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Large-scale testing of low-damage superstructure connections in precast bridges

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ABSTRACT: Prefabrication of bridges offers many advantages compared to traditional in-situ methods, including rapid construction, higher quality concrete, improved work zone safety and minimal traffic disruption. However application of this technology in seismic areas is currently limited due to a requirement for reliable connections between prefabricated elements that can provide sufficient ductility. The research presented herein aims to address this issue by introducing a low-damage connection for precast bridge superstructures.

A simply supported bridge traditionally uses a combination of shear keys between the superstructure and steel linkages to provide transverse support to the superstructure and to prevent unseating of the spans during earthquake loading. The proposed connection adds longitudinal post-tensioning in combination with axial dissipaters to the superstructure to achieve self-centring of the structure and to dissipate energy. As a result adjacent superstructure segments aid each other in countering the transverse seismic load, rather than merely relying on the shear keys to do so.

This paper presents the results of three quasi-static tests performed on a 1:3 scaled concrete bridge specimen with low-damage hybrid connections in the superstructure subjected to loading in the transverse direction. The two hollow-core decks of this short-span, two-bay bridge were connected using post-tensioning tendons, and mild steel dissipaters linked the two superstructure spans to increase energy dissipation. The testing was carried out using two different values of post-tensioning in the tendons with and without the axial dissipaters and the results were compared.

INTRODUCTION

Many bridges in New Zealand are reaching the end of their service life, others have been damaged due to earthquakes and need to be repaired or replaced (Palermo & Mashal 2012). A major issue associated with replacing damaged or vulnerable bridge structures is how this task interferes with the traffic flow of an area resulting in direct and indirect economic losses. Currently most bridges in New Zealand are constructed using cast-in-place technology whereas alternative prefabrication methods are generally less time-consuming, offer higher construction quality and improve work-zone safety. Such methods have seen widespread use mainly in regions of low seismic activity (Khaleghi et al. 2012). However their implementation in seismic areas is dependent on the development of reliable connections between prefabricated segments that can provide sufficient ductility (Marsh & Stanton, 2011).

Developing a ductile connection between substructure segments in precast bridges has been the topic of several studies. However little research has focused on this issue for bridge superstructures, mainly because seismic loading is not critical in their design (Holombo et al. 2000; Veletzos et al. 2006; Megally et al. 2009; Sideris et al. 2014). Traditionally a concrete bridge superstructure is either designed to be monolithic with the cap beam and abutments through a moment resisting connection or is simply supported through bearings. Numerous failures of bridges using the latter type of connection have been observed under seismic loading, including unseating of the superstructure (Buckle et al. 2012) and damage to the cap beam or abutment shear keys (Yen et al. 2011, Megally et al 2002). This behaviour can be controlled to some extent by increasing the seating length and connecting adjacent sections of the superstructure together using steel linkages (Wood & Chapman 2013).

In this paper a connection is proposed that uses unbonded longitudinal post-tensioning and mild steel dissipaters as a way of improving the behaviour of simply supported superstructures under seismic loading. The post-tensioning provides a self-centring capability and reduces the residual deformation in the connection, which in combination with the mild steel dissipaters results in a flag-shape behaviour for the connection in which it is used (Figure 1). This connection was originally developed for use in precast concrete frames (Priestley et al. 1999) and later used in shear walls and bridge piers (Palermo et al 2007; Pollino et al. 2007; Mander & Cheng 2007; Marriott et al. 2009; Mashal et al. 2013). Unlike traditional monolithic joints, which are designed to provide ductility and dissipation through plastic hinging, the hybrid connection allows for the inelastic demand to be accommodated outside the precast elements (in the mild steel or dissipation devices) while other parts of the seismic-resisting structure remain elastic and relatively intact. This improves overall performance of the bridge and prevents unseating of the superstructure.

