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QUASI-STATIC TESTING OF A 1/3 SCALE PRECAST CONCRETE BRIDGE UTILISING A POST-TENSIONED DISSIPATIVE CONTROLLED ROCKING PIER

R. Liu [1], A. Palermo [2]

[1] *PhD Candidate; University of Canterbury, Christchurch, New Zealand; royce.liu@pg.canterbury.ac.nz*

[2] *Associate Professor; University of Canterbury, Christchurch, New Zealand; alessandro.palermo@canterbury.ac.nz*

Abstract

Low damage design is an alternative design philosophy to conventional capacity design. Its aim is to significantly reduce seismically induced damage to structural members and improve post-earthquake functionality, through replacing member plastic hinging, with replaceable ductile connections. An example of a low damage design structure is one which utilises dissipative controlled rocking (DCR) or the hybrid PRESSS connection. In such a structure, dissipative devices are combined with rocking and unbonded post-tensioning to form a structural system with self-centering behaviour and damage confined to replaceable dissipative devices.

For the seismic design of bridges, the need for low damage design is becoming more apparent due to major indirect losses caused by damaged bridges in past major earthquakes. Much experimental research on low damage bridge design has focused on the testing of sub-assemblies, such as, bridge piers. Despite, these experiments being valuable in confirming the performance of these sub-assemblies, they are unable to indicate the real behaviour of these components within a complete bridge system or their effect on other components of the bridge (e.g. displacement compatibility effects).

At the University of Canterbury, a 1/3 scale, two span, precast concrete bridge supported by a cantilever, post-tensioned, rocking column was designed and tested as part of a government funded program called Accelerated Bridge Construction and Design (ABCD). In addition to investigating interaction effects between the pier and the rest of the bridge, the behaviour of the same bridge utilising a modified dissipative controlled rocking (MDCR) pier was also investigated. The pier type is said to be modified because it utilises a novel technique of increasing structural redundancy called “hierarchical activation”. The motivation for this being, that, cantilever piers utilise a column sway mechanism whereby all inelastic rotation is concentrated at the base. Therefore, it is vitally important that the base connection is protected from failure in order to prevent collapse. For DCR piers in particular, they rely on the restoring force of the post-tensioning and multiplicity in the number of dissipators to provide structural redundancy under lateral loading. However, once rupture of a few dissipators and yield of the post-tensioning occur, the pier would lose significant stiffness and would be prone to P-Δ effects eventuating in collapse. The technique of “hierarchical activation” involves having a second set of dissipative devices which are only activated after the structure exceeds a certain drift level e.g. ultimate limit state. Hence, for seismic demands larger than the design level, the pier is given an extra layer of structural robustness in the form of increased damping and stiffness which contribute to limiting excessive displacements protecting the post-tensioning from yielding.

This paper describes the results of the experimental work undertaken covering 8 tests, each, using different configurations (e.g. dissipation device locations, post-tensioning levels in pier and deck). Significant interaction effects between the pier and deck were observed, in addition to, unexpected deck elongation effects. These observations are shown to have important consequences for the design of cantilever/hammerhead pier bridges in general and bridges using discontinuous decks between spans.

Keywords: bridge; low damage; rocking; unbonded-post-tensioning; precast concrete

1. Introduction

Much current research on the seismic design of RC bridges, has focussed on improving pier performance in order to reduce physical damage and residual drift associated with plastic hinging. A particular, low damage, seismic design strategy which is of current research interest, applicable to bridge piers, is dissipative controlled rocking (DCR), also known as the PRESSS hybrid system. This design strategy was originally developed during the US Precast Seismic Structural Systems (PRESSS) program [1] undertaken in the 1990's for concrete structures, and since then has also been applied to steel [2] and engineered timber [3]. In essence, DCR involves the following: create a discontinuity between members at a connection so that rocking can occur; clamp the two members together using unbonded post-tensioning for improved and controllable self-centering behaviour (Fig. 1a) (the combination of rocking with unbonded post-tensioning is termed controlled rocking CR); and add dissipative devices (usually metal hysteretic) across the rocking interface (internal or external to the member (Fig 1c)) to increase moment capacity of the connection and or damping (Fig. 1b). The combination of these three elements results in a flag shaped hysteresis curve (Fig. 1b).

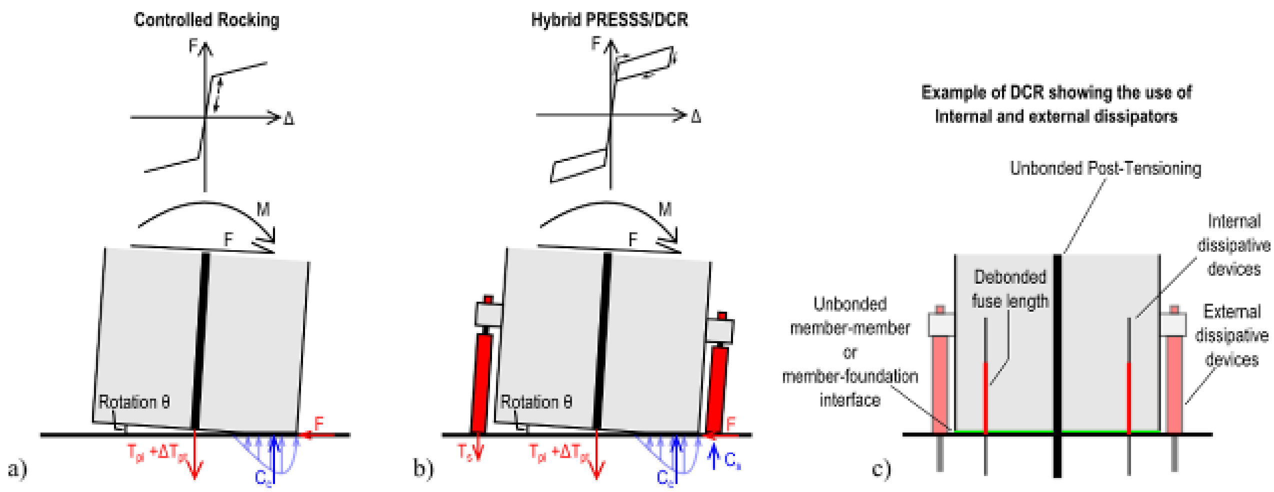

Fig. 1: **a)** Combination of unbonded post-tensioning with rocking (controlled rocking) along with the force-displacement behaviour. **b)** Deformed hybrid PRESSS/DCR connection with force-displacement behaviour. **c)** Illustration of the different ways dissipative devices can be placed in a DCR connection.

