCD). The aim of the experiment was to ascertain the seismic performance of a full bridge system using PRESSS technology. The work presented in this contribution is unique internationally due to the large scale of the specimen tested, the fidelity to which the specimen reproduces the details of a real bridge, and the testing of a low damage technology within a system rather than a component. The focus of this paper is in three main areas: the design and construction challenges experienced with the DCR pier used, in addition, to some of the solutions developed; presentation of experimental results related to the effects of different variables (e.g. dissipation device locations, post-tensioning levels in pier and deck) on specimen performance; and presentation of important observations related to observed compatibility issues found to occur between bridge components, which, have far reaching consequences for the transverse seismic performance of not only bridges using hammer head DCR piers, but, hammer head pier bridges in general, especially, short to medium span, wide deck, beam bridges.

INTRODUCTION

The use of precast construction for concrete bridges offers many advantages. It accelerates construction thus reducing traffic disruption; there is less environmental impact due to the reduced need for machinery, equipment, and construction area; the quality and dimensional accuracy of construction is improved due to those attributes being able to be tightly controlled under factory conditions; and reduced life cycle costs due to higher quality construction increasing the component durability (Billington, Barnes, & Breen, 2001; Culmo, 2011).

But the benefits of precasting don’t just end there, for even the seismic performance of bridges using precast substructure components such as piers/columns can be designed to be superior to that of conventional construction (Figure 1 a & b). This is due to the high compatibility which precast construction has with low damage design strategies such as the PREcast Seismic Structural System (PRESSS) (Priestley, 1996) also known as Dissipative Controlled Rocking (DCR). DCR involves replacing member plastic hinging with rocking of the member at a joint and combining the aforementioned with internal unbonded post-tensioning for self-centering behaviour (to minimise residual displacements) and dissipative devices across the rocking joint to provide damping (Figure 1 c).
Past experimental research on the application of DCR to concrete bridge columns have shown such technologies to significantly improve seismic performance in comparison to their monolithically constructed or monolithically emulative counterparts in terms of restricting damage to internal or external devices and eliminating the majority of residual drift (Guerrini, Asce, Restrepo, Massari, & Vervelidis, 2015; Marriott, 2009; Moustafa & ElGawady, 2016; White & Palermo, 2016). However, there are two facets of this technology which still warrant further investigation. The first is with regard to construction detailing especially for the case where external devices are used and the second is that all of the tests conducted so far have been on bridge components and not full systems. Therefore there is a current lack of knowledge with regards to system effects such as unknown interactions between the behaviour of the DCR column and the rest of the bridge. This paper addresses these two points focussing more on the second through presentation of the results obtained from testing of a bridge system.

**SPECIMEN DESCRIPTION**

The specimen is a 1/3 scale, two span, fully precast, simply supported bridge with single hammerhead DCR pier (Figure 2 & 3). It was designed and tested as part of a two phase project under the parent program titled Accelerated Bridge Construction and Design (ABCD). The specimen had spans of 4.2m, a deck width of 2.4m, and a circular pier Ø0.5m. The decks were hollow core slabs and were seated on square ultra-high molecular weight polyethylene bearings.

![Figures 2 and 3](image)

**Figure 2:** South (a) and North (b) elevations of the bridge specimen during phase 2 of the ABCD program.

The decks were only restrained in the horizontal plane (Figure 3). Wooden blocks placed between the ends of the decks and the abutment back walls prevented longitudinal movement; shear keys at the abutments restrained transverse movement of the deck ends; and shear keys either side of the decks over the cap beam prevented relative transverse movement between the deck and cap beam. At the abutment ends stacks of steel plates combined with one rubber pad was used to fill the gaps between the decks and concrete shear keys. The
choice of filler was deliberate in that the use of stacked steel plates allowed the gap to be adequately filled to minimise transverse sliding, whilst, also minimising the moment capacity of the deck-abutment connection so that it behaved more like a pin in the horizontal plane.