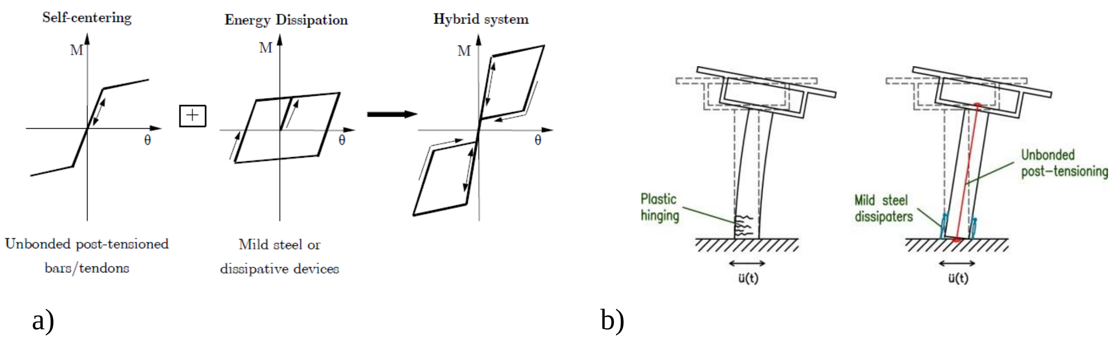


Figure 1 – a) Flag shape behaviour of a hybrid connection (fib, 2003); b) Comparison of deformation in a bridge pier with monolithic and hybrid connections

When this connection is used between superstructure spans, it can provide a re-centring moment as well as increase dissipation during lateral transverse seismic motion (Figure 2). This results in an improvement in the overall performance of the bridge and prevents unseating of the superstructure.

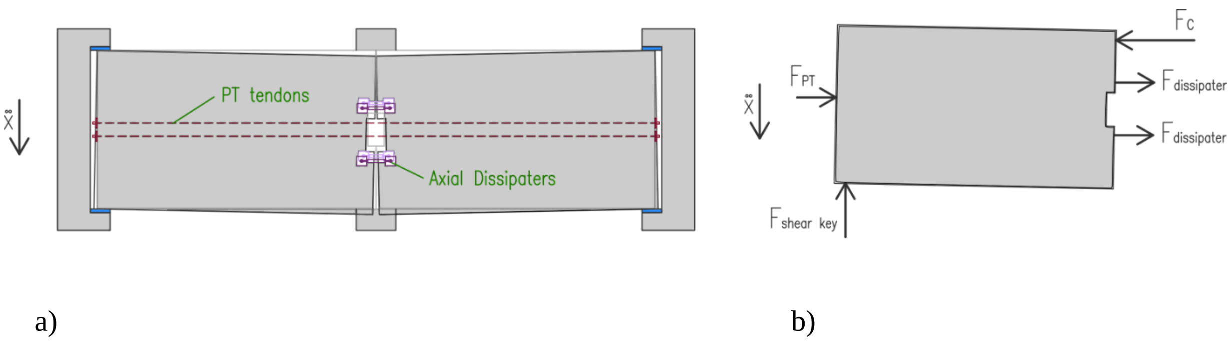


Figure 2 – a) Deformed shape of superstructure subject to lateral transverse loading b) Re-centring forces acting on superstructure

In this paper the results from a series of quasi-static tests on a 1:3 scaled concrete bridge specimen using described hybrid connections in the superstructure are presented.

1 EXPERIMENTAL TESTING

1.1 Specimen

The prototype structure, which the test specimen was based on, is representative of a typical short-span concrete highway bridge in New Zealand (Figure 3). A span length of 13m and a width of 10.35m (one lane in each direction) was considered for this bridge. The superstructure was selected to represent the double hollow-core section as described in NZTA Research Report 364 (NZTA 2008), see Figure 3-b. This section is composed of multiple smaller precast concrete hollow-core segments which are assembled together using transverse post-tensioning. Each span of the bridge is supported by a pier cap beam connected to a circular pier with a diameter of 1.5m. The center of mass of the super structure is located 7m above the base of the pier. The footings shown are for indicative purposes only. The prototype bridge is assumed to have an importance level equal to normal, and is located in Christchurch with a zone factor of 0.3, soil type C and no near field effects. A ductility of 3 was adopted for design with a drift of %2.7.