Inelastic rotation in a DCR connection is accommodated by rocking rather than by member plastic hinging. However, the dynamic response of a DCR system is more similar to that of a conventional RC structure than a free rocking structure [4]. Hence, DCR is treated in a similar manner to conventional RC structures in terms of seismic design [5–7]. Outside the DCR connection, structural members remain elastic and the structure self-centres after a ground motion (due to self-weight and post-tensioning) virtually eliminating residual drift [8, 9]. Despite the performance benefits of this strategy, there are two points of major concern: the first is structural robustness, and the second is the displacement interaction of a rocking pier within a complete bridge system. This paper addresses both concerns.

1.1. DCR: structural redundancy

In the context of cantilever bridge piers, the plastic hinging mechanism is of column sway, where, collapse prevention is dependent upon the ductility capacity of the plastic hinge at the base. In DCR cantilever bridge piers the rotational ductility against overturning is dependent upon the dissipators and post-tensioning. Currently, all the dissipators are designed to activate at a set level of earthquake loading and all have the same ultimate capacities. Hence, currently the robustness of DCR is purely provided by multiplicity in the number of dissipators and post-tensioning. Once rupture of a few dissipators and yielding of the post-tensioning occurs, the pier would lose significant stiffness and would be prone to P-Δ effects eventuating in collapse (Fig. 2a).

a) b)

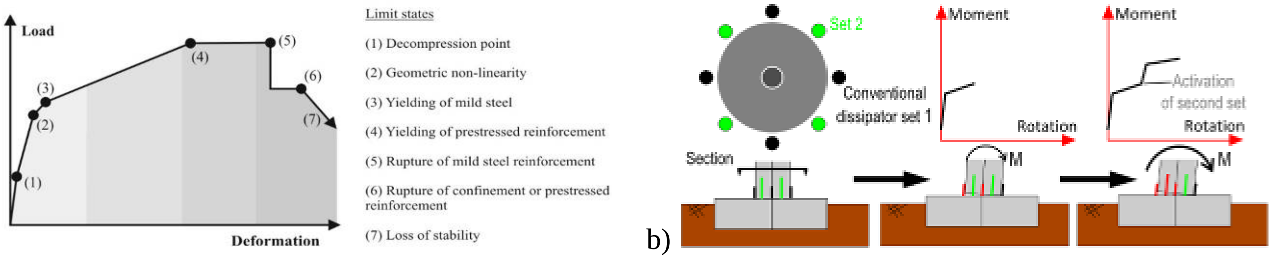

Fig. 2: **a)** Performance objectives and limits for a DCR pier [10]. **b)** Example of hierarchical activation used in experimental testing presented in this paper.

To mitigate this issue, research is being conducted by the authors on a new design strategy based on dissipative controlled rocking called “Multi-Performance Dissipative Controlled Rocking” (MDCR). The concept is to delay the onset of collapse through discretizing the capacity of the structure provided by dissipative devices and or mechanisms (rocking), such that, the devices and or mechanisms are activated in a hierarchical manner under increasing levels of shaking. The authors have proposed three methods of achieving this [4], however, only one is explained in detail here due to its relation to the experimental results being presented.

The method of achieving hierarchical activation focussed on in this paper, is to have two sets of dissipators across one rocking interface (Fig. 2b). One set, is directly attached to the members either side of the rocking interface, while, the other is attached in such a way that it is only engaged when uplift at the rocking interface exceeds a specified value. The purpose of this arrangement, is that under frequent seismic loading (return period less than ULS) only one set of dissipative devices is relied upon and if the intensity of the ground motion exceeds that of the ULS ground motion then the second set of dissipative devices is activated in addition to the first set. In this way, after a ULS ground motion, even though one set of dissipators may be spent, the vulnerability of a DCR pier to a sequential significant ground motion (if the first set is not immediately replaced) is lessened due to the presence of the second unused set on standby. In addition to this, in extreme events where both sets contribute to resisting the ground motion, the onset of yielding of the post-tensioning and major P-Δ effects are delayed due to the activation of the second set reducing overall displacements.

1.2. DCR: effect of displacement compatibility within a complete bridge system

Since its proposed application to bridges by Palermo et al [11], experimental research into the DCR design strategy applied to bridges has been intense. At the University of Canterbury alone, the following experimental research has been conducted: validation of the uniaxial performance of the DCR column concept with internal dissipators [9] and external dissipators [12], both tests were compared to monolithic benchmarks; investigation into the biaxial performance of DCR columns and comparison with monolithic benchmarks by both Solberg et al [13] and Marriott et al [14]; application of DCR to Accelerated Bridge Construction (ABC) as a means of creating a controlled damage pier and validation of performance through biaxial testing and comparison with an emulative monolithic benchmark [15]; and validation of the DCR concept as a suitable low damage connection to be used in ABC in high seismic regions through, uniaxial testing of a pier bent and comparison with an emulative monolithic benchmark [16]. Outside the University of Canterbury, research has been more focussed on applying low damage technologies to accelerated bridge construction (especially in the US) and the application of DCR to tall segmental concrete bridge piers as a means of increasing dissipative capacity [17, 18].