Figure 3: Schematic of bridge specimen for Phase 2

As previously mentioned this specimen was used in two phases of testing. Phase 1 of this project focused on creating a low damage superstructure, where the central pier only provided vertical support for the decks which were designed to slide on top of it (Chegini & Palermo, 2015). Whilst, phase 2 of the project focussed on the design and testing of the substructure within a bridge system and involved replacement of the pier from phase 1 with a new precast concrete hammerhead DCR pier. This paper describes phase 2 of this project focussing on the details of the replacement pier, the changes made in testing arrangement, and the results of testing in this phase. This contribution is a companion paper to (Chegini & Palermo, 2015) which describes Phase 1 and where further detail of the entire specimen can be found.

TESTING ARRANGEMENT & VARIABLES EXPLORED

Simulation of seismic loading was achieved through uniaxial, cyclic, quasi-static loading of the bridge in the transverse direction by a single 300kN ram. In plan view of the bridge, the ram acted at the centreline of the cap beam, whilst in elevation, the ram still acted on the cap beam but at the vertical location corresponding to the lumped mass of the idealised, transverse SDOF of the specimen (Figures 3 and 4a).

Figure 4: a) Transverse section of the bridge showing the testing arrangement. b) Loading protocol
The loading protocol used was derived from ACI T1.1-01 (ACI Innovation Task Group 1, 2001) where: three fully reversed cycles are applied at each drift ratio (ratio of deck displacement to pier length); the first drift ratio is within the linear elastic response range; and subsequent drift ratios are between 1.25 and 1.5 times the previous value (Figure 4b). The ULS drift ratio for the specimen is 2% ($\Delta = 46.5\text{mm}$) whilst the MCE drift ratio is 3.5% ($\Delta = 82\text{mm}$), however during testing, the maximum applied drift ratio that could be achieved was 2.75% ($\Delta = 63.5\text{mm}$) due to safety reasons related to the deck displacements and the limited strength capacity of the ram to cap beam loading attachment.

Test Variables

The main variables controlled were: dissipation device locations (base of pier and deck to deck joint, see Figure 3a); pier post-tensioning force; and deck post-tensioning force. A matrix describing the combinations of variables investigated is presented in the Table 1 below.

Table 1: Summary of the different configurations the bridge specimen was tested in. $F_{pu}$ is the characteristic failing load of the post-tensioning being referred to.

<table>
<thead>
<tr>
<th>Test number and configuration description</th>
<th>Pier post-tensioning kN</th>
<th>Pier base dissipators no.</th>
<th>Deck post-tensioning kN</th>
<th>Deck joint dissipators no.</th>
</tr>
</thead>
<tbody>
<tr>
<td>(T1-PT): Post-tensioned rocking pier</td>
<td>89.2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(0.16$F_{pu}$)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(T5-PT): Post-tensioned rocking pier</td>
<td>149.1</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(0.26$F_{pu}$)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(T6-PT): Post-tensioned rocking pier</td>
<td>195.7</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(0.34$F_{pu}$)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(T4-DCR): PRESSS/DCR pier</td>
<td>92.3</td>
<td>4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(0.16$F_{pu}$)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(T8-DPT): Post-tensioned rocking pier</td>
<td>95</td>
<td>-</td>
<td>40</td>
<td>-</td>
</tr>
<tr>
<td>and post-tensioned deck (2 x 15.2 dia</td>
<td>(0.17$F_{pu}$)</td>
<td></td>
<td>(0.15$F_{pu}$)</td>
<td></td>
</tr>
<tr>
<td>tendons)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(T9-DDCR): DCR pier, post-tensioned</td>
<td>95</td>
<td>4</td>
<td>40</td>
<td>2</td>
</tr>
<tr>
<td>deck (2 x 12.7dia tendons) and deck</td>
<td>(0.17$F_{pu}$)</td>
<td></td>
<td>(0.23$F_{pu}$)</td>
<td>2</td>
</tr>
<tr>
<td>joint dissipators</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

DESCRIPTION OF PIER USED IN PHASE 2

The replacement pier was 2140mm long, 500mm in diameter and used 40MPa concrete. Steel armouring at the pier base consisted of a custom made 500 x 10 CHS with an annular ring welded to the bottom (Figure 5). The CHS had 4 “H shaped” brackets welded to the outside for the direct attachment of external dissipative devices. At the centre of the pier was a 39mm internal diameter Drossbach duct which accommodated a single Ø26.5mm Macalloy bar for post-tensioning. The pier used the central post-tensioning and two grade 8.8 M30 threaded rods (Figure 5) to create a moment resisting joint between the pier and cap beam. The dissipative devices used were grooved type dissipators made from Ø16mm, grade 300, mild steel rod with a fuse area of 135mm$^2$ and a fuse length of 185mm (Figure 6). The dissipator to dissipator circle diameter was 570mm.

