EXPERIMENTAL INVESTIGATION INTO THE SEISMIC PERFORMANCE OF HALF-SCALE FULLY PRECAST BRIDGE BENT INCORPORATING EMULATIVE SOLUTION

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\textbf{ABSTRACT}
This paper aims to present experimental observations and results so far for a half-scale fully precast bent similar to what has been proposed in the NCHRP 698 in seismic regions. The bent is representative of a typical highway bridge with 16 m span length in New Zealand. The specimen comprises of member socket connections between the pier and foundation, while using grouted ducts for the pier to cap beam connections. The specimen is designed and detailed to achieve plastic hinging at the base and top of the piers, with no damage to the foundation and cap beam. This type of solution which uses emulative connections can be called “Accelerated Bridge Construction High Damage”. The ongoing research is part of the project titled “Advanced Bridge Construction and Design” (ABCD) funded by New Zealand Natural Hazards Research Platform (NHRP) at the University of Canterbury, New Zealand. As part of the 2\textsuperscript{nd} phase of the research, a similar bent specimen but using a low damage approach is currently being developed. The locations for potential plastic hinging are replaced by a combination of post-tensioning with the external replaceable dissipaters. This solution is named Dissipative Controlled Rocking (DCR) or hybrid connection, and when combined with Accelerated Bridge Construction can be called “Accelerated Bridge Construction Low Damage”. The post-tensioning provides self-centering capability for the piers, while external devices are intended to absorb seismic energy by going through nonlinear deformation. The resultant hysteresis is called “flag-shaped”. For more information regarding ABCD visit \url{http://www.bridgethecommunity.co.nz/}

See the following link to access the testing videos:
\url{http://www.youtube.com/channel/UCnE-dp6hQ2LdPvo2emepxRw}

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Experimental Investigation into the Seismic Performance of Half-Scale Fully Precast Bridge Bent Incorporating Emulative Solution

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ABSTRACT

The National Cooperative Highway Research Program (NCHRP) Report 698 (2011) includes several concepts for the connection of the precast members for ABC. Several types of potential emulative connections are proposed. Emulative solutions aim to achieve the traditional plastic hinging at the high demand locations in a pier. The NCHRP 698 also presents a concept for Highways for LIFE precast bent for seismic regions. In this bent structure, the precast pier is connected to the foundation using member socket connection, the pier to cap beam connection is grouted duct. This paper presents a summary of design, detailing, construction technology, assembly process, experimental testing, and results so far for an emulative half scale fully precast bridge bent in seismic regions which is similar to the concept proposed by NCHRP 698.

Introduction

Accelerated Bridge Construction (ABC) can be defined as any method to speed up the construction of bridges. In case of concrete bridges, the use of precast elements for substructure and superstructure systems can significantly reduce the construction time of a bridge. ABC aims for minimizing traffic disruption, improving safety in the work zone, reducing life cycle costs, improving construction quality, and limiting environmental impacts. Precast concrete has had widespread use in bridge substructures for regions of low seismicity, where it is unlikely that the bridge will be subjected to extreme lateral loads. Figure 1 presents some applications of the precast substructure systems in the United States.

Figure 1. Examples of precast bents in low-seismicity areas in the United States.

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General background on ABC from different nations around the world has been summarized in (Palermo and Mashal, 2012) [1]. Over the past several years, there has been increasing attention given to ABC. A notable example is research into standardized precast substructure systems by (Billington et al., 1999) [2]. There has also been significant interest into ABC by the United States Departments of Transportation including Washington (Khaleghi, 2010) [3], Texas (Ralls et al., 2004) [4], Utah (Burkett et al., 2004) [5] and The Federal Highway Administration (U.S. FHWA, 2011) [6]. However, using precast concrete in regions of high seismicity has been limited mainly due to concerns regarding the performance of connections between the precast components. Lessons from past earthquakes have shown vulnerability of the precast connections in high seismicity (Buckle, 1994) [7]. Therefore, application of ABC in high seismicity requires appropriate solutions proven by experimental testing.