The previously described experimental work related to DCR are all examples of sub assembly tests (cantilever column and pier bent). In terms of bridge system testing, only 5 examples of large scale experimental work could be found [19–23]. Of the work on bridge systems that has been conducted, only that of Thonstad et al [20] used DCR bridge piers. However, the specimen tested by Thonstad et al [20] was not a complete bridge as it did not have abutments, and the main objective of their testing was to investigate the effect of pier stiffness irregularities (Fig. 3a). Only the tests conducted by Nelson et al [22] (Fig. 3b) and Levi et al [23] (Fig. 3c) were on complete bridge specimens and even then, only Levi et al [23] identified substructure-superstructure interaction effects due to system deformation compatibility. Hence, the experimental work presented here is an important first step in identifying any unknown interaction between a cantilever DCR bridge pier and the rest of a bridge system which could have an overall adverse effect on both structural and non-structural aspects of bridges using DCR.

Fig. 3: **a)** Specimen tested by Thonstad et al [20] showing lack of abutments. **b)** Conventional RC bridge using pier bents tested by Nelson et al [22]. **c)** Curved conventional RC bridge tested by Levi et al [23].

1.3. Overview of research program

The research presented here, are the results from the second phase of a two phase project under the parent program, Advanced Bridge Construction and Design (ABCD), funded by the New Zealand Natural Hazards Research Platform. The entire project consisted of experimental testing of a scale, low damage, precast concrete bridge. Phase 1 of this project focused on the superstructure, details can be found in [24]. Phase 2 of this project (conducted by the first author) focused on the substructure (pier) and its interaction with the superstructure.

2. Experimental investigation

2.1. Specimen overview

The specimen is based on a prototype structure (Fig. 4) typical of a two lane, short-span, reinforced concrete, New Zealand Highway Bridge. The prototype consists of two, 13m simple spans; a single, 1.5m diameter circular pier; and hammer head type cap beam, supporting, a precast superstructure made of standard design double hollow core units [25]. The height from the base of the pier to the centre of mass of the superstructure was assumed to be 7.65m (Fig. 4).

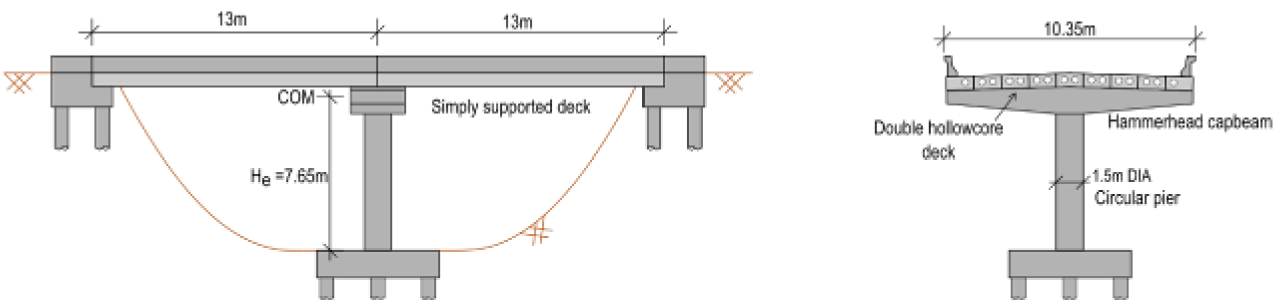

Fig. 4: Prototype bridge using single hammerhead pier.

The test specimen, is a fully precast concrete structure (design f'c = 40MPa) and is 1:3 scale of the prototype. Like the prototype, it is a two span simply supported bridge. The decks are hollow core slabs each nominally 4222mm long by 2400mm wide and rest on rectangular ultra-high molecular weight polyethylene bearings. The deck slabs are hollow core both to replicate the prototype and to allow the installation of un-bonded post-tensioning tendons (Fig. 5). In terms of lateral restraint, at the abutments there are elastomeric (IRHD60) and steel plate packing either side of the decks (Fig. 5); and at the cap beam, 50 x100 MSG8 timber is placed either side of the decks to fill the gap between the decks and cap beam shear keys (Fig. 5). Free longitudinal movement of the decks is prevented by wood packing placed between the decks and abutment back walls (Fig. 5). The pier is 500mm in diameter, and has a clear length of 2140mm. The armouring at the base of the pier is a custom made, 500 x 10 CHS with an annular steel ring welded to the base (Fig. 6c).Shear studs welded to the inside of the CHS ensure composite action with the pier. Shear and torsion restraint of the pier base is provided by external shear and torsion keys (Fig. 6b). Pier post-tensioning is provided by a single 26.5mm diameter Macalloy bar of 3182mm unbonded length. Axial metal hysteretic dissipative devices of the grooved type [15, 26] were used. The dissipators had 4 grooves, are 16mm in diameter, have a design fuse length of 185mm, a design fuse area of 135mm^2 and are made from mild steel (Fig. 6e) The pier base accommodates a maximum of 8 dissipators (Fig. 6a) which are evenly distributed around the pier (Fig. 2b): 4 for conventional PRESSS hybrid configuration and 4 for hierarchical activation ($\theta_{engagement}$ =2.5% so that yielding would occur prior to θ_{MCE} =2.75%). The dissipator to dissipator circle diameter is 570mm.