Design

The dimensions, design strength, and design displacement of the replacement pier were determined based off scaling of the designed prototype structure. For the seismic design of the prototype structure, direct displacement based design was used whilst NZS1170.5 was only used to obtain the seismic action design parameters. NZS 3101: 2006 (Standards New
Zealand, 2006) was used to design the reinforcing cage which used 8-HD20 longitudinal bars and HR10 hoops spaced nominally at 120mm and 150mm (Figure 5). The PRESSS handbook (Pampanin, Marriott, Palermo, & New Zealand Concrete Society, 2010) was used to size the required fuse area and fuse length of dissipator; and determine the size and initial post-tensioning force required for the central post-tensioning bar. The three main challenges with respect to design of the pier were 1) choosing a suitable thickness of the steel casing to resist forces from the dissipators; 2) ensuring composite action between the casing and the concrete, such that, when the pier rocks and a number of external dissipators go into tension that this tension force is transferred through the concrete to the internal reinforcing in tension; and 3) achieving full development of the internal reinforcement prior to it reaching the position where the dissipators are attached to the pier. The thickness of the steel armouring was chosen to be 10mm thick to minimise the possibility of local deformation of the casing around the dissipator brackets when loaded by the dissipators. Four layers of 12.7 x 79mm Nelson studs were provided on the inside of the casing to help transfer forces from the dissipators in tension to the internal reinforcement in addition to reducing the ability of moments generated by the loaded brackets from locally peeling the armouring off the concrete core. Minimization of development length was achieved by using 180° hooks and accounting for confinement provided by the casing.

Figure 5: Bottom (left) and side view (right) of the replacement pier showing half sections.

Figure 6: Typical 4 groove dissipator used at the pier base with anti-buckling sleeve.

Figure 7: a) Column base and zoom in on shear and torsion key detail; b) Freebody diagram showing the decoupling of components which provide moment, shear and torsion resistance.
For shear and torsion resistance at the rocking interface, it was decided to use external shear and torsion keys (Figure 7a) to minimise any free sliding prior to the pier contacting the shear keys as experienced by (White, Mashal, & Palermo, 2014) when they used an internal shear key. The shear keys consisted of steel blocks with an angled end facing the pier to accommodate rocking movement. These were welded to the base plate on which the column rested on. The torsion keys also consisted of the same blocks used as shear keys, except, they were welded vertically to the column casing either side of the shear keys. It is interesting to note at this point that the PRESSS system allows the decoupling of components used to provide moment and shear capacity across the rocking interface (Figure 7b).

Construction

Construction of the pier consisted of 5 main stages: 1) fabrication of steel work (Figure 8a-d); 2) assemblage of steel work (Figure 8e & f); 3) reinforcement cage construction (Figure 8g); 4) assembly of the reinforcing cage with the CHS casing, formwork and additional components (central Drossbach duct and threaded starter bars) (Figure 8h); and 5) concrete pouring (Figure 8i) and removal from the mould. The novelty in construction for this pier comes from stage 1 where major issues related to ensuring accurate bracket positioning on the casing during stage 2 and coordination of the bracket centrelines with threaded holes in the base plate (Figure 8c) (used for fixing the foundation end of the dissipators) were overcome. The casing, annular ring, bracket components and base plate were fabricated using a CNC combination drilling and cutting machine. To ensure accurate positioning of the brackets when they were welded to the finished casing in stage 2, the outlines corresponding to the bracket locations were etched onto the flat plate steel prior to it being rolled and seam welded to form the casing. The use of the CNC combination drilling and cutting machine to drill the dissipator fixing holes in the base plate and etch the bracket locations on the casing, also helped ensure accurate alignment of the base plate fixing holes with the U slots in the dissipator brackets in order to allow the external dissipators to be able to be attached to the pier and base plate.