The National Cooperative Highway Research Program (NCHRP) Report 698 (2011) [8] includes several concepts for the connection of the precast members for ABC in seismic areas. Several types of potential emulative connections are proposed, such as grouted duct and member socket connections. These types of connections are designed to emulate conventional cast-in-place connections in performance while offering the advantages of prefabrication. In this paper, they are termed “High Damage” as they are detailed such that the plastic hinges form in the structure during a design earthquake. The formation of plastic hinges means spalling of the concrete and yielding of the steel reinforcement, which then requires repair following an earthquake. High Damage connections offer no supplementary self centering ability to the structure, other than that offered by the weight of the structure, meaning the structure may be left with residual displacements following a seismic event (Palermo and Pampanin, 2008) [9].

Prototype Structure and Testing Arrangement

Description of Prototype

A prototype structure was developed on which the designs of the connections and test specimens are based. This prototype structure is intended to represent a typical highway bridge structure with low to medium span in New Zealand. Figure 2 gives an overview of the prototype structure used for the first specimen.

Figure 2. Prototype structure; (Left) transverse section; (Right) longitudinal profile.
A span length of 16 metres is considered for the prototype bridge. The bridge consists of double column piers with a rectangular pier cap. The superstructure is selected to be I-Beam 1600 section as given in NZTA Research Report 364 (2008) [10]. The columns are circular cross section with a diameter of 1 metre. For simplicity, it is assumed the piers are of an equal height of 5.8 metres to the centre of mass of the superstructure. The footings shown are for indicative purposes and the type of footing will depend on ground conditions as determined by a geotechnical engineer. For testing purposes, it is assumed that the footings are fully fixed. However, further research into soil-structure interaction will take place as part of the ABCD research program in the future. According to NZTA Bridge Manual (2013) [11] for earthquake resistant design of the prototype shown in Figure 2, the energy dissipation system relies on a ductile or partially ductile structure. The plastic hinging is expected to happen at design load intensity in the top and bottom of piers. The plastic hinges will form above ground or normal water level. The maximum allowable design displacement ductility is 6 for this type of structure.

Testing Arrangement

Half scale specimen of Figure 2 was constructed offsite and transported to the lab for quasi-static testing. Figure 3 shows the testing arrangement and loading history (uni-directional) from ACI T1-01 (2001) [12] loading protocol for the specimen. Two hydraulic actuators, each with a capacity of 1000kN, were used to apply gravity and lateral loads to the bent structure as shown in Figure 3 (Left). The gravity load was being held constant (to within approximately ±10kN) during testing.

![Figure 3. (Left) Test Setup; (Right) displacement history for uni-directional loading.](image)

Development of a High Damage Specimen

Gravity and Seismic Loadings

For the half-scale specimen, the gravity loads include self weight of the bent and dead load of the superstructure (390 kN). For this specimen, only dead load of the superstructure is used during testing. Other loads such as live, breaking, etc are not included. The seismic design loading for the specimen was according to NZTA Bridge Manual for soil class A and B (strong rock), return period of 2500 years (bridges of high importance), an assumed ductility of 4.0 at Ultimate Limit State (ULS), and zone factor (Z) of 0.3. This yields to base shear coefficient of 0.706 (base shear of 330 kN) using an equivalent static procedure from Bridge Manual. The design displacement at ULS is 2.1 % drift or 60 mm from the modal response spectrum method as outlined in Section 5.2.6 of NZTA Bridge Manual.
Overview, Design and Construction

The member socket connection (MSC) is formed by embedding a precast element inside another element which can be either precast or cast-in-place. If both elements are precast, then the connection is secured using a grout or concrete closure pour in the preformed socket. The other solution is to have the second element cast around the first one. The former case where both elements are precast was considered in this case as this solution allows for the minimum amount of on-site labor required for construction. MSC can be used for footing to column, column to cap beam, and pile to pile cap locations.