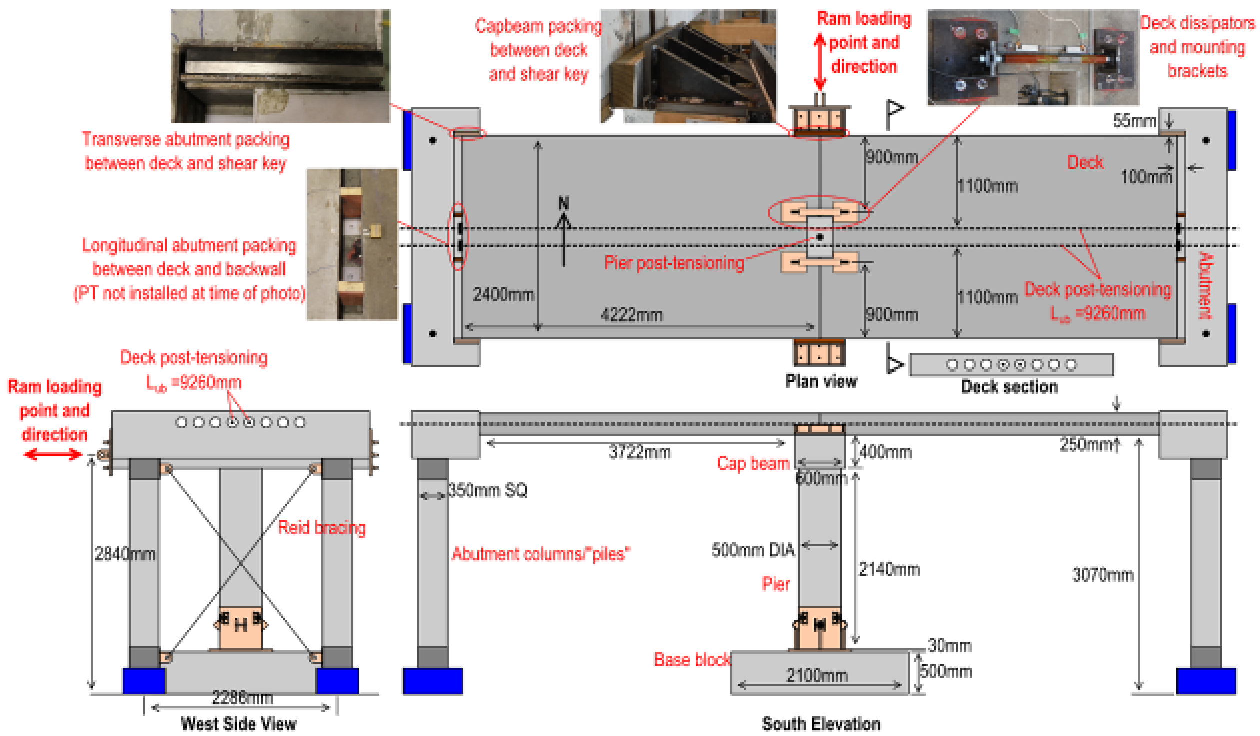

Fig. 5: Overall specimen dimensions and detail regarding the boundary conditions around the decks.

2.2. Pier details: hierarchical activation of two sets of dissipators

In the experimental testing shown here, hierarchical activation, involved two sets of dissipators, where, one set is engaged at a later pier displacement to the other. Many options for achieving this were investigated, however, most were either infeasible or impractical. The final option chosen, consists of dissipators being connected by pins to the foundation and pier (Fig. 6a), but where, the pinned bracket connecting the dissipator to the pier has a slotted hole (Fig. 6d).

At zero lateral pier displacement, the pin in the slotted hole is designed to sit at the top of the slot. Then as the pier uplifts, the dissipator bracket with slotted hole moves upwards and relative to this the pin approaches the bottom of the slot. Only when the uplift is sufficient for the pin to engage with the bottom of the slot, is the dissipator pulled upon. This option for producing hierarchical activation was chosen for three reasons: the pins allow the dissipator to be pulled on axially, despite, the arced uplift movement of the pier; the slotted hole allows the dissipator to undergo compression once it has been extended, whilst, also providing vertical and horizontal restraint of the dissipator ends; and this option was reasonably practical to implement, along with the spent dissipators being accessible for replacement.

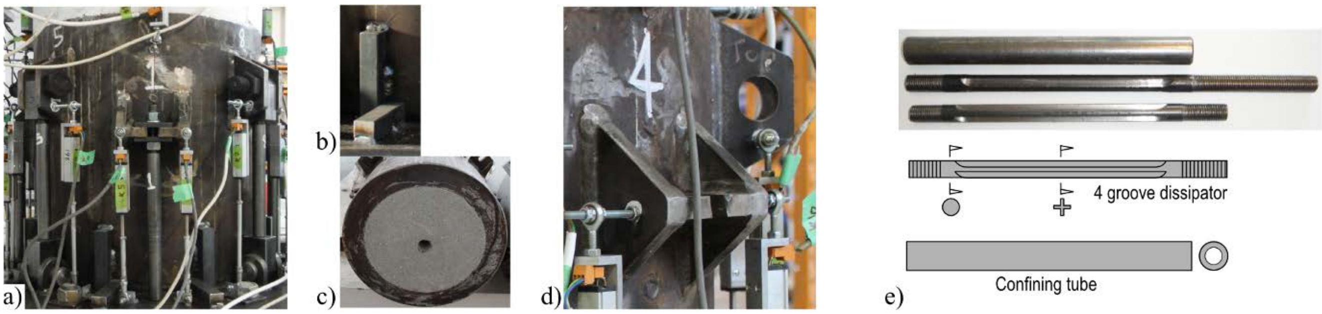

Fig. 6: **a)** Pier with all dissipators attached; **b)** Shear and torsion keys; **c)** Pier base rocking interface; **d)** Dissipator bracket detail, pinned dissipator bracket in background; **e)** Groove type dissipator and confining tube.