![Figure 8: Components used to construct the pier and various stages of construction.](image)

**EXPERIMENTAL RESULTS**

The very first test conducted on the bridge (T1-PT) showed that in the absence of supplemental dissipative devices that frictional sources of energy dissipation (sliding interfaces e.g. deck-bearing interface and deck to shim interface at the abutments) for this specimen were great enough that in combination with the self-centering characteristic of the pier produces a flag shaped response (Figure 9a). The trio of tests T1-PT, T5-PT, and T6-PT show that increasing the initial pier post-tensioning level results in a very minor increase in the transverse stiffness and self-centering behaviour of the specimen (Figure 9a).
Two reasons were found for this. Firstly, the pier provides only a small contribution to the total transverse strength of the bridge (about 30%, the rest being provided by deck elongation which is addressed later) and secondly the strength contribution from tendon elongation reduces as a function of increasing initial post-tensioning force (Figure 9b). The strength contribution from tendon elongation reduces under increasing initial post-tensioning force because the neutral axis depth increases under increasing axial load, thus reducing the rate of tendon elongation (Figure 9b) and hence its contribution to strength.

![Figure 9](image-url)  
**Figure 9**: Effect of initial pier post-tensioning force on the force-displacement behaviour of the bridge specimen.

![Figure 10](image-url)  
**Figure 10**: Force-displacement plots showing overall bridge behaviour due to the inclusion of dissipative devices at the pier base and deck and the effect of deck post-tensioning.

![Figure 11](image-url)  
**Figure 11**: a) Jacking apart of the decks due to plastic set of the dissipators; b) Disappearance of the deck gap after removal of deck dissipators with deck post-tensioning still in place.

The addition of 4 dissipators in T4-DCR so that the pier was in DCR configuration increased both the stiffness and damping of the entire system (Figure 10a). In test T8-DPT longitudinal deck post-tensioning was installed and found to create a large increase in the transverse...
stiffness of the bridge and self-centering ability (Figure 10a). Then building upon work by (Chegini & Palermo, 2015) in Phase 1 on applying DCR to the bridge deck, in test T9-DDCR the pier was put into the DCR configuration, the deck post-tensioning from Test T8-DPT retained, and two grooved dissipators were installed across the deck-deck joint (Figure 11a). The addition of deck dissipators massively increased both the hysteretic energy dissipation capacity and the transverse stiffness of the bridge system (Figure 10b). The increase in energy dissipation was more significant than that from pier dissipators (Figure 10) alone. Despite these improvements, pinching of the force-displacement behaviour was observed (Figure 10b). This pinching was found to be due to plastic set from the deck dissipators jacking the decks apart, such that, at zero lateral displacement a small gap existed between the decks (Figure 11) and only after some amount of lateral displacement were the decks able to contact each other and induce tension into the deck dissipators.

In terms of the observed behaviour of the bridge system, it was found that the mainly rotational displacement path of the pier-cap beam system (Figure 12a) caused rocking to occur between the deck and cap beam. This resulted in a portion of the deck effectively uplifting from the cap beam (Figure 12b) during pushing and pulling of the specimen. Inclinometer measurements of the deck showed that it remained relatively horizontal, while spring pot measurements at the abutments showed no deck uplift to occur there. Spring pots at the cap beam measured uplift of the deck. Deck uplift was found to increase proportionally to the pier lateral drift. At 2.75% drift the maximum uplift which occurred was 43mm (Figure 12d). Deck to deck rocking was also observed (Figure 12c). This movement resulted in the bridge decks being deformed in the manner shown in Figure 12e. In Test T1-PT at 2.75% drift, the deck-deck corner gap opening was measured to be 80mm. In addition to this, measurements of abutment movement in the longitudinal axis of the bridge, show, that as the decks rotate due to transverse movement at the cap beam, they elongate and both twist, and push the abutments apart. Deck elongation was found to increase linearly with pier displacement and at 2.75% drift a total of 30mm deck elongation was measured. It was also found from post-testing analysis, that the majority of the force required to deform the bridge (approx. 70%) is attributed to the force required to push the abutments apart through the mechanism of deck elongation caused by deck-deck rocking.