The test specimen was a 1:3 scaled model of the prototype bridge. The precast concrete components of the specimen were manufactured off-site before being transported and assembled at the Structures Extension Laboratory at the University of Canterbury. These components consisted of 2 abutments, 4 rectangular columns which represented the abutment piles, a foundation for the central pier, a central circular pier, a rectangular cap beam and two hollow core superstructure spans. In order to facilitate manufacturing and assembly of the superstructure the hollow-core slabs were cast as one piece rather than in smaller segments. PVC pipes were used to provide the hollow cores in the slabs.

Assembly of the specimen started from the rectangular piles which were erected on rectangular steel foundations. The protruding longitudinal reinforcement at the bottom of each pile was welded to a base plate which was in turn bolted to steel foundations connected to the strong floor. A 32mm MacAlloy bar was used to post-tension each pile to 150kN. This was done in order to decrease the tensile stresses in the concrete and increase the bending moment capacity of the piles. The protruding longitudinal reinforcement of each pile (8×25mm Reidbars) at the top was grouted into corresponding ducts inside the abutments to create moment-resisting connections (Figure 5-c). In order to further increase the lateral stiffness of the abutment-pile combination, the piles were braced using 25mm diameter Reidbraces (Figure 4-d).

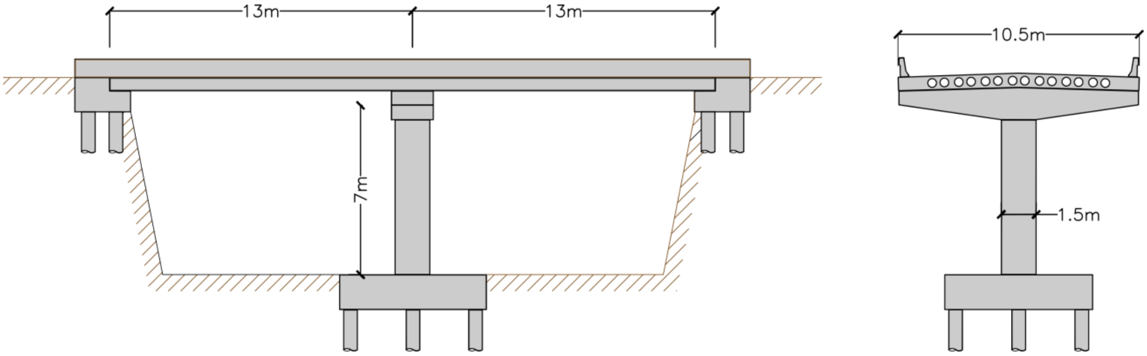
Assembly of the central part of the substructure began by attaching the precast concrete foundation to the strong floor using bolts, and erecting the circular pier on top followed by the cap beam. The foundation, pier and cap beam were then connected using a 40mm MacAlloy bar which was later post-tensioned to 700kN. Two dowel bars measuring 600mm in length connected the pier and cap beam on either side of this post-tensioning to prevent their relative rotation. At the bottom of the pier four 25mm threaded bars connecting the pier to the foundation provided a similar restraint.

The superstructure was simply supported by the abutments and cap beam. The external shear keys on the abutments provided lateral restraint for the superstructure in the transverse direction (Figure 2). Elastomeric bearings of type IRHD60 with a thickness of 25mm were placed vertically between each superstructure segment and corresponding shear key on the abutment to prevent stress concentration and damage to the abutment. Since the aim in this stage of testing was to monitor the response of the superstructure to transverse forces, no shear keys linked the superstructure and the cap beam. The superstructure was post-tensioned in the longitudinal direction using two 15.2mm post-tensioning tendons that passed through the existing ducts in the hollow-core concrete section. A 100mm gap separated the end of each deck and the adjacent abutment. This was to provide sufficient allowance for the rotation of the deck, which would otherwise press against the abutment either causing damage or forcing it to displace in the longitudinal direction of the bridge. In practice this gap can be bridged using methods similar to those used in deck joints, such as link slabs which are constructed using engineered cementitious composites (Li & Lepech 2012) and will allow for considerable deformation with minimal damage.