2.3. Specimen design

In terms of seismic action design parameters, the prototype was assumed to be of importance level 2; sited in Christchurch, with a hazard factor of 0.3; constructed on non-liquefiable soil of class C; and not subject to near fault effects. Seismic design was only conducted in the transverse direction due to this direction of loading being the governing load case for the pier. Direct Displacement Based Design (DDBD) was used to determine the seismic design parameters for the prototype pier which were then scaled for specimen design. Table 1 below summarizes the salient pier seismic design parameters calculated.

Table 1: Summary of specimen seismic design parameters from displacement based design.

Specimen ULS design actions and dimensions	**Units**	**Value**
Design gravity load, W_{scaled}	kN	195
Design lateral load, V_{scaled}	kN	42.9
Effective height of equivalent SDOF, $H_{e,\ scaled}$	mm	2550
Pier diameter, D_{scaled}	mm	500
Design base moment, M_{scaled}	kNm	109.4
Design displacement, Δds	mm	52
MCE displacement, ΔMCEs (ADRS assuming EPP F-Δ)	mm	81.4

2.4. Test set up and loading regime

The test set up consisted of a single 300kN ram, loading the bridge transversely at the cap beam level 2310mm above the base of the pier (Fig. 7a). The position and direction of loading were chosen to simulate transverse seismic loading of the bridge and to allow later pier only tests to be conducted. Loading of the bridge was cyclic, displacement controlled, and quasi-static. The loading protocol used for testing (Fig. 7b) was derived from ACI T1.1-01 [27]: three fully reversed cycles are applied at each drift ratio; the first drift ratio is within the linear elastic response range; subsequent drift ratios are between 1.25 and 1.5 times the previous value. Lateral drifts of 0.1%, 0.125%, 0.175%, 0.25%, 0.35%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0% 2.75% 3.5%, and 4.5% were imposed. However, only during testing of the pier in isolation were all of the drift ratios imposed. During bridge testing, the maximum applied drift ratio was 2.75% due to safety reasons and the load capacity of the ram to cap beam loading attachment.

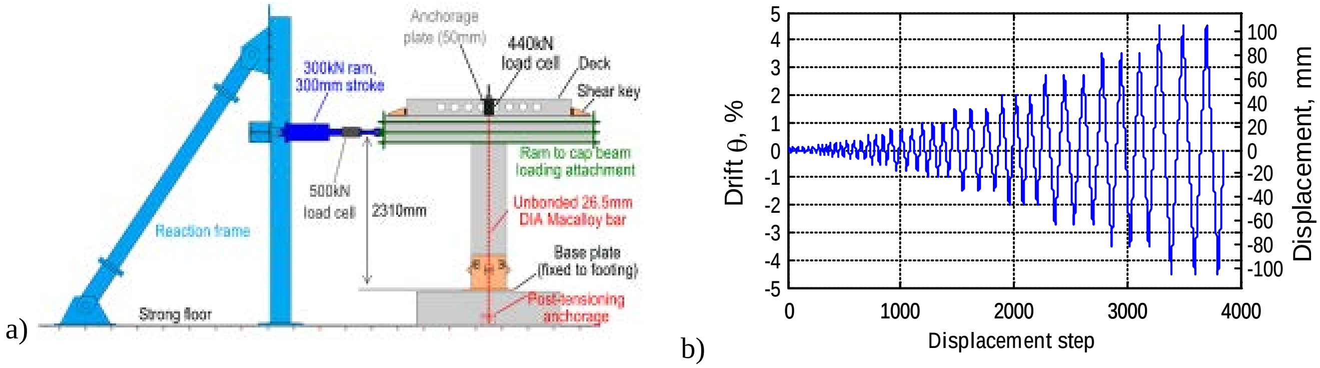

Fig. 7: **a)** Drawing of experimental test set up. **b)** Plot of loading protocol

2.5. Testing schedule

A total of 14 tests were conducted (10 bridge and 4 isolated pier tests). In each test, the variables which were changed are: initial pier post-tensioning force; number of dissipators attached to the pier; inclusion of deck post-tensioning; inclusion of deck dissipators, and one bridge test excluding the transverse packing at the abutments. For brevity and relation to the findings described in this paper, only eight of the ten bridge system test configurations are summarized in Table 2 below.

Table 2: Schedule of bridge tests conducted.

Test number and configuration description (All tests described are bridge system tests)	**Pier post-tensioning kN**	**Conventional pier dissipators**	**Pinned dissipators**	**Deck post-tensioning kN**	**Deck dissipators**
(T1-PT): Controlled rocking pier $T_{pi} = 0.2F_{pty}$	89.2	-	-	-	-
(T3-MDCR): MDCR pier	91.37	4	4	-	-
(T4-DCR): PRESSS/DCR pier	92.3	4	-	-	-
(T5-PT): Controlled rocking pier $T_{pi} = 0.4F_{pty}$	149.1	-	-	-	-
(T6-PT): Controlled rocking pier $T_{pi} = 0.6F_{pty}$	195.7	-	-	-	-
(T7-DPT): Controlled rocking pier and post-tensioned deck (2 x 15.2 dia tendons)	100	-	-	40	-
(T8-DPT): Controlled rocking pier and post-tensioned deck (2 x 12.7 dia tendons)	95	-	-	40	-
(T9-DDCR): DCR pier, post-tensioned deck (2 x 12.7 dia tendons) and deck dissipators	95	4	-	40	2

3. Experimental results and analysis

Tests T1, T5 and T6 of the bridge system (Fig. 8a) involved having a simply supported deck with post-tensioned only pier (controlled rocking) under different levels of initial post-tensioning force (0.2, 0.4 & $0.6F_{pty}$ respectively). In these tests, the only source of energy dissipation was from friction, however, the amount present was of enough significance that in combination with the controlled rocking pier produced a flag shaped response (Fig. 8a). It was found that increasing the initial pier post-tensioning force had very little effect on increasing the transverse stiffness of the bridge system (Fig. 8a). 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 (Fig. 9a). 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 (Fig. 9a) and hence its contribution to strength.