The column itself was designed using NZS 3101 (2006) [13] using conventional design methods. Minimum specified strength for concrete is 40 MPa, steel yielding 500 MPa, and grout strength 50 MPa. The column base contains 8HD16 bars. Transverse reinforcement consists of HD10 bars at a spacing of 75 mm in the socket and plastic hinge region of the column (500 mm above column-footing interface), with the spacing increasing to 150 mm above the plastic hinge region, refer to Figure 4 (Left). 2.1 meter square footings with a depth of 500 mm were used for both columns. The footing was reinforced using a top and bottom grid of HD16 bars at an average spacing of 150 mm and HD16 circular bars at spacing of 150 mm around the socket in top and bottom. A socket of 500 mm depth and 520 mm diameter was used to support the columns. Both the socket walls, and base of column were left roughened during casting through the use of a retarding agent. This leaves aggregate exposed after casting, which provides a better bond between the layer of grout and the precast surfaces. Refer to Figure 5 for construction, and assembly of MSC.

Figure 4.  (Left) Section details; (Middle) socket footings; (Right) column foot inserts and cage.

Figure 5.  Assembly and grouting procedure for member socket connections.


**Detailing Considerations**

The main considerations that are required for this type of connection are the socket depth, column diameter, development length of column longitudinal bars, and the socket diameter relative to the column diameter. Sufficient socket depth is required for the loads from the column to be transferred to the footing. Foot inserts can be used at the base of column to achieve the full development length of the longitudinal bars in the column (Figure 4, Right) without necessarily increasing the socket depth.

For the socket, the loads that must be transferred are axial loads from the weight of the piers and superstructure, and vertical acceleration loads during seismic excitation. Shear and bearing loads must be transferred through the grouted interface between column and footing. Shear forces are induced by vertical loads in the structure including dead loads from the weight of the structure, live loads from vehicle loading, and vertical acceleration loads during seismic loading (Figure 6, Left). Lateral loads also contribute to shear in the grouted interface as shown in Figure 6 (Middle). Inadequate socket depth means there is an insufficient area over which the shear loads can be carried and shear failure of the grouted interface might occur. This leads to a punching shear failure of the structure where the column slips through the footing.

![Figure 6. Internal actions under: (Left) vertical loading; (Middle) lateral loading; (Right) plan view showing radial compressive and tensile hoops stresses under lateral loading.](image)

Lateral loads also induce bearing stress in the grouted interface. A load couple forms in the socket under lateral loading of the structure as shown in Figure 6 (Middle). Increasing the socket depth increases the distance between the coupled loads, implying less bearing force is required to overcome the moment caused by the lateral loading. Insufficient socket depth leads to bearing loads in the interface that exceed the grouts bearing capacity, causing compressive failure of the grout. The bearing stresses induced in the footing by lateral loads are shown in Figure 6 (Right). It can be seen that accompanying the compressive stresses in the radial direction are hoop tensile stresses that lie at a perpendicular direction to the compressive bearing stresses. This tensile stress field causes radial cracks to form which originate at the socket and propagate to the outside of the footing. This cracking can be mitigated by providing reinforcement orientated in the direction of these tensile hoop stresses. This can be achieved by providing circular hoops in the footing or hoops orientated tangentially to the hoop stresses.

Sufficient gap must be left between the column and footing to allow for tolerance when assembling the precast elements, and to allow for flow of grout when pouring into the joint. Experimental testing has found that a 10mm gap is sufficient for adequate grout flow. However, a larger gap may be required on-site to accommodate for construction tolerances. Further research is required to determine the maximum gap width that is permitted to ensure good shear transfer through the grout layer.
Column to Cap Beam Connection: Grouted Duct Connection

Overview, Design and Construction

The grouted duct connection is one in which the reinforcing starter bars extending from one precast element are inserted into ducts which are cast into a second element. Grout is pumped into the ducts through external tubes after assembly and alignment of the segments on top of each other, which then bonds the two elements together. This type of connection accelerates the construction process as it eliminates the need for on-site concrete pouring, with the only wet work required being the formation of a grout bed between the segments and pumping of grout which remains contained inside the ducts of the precast element.