1.2 Test setup and loading protocol

The test setup and loading protocol are shown below (Figure 4 and Figure 5-c). Transverse loading was provided at the deck level using two rams mounted on reaction frames. The force was applied to the centre of each deck causing them to rotate in opposite directions. The corners of the decks were armoured using steel plates with a thickness of 6-10mm to prevent spalling in areas of stress concentration. The decks were allowed to slide on the cap beam and in order to reduce friction between the sliding surfaces UHMWPE plates were attached to the top of the cap beam and abutments in such a way that the armoured areas of the deck were supported by these raised sections (Figure 5-c).

The loading protocol was based on ACI ITG-5.1-07 (ACI Innovation Task Group 5, 2008), where the maximum force applied to the structure in the first three cycles should not exceed 60% of the design strength and the maximum displacement of the subsequent cycles should be between 1.25 and 1.5 times the maximum displacement in the previous cycles (Figure 5-d). Since the loading was quasi-static and was applied to the specimen in the transverse direction, it was decided that supplementary gravity loading would not affect the results of the testing and therefore was not applied to the bridge.



a) b)

Figure 3 – Prototype bridge which the specimen was based upon a) Elevation view; b) Cross section

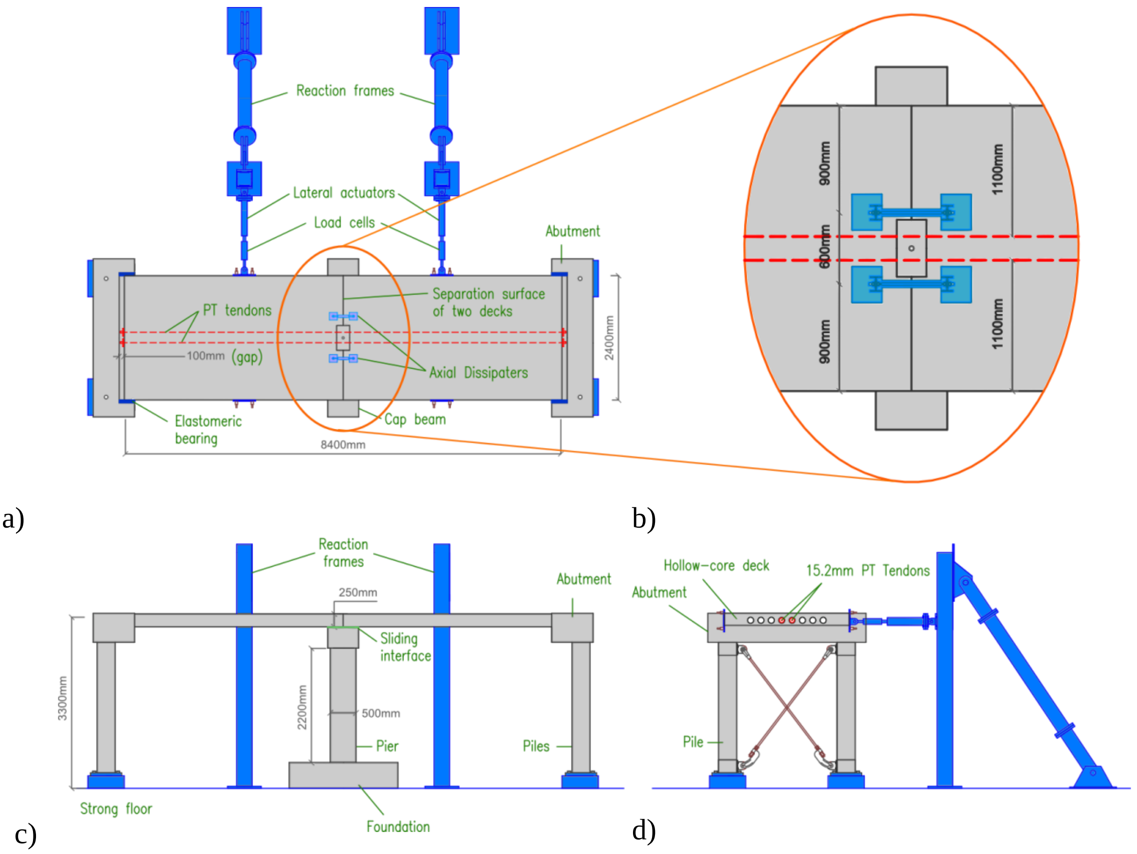
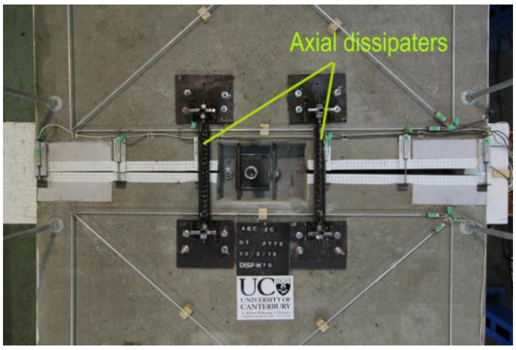