![Figure 12: a) – c) observed behaviour of the pier, deck-cap beam joint and deck-deck joint; d) measured cap beam to deck uplift at the south deck edge; and e) Schematic of the mode of deformation of the bridge decks and their impact on the abutments.](image-url)
Pier curvature measurements revealed that the pier was subject to double instead of single bending (Figure 13a). The cause of this was determined to be the cap-beam to deck rocking action changing the point along the cap beam where the deck was supported. Thus, when the pier rotated about its base from rocking, the cap beam becomes eccentrically loaded inducing a constant moment, which, when combined with the triangular bending moment from lateral loading, results in the observed double bending (Figure 13b). The eccentric cap beam loading is always such, that it aids self-centering of the pier (Figure 13b), analogous to a positive P-Δ effect.

Figure 13: a) Typical measured pier curvature showing double bending; b) Explanation of the cause of the pier double bending through free body diagram of the pier – capbeam – deck subsystem

DISCUSSION OF EXPERIMENTAL RESULTS

The observations made regarding deck-cap beam rocking, deck to deck rocking and consequent deck elongation have significant consequences for the seismic design of simple span beam bridges, especially, with short spans and wide decks. These consequences are discussed below.

1. Allowance of deck uplift from the cap beam could present significant structural issues, such as, excessive transverse bending of the deck under self-weight due to non-uniform edge support; and non-structural issues, such as, the safety of users of the bridge during a ground motion, due to live loading of the unevenly supported deck potentially causing it to tip resulting in vehicle collisions.

2. Eccentric vertical loading of the cap beam under seismic action due to the rotational component of deformation of the pier may occur in simply supported bridges using hammerhead piers. This will cause the bending moment distribution in the pier to deviate from that of the idealised SDOF used in design. Hence, it may be important to recheck whether capacity design of the pier is still respected (protect the top of the pier from yielding due to the increased bending moment there), in addition to accounting for the positive P-Δ effect increasing the moment capacity of the pier base.

3. If vertical restraint is provided between the deck and cap beam (e.g. monolithic connection), eccentric loading of the cap beam may be prevented. However, torsion of the deck from the rotational component of deformation of the pier will occur. This will also result in the generation of a restoring moment at the top of the column which may be larger than that described in point 2.

4. Deck elongation will occur in bridges using decks discontinuous between spans and will be most significant for wide, short span decks. Any restraint of deck elongation (e.g. by the abutments or linkage bars) will considerably increase the bridges transverse stiffness. Therefore, if deck elongation is not accounted for in design, then the actual transverse displacement experienced by the structure during an earthquake is likely to be less than the design displacement (implying reduced pier plastic hinge damage),
while simultaneously, a transverse seismic load larger than that estimated in design will be taken by abutments. Therefore, omission of this effect (especially for DDBD) will result in a design which will not perform as intended.

Points 1-3 are all consequences arising from deck-cap beam interaction but can be easily eliminated if multi-column bents are used instead of hammerhead piers. However, point 4 is a more difficult issue to address as it is a problem arising purely from movement of the deck under seismic action.

CONCLUSION

Experimental results obtained from quasi-static cyclic testing of a 1/3 scale bridge using a single hammerhead DCR cantilever pier was presented in this paper. The experiment was unprecedented in terms of its focus on obtaining the system performance of a bridge using low damage technology subject to lateral loading. The effects of different variables on the bridges performance such as the amount of initial pier post-tensioning; location of dissipative devices; and the inclusion of unbonded longitudinal deck post-tensioning were investigated. The experiment yielded two previously unstudied but significant observations of deck-cap beam rocking and deck geometric elongation. Deck-cap beam rocking was found to change the bending moment distribution along the height of the pier resulting in a reduction in pier base moment and simultaneous increase in moment at the top of the pier. Whilst, deck elongation was found to cause a significant increase in the bridges transverse stiffness leading greater forces than expected being transmitted to the abutments. In addition to the experimental results, design and construction challenges related to the low damage detailing was also presented along with some solutions to address those issues.

ACKNOWLEDGEMENTS

Research funders: UC Quake Centre and Natural Hazards Research Platform; Collaborator: Zeinab Chegini; Sponsorship and specimen manufacture: Bradford Precast Ltd; Contributions to experimental work: UC technical staff Gavin Keats, Russell McConchie and Peter Coursey.

REFERENCES