Tests T3 and T4 of the bridge system involved the addition of dissipators to the base of the pier. In test T3 the pier utilised hierarchical activation and was in the MDCR configuration with 8 dissipators, while, in test T4 only 4 dissipators were attached to the pier so that it would be in a conventional PRESSS/DCR configuration. From Fig. 8b it is clear that the addition of dissipators increases both the stiffness and damping of the entire bridge system. In addition to this, Fig. 8b also shows the successful activation of the second set of dissipators occurring in Test T3. However, the contribution from the second set of dissipators is small because the activation drift is just over 2% while the maximum applied drift was 2.75%.

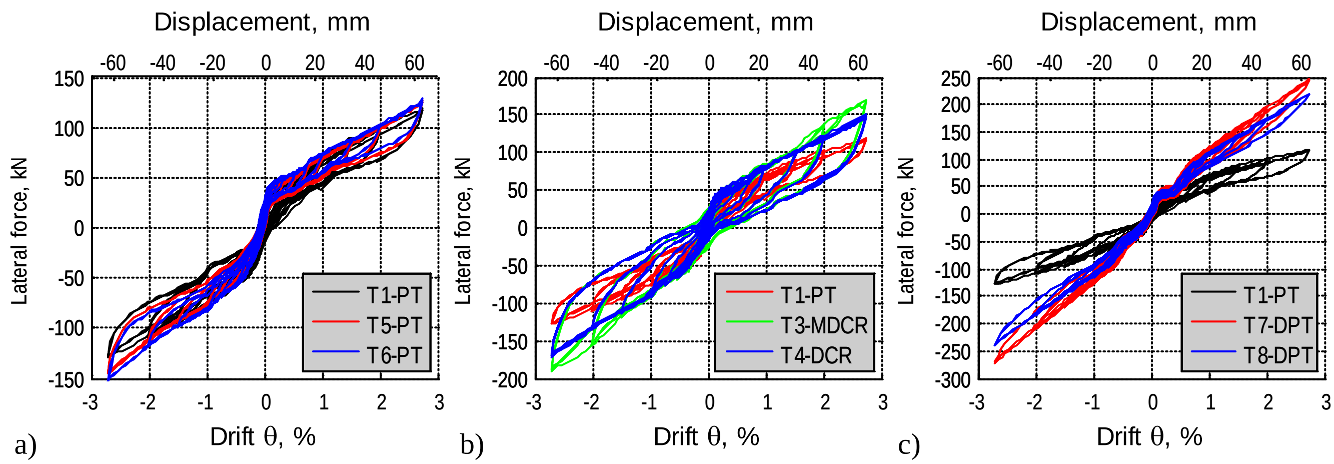

Fig. 8: Comparison of the force-displacement behaviour between various tests

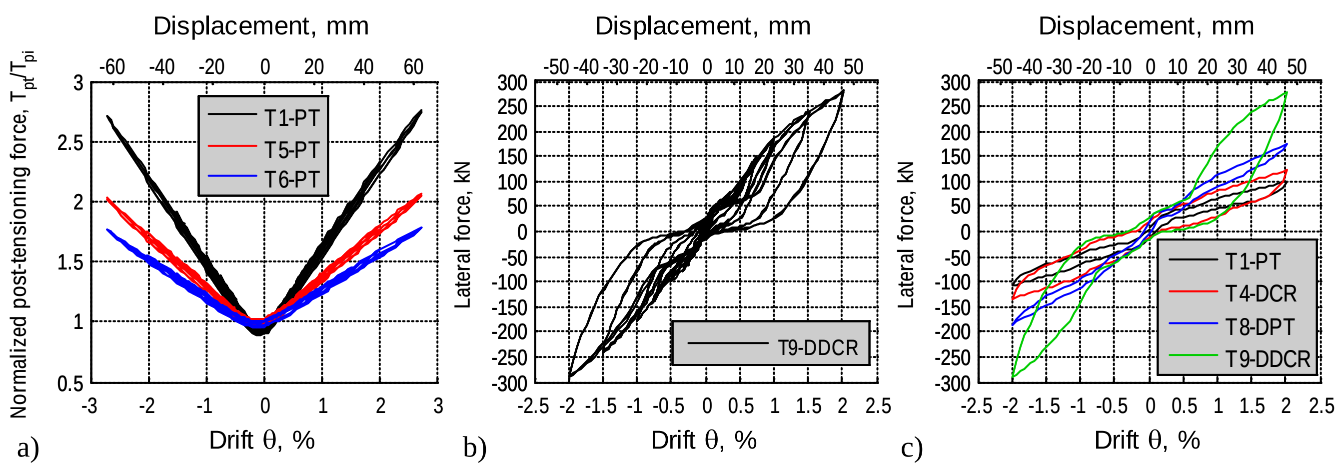

Fig. 9: a) Effect of increasing the piers initial post-tensioning force on tendon elongation due to uplift. b) Force-displacement behavior of Test 9. c) One hysteresis cycle at 2% drift for various Tests.

Tests T7 and T8 involved having the pier in the controlled rocking configuration while having the deck post-tensioned. In Test T7, two 15.2mm diameter tendons were used, while in Test T8, two 12.7mm diameter tendons were used. The addition of deck post-tensioning considerably increased: the bridge systems transverse stiffness (Fig. 8c), linearity of the force-displacement behaviour, and the bridges ability to self-centre.

Building upon work by Chegini et al [24] on applying DCR to the bridge deck, in Test T9, the pier was put into the DCR configuration, the deck post-tensioning from Test T8 was retained, and two grooved dissipators were installed across the deck-deck joint (Fig's. 10a &c). The addition of deck dissipators massively increased both the hysteretic energy dissipation capacity and the transverse stiffness of the bridge system (Fig. 9b &c). The increase in energy dissipation was more significant than that from pier dissipators (Fig. 9c) alone. Despite these improvements, pinching of the force-displacement behaviour was observed (Fig. 9b). 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 (Fig. 10a) and only after some amount of lateral displacement were the decks able to contact each other and induce tension into the deck dissipators.