Figure 4 – Setup and dimensions of bridge specimen; a) Plan view; b) Loading protocol; c) Elevation view; d) Cross section



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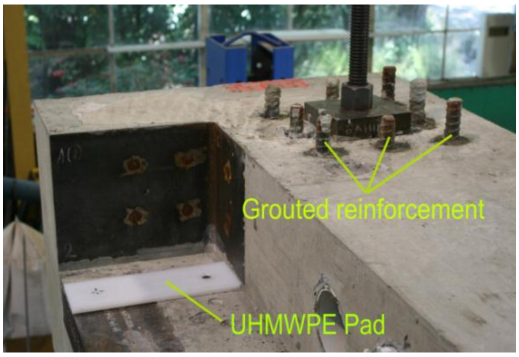
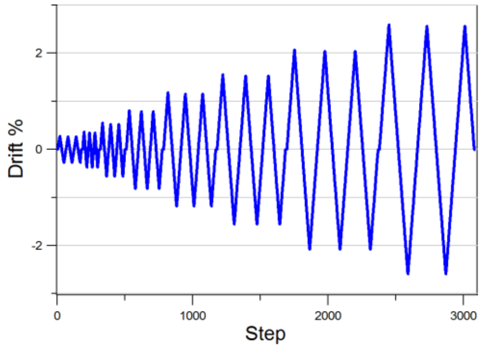




c) d)

Figure 5 – a) View of constructed specimen; b) Axial dissipaters used on deck and gap opening between two superstructure segments during testing; d) Loading protocol

Table 1– Overview of the various testing setups

Test	Post-tensioning force (total) (kN)	Axial dissipaters
A	120	-
B	150	-
C	150	2

Three tests using different configurations of the specimen were selected to be presented in this paper. In configurations A and B, each of the two longitudinal tendons in the superstructure were post-tensioned to 60 and 75 kN respectively (resulting in a 120 and 150kN force in total) and no axial dissipaters were used. The setup of test C was similar to that of test B, however two axial dissipaters with a cross-section of 200 sqmm and fuse length of 440mm each attached the adjacent spans of the superstructure and were placed 900mm from the edge (Figure 4-b). The dissipaters were manufactured using mild steel with a yield stress of 350 MPa.

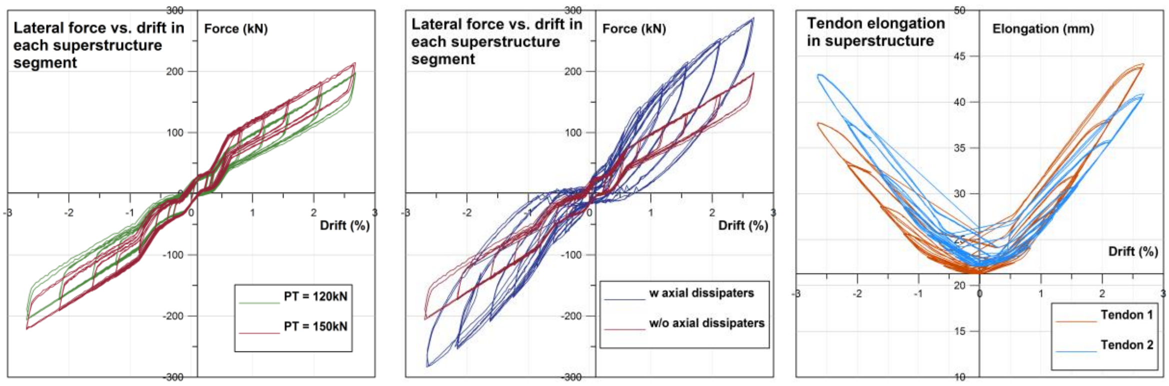
2 RESULTS AND DISCUSSION

The results of the first and second test (Figure 6-a) show the multiple phases of the superstructure's response to lateral transverse loading. The first phase occurs from zero displacement until the specimen has reached a 0.7% drift ratio and represents rigid body translation or sliding of the superstructure relative to the the supports. This can be seen in the force-displacement plots as segments of very low stiffness, and results from compression of the elastomeric bearings protecting the abutment shear keys. The second phase commences after the bearings reach their full compression at 0.6% drift when the superstructure segments begin to rotate and the gap between them starts to open. This is followed by the third phase, in which the gradient of the force-displacement plot is a representation of the increase in post-tensioning forces and resultant re-centring moment as the tendons are stretched further due to the gap opening.

One of the aims in using hybrid connections is minimizing residual displacements which can be achieved by choosing an appropriate ratio between post-tensioning (re-centring) and dissipative forces in the connection (Figure 1). However it can be seen from Figure 6-a that after each loading cycle the superstructure does not relocate to its neutral position of zero drift. This can be attributed to the friction that exists between the superstructure when it is sliding on the bearing pads. This effect can also be seen in the results of the third test (Figure 6-b).

A side-by-side comparison between the results of the first and second tests shows that increasing the post-tensioning level in the superstructure's longitudinal tendons increases the lateral stiffness of the superstructure prior to gap opening but has no such effect afterwards (Figure 6-a).

The results of the third test show a promising flag shaped hysteresis loop which confirm the self-centring and energy dissipation properties of the hybrid superstructure (Figure 6-b).



a) b) c)

Figure 6 – a) Force vs. drift of each superstructure segment for different post-tensioning values; b) Force vs. drift of each superstructure segment for a post-tensioning value of 150kN with/without axial dissipaters between superstructure segments; c) Tendon elongation in each of the superstructure longitudinal tendons for test C

3 CONCLUSIONS

Despite their many advantages the use of prefabricated bridges is still limited in seismic areas due to the need for reliable connections that offer sufficient ductility. This paper presented the findings of an experimental testing program on a 1:30 scale prefabricated concrete bridge with hybrid connections between the superstructure spans. The specimen was pre-fabricated, assembled and tested in the lateral transverse direction using a quasi-static method. The specimen showed promising results in terms of self-centring and dissipation; both which can improve the performance of the bridge during seismic loading. If used in combination with shear keys between the superstructure and cap beam, this connection can contribute to the lateral stiffness of the bridge system in addition to prevention of unseating of simply-supported superstructures and damage to shear keys in a seismic event.

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