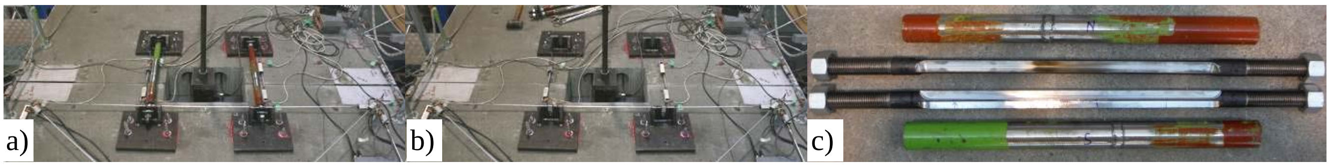

Fig. 10 a) Jacking apart of decks due to plastic set of the deck dissipators. b) Closure of deck-deck gap after removal of deck dissipators. c) Deck dissipators prior to installation

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 (Fig. 11a) caused rocking to occur between the deck and cap beam. This resulted in a portion of the deck effectively uplifting from the cap beam (Fig. 11b) 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 (Fig. 11c).

Fig. 11: a) Pier deformed shape at 2% drift (push phase); b) Close up of gap opening between deck and cap beam at 2% drift (push phase); c) Measured deck-cap beam uplift at the southern edge of the deck.

Fig. 12: a) Deck-deck gap opening at 2% drift test 1 during the push cycle (left) and pull cycle (right). b) Plan view of deformed bridge shape (black lines).

Deck to deck rocking was also observed (Fig. 12a & b). This movement resulted in the bridge decks being deformed in the manner shown in Fig. 12c. In Test T1 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.

Pier curvature measurements revealed that the pier was subject to double instead of single bending (Fig. 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 (Fig. 13b). The eccentric cap beam loading is always such, that it aids self-centering of the pier (Fig. 13b), which is analogous to a positive P-Δ effect.

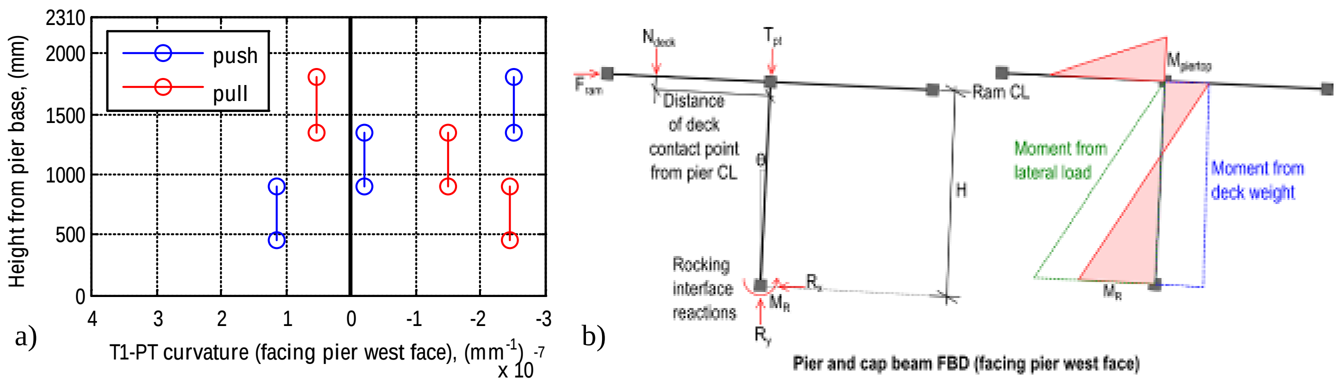

Fig. 13: **a)** Plot of pier curvature at 0.25% drift showing that the pier was subject to double bending. Pier curvature is presented such that it follows the bending moment looking at the west face of the pier. **b)** Free body and BMD diagram of the pier-cap beam system showing how double bending of the pier occurs.

4. Discussion

From the experimental results and observations presented, there are several key consequences regarding seismic design that not only concern bridges which use DCR, but bridges in general using hammerhead piers, and bridges with discontinuous decks between spans. These consequences are discussed below.

- Allowance of deck uplift from the cap beam could present significant structural issues, such as, unintended 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 uneven live loading of the unevenly supported deck potentially causing it to tip resulting in vehicle collisions.
- 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.
- 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 produced in the previously described case of deck to cap beam rocking.
- Deck elongation will occur in bridges using decks discontinuous between spans and will be most significant for wide, short spanning 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.

5. Conclusion

Two issues related to dissipative controlled rocking were described: structural robustness and displacement interaction of a rocking pier within a complete bridge system. A solution to increasing structural robustness called "Multi-Performance Dissipative Controlled Rocking" was proposed, the concept explained, and some experimental results presented. For the second issue related to DCR, a short literature review was given and showed the current lack of experimental work undertaken to assess the performance of a DCR pier within an entire system. Then the design, testing, and results for a 1/3 scale, two span, precast concrete bridge supported by hammerhead, post-tensioned, rocking column was presented. Observations from experiment showed that there is significant interaction between the pier and deck, in addition to, unexpected deck elongation effects. The consequences of these observations were discussed and were shown to be a general problem for hammerhead pier bridges and bridges using discontinuous decks and that these interactions need to be appropriately handled in the design of these structures.

6. Acknowledgements

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7. References

[1] Priestley, M. J. N. (1996): The PRESSS Program Current Status and Proposed Plans for Phase III. *PCI Journal*, *41*(2), 22–40.

[2] Christopoulos, C., Filiatrault, A., Uang, C. M., & Folz, B. (2002): Posttensioned Energy Dissipating Connections for Moment-Resisting Steel Frames. *Journal of Structural Engineering*, *128*(9), 1111–1120.

[3] Palermo, A., Pampanin, S., Buchanan, A., & Newcombe, M. (2005): Seismic design of multi-storey buildings using laminated veneer lumber (LVL). *NZSEE Conference*.

[4] Liu, R., & Palermo, A. (2015): Low Damage Design and Seismic Isolation : What's the difference ? *2015 NZSEE Conference*. Rotorua, New Zealand.

[5] Priestley, M. J. N., Calvi, G. M., & Kowalsky, M. J. (2007): *Displacement Based Seismic Design of Structures*. IUSS Press, Pavia, Itlay.

[6] Pampanin, S., Marriott, D., Palermo, A., & New Zealand Concrete Society. (2010): *PRESSS design handbook*. NZCS, Auckland, N.Z.

[7] Standards New Zealand. NZS3101 Concrete structures standard: Part 1 - the design of concrete structures (2006). New Zealand.

[8] Palermo, A., Pampanin, S., & Calvi, G. M. (2004): Use of "Controlled Rocking" in The Seismic Design of Bridges. *13th World Conference on Earthquake Engineering*.

[9] Marriott, D., Pampanin, S., & Palermo, A. (2006): Experimental Validation of High-Performance Hybrid Bridge Piers. *2006 NZSEE Conference*. Napier, New Zealand.

[10] Marriott, D. (2009): *The Development of High-Performance Post-Tensioned Rocking Systems for the Seismic Design of Structures*. University of Canterbury.

[11] Palermo, A., Pampanin, S., & Calvi, G. M. (2005): Concept and development of hybrid solutions for seismic resistant bridge systems. *Journal of Earthquake Engineering*, *9*(6), 899–921.

[12] Marriott, D., Pampanin, S., & Palermo, A. (2009): Quasi-static and pseudo-dynamic testing of unbonded post-tensioned rocking bridge piers with external replaceable dissipaters. *Earthquake Engineering and Structural Dynamics*, *38*(3), 331–354.

[13] Solberg, K., Mashiko, N., Mander, J. B., & Dhakal, R. P. (2009): Performance of a Damage-Protected Highway Bridge Pier Subjected to Bidirectional Earthquake Attack. *Journal of Structural Engineering*, *135*(5), 469–478. doi:10.1061/(ASCE)0733-9445(2009)135:5(469)

[14] Marriott, D., Pampanin, S., & Palermo, A. (2011): Biaxial testing of unbonded post-tensioned rocking bridge piers with external replacable dissipaters. *Earthquake Engineering and Structural Dynamics*, *40*(15), 1723–1741. doi:10.1002/eqe.1112

[15] White, S. (2014): *Controlled damage rocking systems for accelerated bridge construction*. University of Canterbury.

[16] Mashal, M., & Palermo, A. (2014): Quasi-static experimental testing of emulative and low damage seismic technologies for accelerated bridge construction (ABC) in seismic areas. *2014 National accelerated bridge construction conference* (pp. 344–353). Miami, US.

[17] Wang, J. C., Ou, Y. C., Chang, K. C., & Lee, G. C. (2008): Large-scale seismic tests of tall concrete bridge columns with precast segmental construction. *Earthquake Engineering & Structural Dynamics*, *37*(12), 1449–1465. doi:10.1002/eqe.824

[18] Hewes, J. T., & Priestley, M. J. N. (2002): *Seismic design and performance of precast concrete segmental bridge columns* (SSRP-2001/25) (Vol. 7).

[19] Sideris, P., Aref, A., & Filiatrault, A. (2015): Experimental seismic performance of a hybrid sliding – rocking bridge for various specimen configurations and seismic loading conditions. *Journal of Bridge Engineering*, *20*(11). doi:10.1061/(ASCE)BE.1943-5592.0000742.

[20] Thonstad, T., Mantawy, I. M., Stanton, J. F., Eberhard, M. O., & Sanders, D. H. (2016): Shaking table performance of a new bridge system with pretensioned rocking columns. *Journal of Bridge Engineering*. doi:10.1061/(ASCE)BE.1943-5592.0000867.

[21] Johnson, N., Ranf, R. T., Saiidi, M. S., Sanders, D. H., & Eberhard, M. O. (2008): Seismic testing of a two-span reinforced concrete bridge. *Journal of Bridge Engineering*, *13*(2), 173–182. doi:10.1061/(ASCE)1084-0702(2008)13:2(173)

[22] Nelson, R. B., Saiidi, M. S., & Zadeh, S. (2007): *Report No . CCEER-07-04: Experimental evaluation of performance of conventional bridge systems*.

[23] Levi, M. J., Sanders, D. H., & Buckle, I. G. (2012): Seismic response of columns in horizontally curved bridges. *15th World Conference on Earthquake Engineering*. Lisbon, Portugal.

[24] Chegini, Z., & Palermo, A. (2015): Large-scale testing of low-damage superstructure connections in precast bridges. *Proceedings of the Tenth Pacific Conference on Earthquake Engineering*. Sydney.

[25] Beca Carter Hollings & Ferner Ltd, & Opus International Consultants Ltd. (2008): *Research Report 364 Standard precast concrete bridge beams*.

[26] White, S., & Palermo, A. (2016): Quasi-Static Testing of Posttensioned Nonemulative Column-Footing Connections for Bridge Piers. *Journal of Bridge Engineering*, 1–13. doi:10.1061/(ASCE)BE.1943-5592.0000872.

[27] ACI Innovation Task Group 1. (2001): *Acceptance criteria for moment frames based on structural testing*. American Concrete Institute, Farmington Hills, Michigan, USA.