This type of connection can be used for pile to pile cap, spread footing or pile cap to column, column to cap beam and for splices between the column segments or cap beam segments. Examples of the application of grouted duct connection between different precast members can be found in NCHRP Report 681 (2011) [14]. The grouted duct connection has had widespread use worldwide for these purposes. However, it is typically used in capacity protected or low demand parts of the structure, where the precast elements are likely to remain elastic during seismic loading. The testing carried out as part of this research is considering the Grouted Duct connection for use in the connection of column to cap beam. In particular, this research is investigating whether this type of connection allows for plastic hinge formation in the top of column elements without failure of the connection itself or damage to cap beam.

A total of 8HD16 and 4HD10 longitudinal bars were located at the column to cap beam connection. The extra 4HD10 bars at top of the column compared to the base (MSC) was due to a slightly higher moment demand at top of the columns. This can be shown by a simplified static analysis with design base shear and gravity loads acting on the specimen at ULS. The longitudinal bars were grouted into corrugated steel ducts of 50mm diameter which were cast into the cap beam. In this research, the starter bars were extended all the way up to the top of cap beam. The grout can be poured from the top of the cap beam as shown in Figure 8 (Right). A 15mm grouting bed was left at the column to cap beam interface. There was a 100mm un-bonded length at the connection interface between the column and cap beam (Figure 7, Right). The un-
bonded length can be calculated using the NZCS PRESSS Design Handbook (2010) [15] and Priestley and Park (1984) [16]. The purpose of this un-bonded length was to prevent strain concentrations in the starter bars which in turn will lengthen the plastic hinge region as discussed in the next section. HD10 stirrups at 75mm spacing were used to provide shear, confinement and anti-buckling capacity in the plastic hinge regions of the columns. A spacing of 150mm was used above the plastic hinge regions. Shear keys were located at the cap beam to transfer shear loads across the connection interface. In this instance, the dowel action of the rebars was neglected as the shear key capacity was calculated to be sufficient for the applied shear forces.

Figure 8. Assembly and grouting procedure for grouted duct connections.

**Detailing Considerations**

Shear is transferred across the grouted duct connections through a combination of friction and bond in the grouted interface and bearing of the column against the shear keys. For design purposes, it was assumed that the shear load is transferred only through the shear keys. The shear key was designed using the methods outlined in NZS 3101 treating the shear key as a corbel.

Figure 9 (Left and Middle) shows the primary bond mechanism in the corrugated ducts, where tension loads in the column are transferred to the longitudinal starter bars extending from the footing. The primary transfer mechanism in the duct is through bearing of the deformations of the corrugated duct and reinforcing bar against the surrounding grout and concrete. Only a small amount of stress is transferred through chemical adhesion and friction between the steel and surrounding concrete and grout. It is for this reason that a corrugated duct is used in this application. More details can be found in Brenes et al. (2006) [17] which investigated the effects of different type of duct materials on the overall bond strength.

Figure 9. (Left) Primary stress transfer mechanism in corrugated ducts; (Middle) internal actions; (Right) effect of debonding on strain concentration at the interface.
The corrugated duct provides confinement to the grout surrounding the bar, enhancing the strength of the grout and increasing the ultimate bond strength of the bar. This means that full transfer of stress from the surrounding concrete to the reinforcing bar can occur over a shorter length that is achieved in a conventionally reinforced column. However, in this research, the full development length for the bars as specified in NZS 3101 was allowed for. The increased bond strength leads to a lower length of strain penetration at the connection interface. The strain penetration length is defined as the distance of dowel debonding on each side of the interface.

Kawashima et al. (2001) [18] studied the effects of un-bonded length on reinforced concrete columns. The study concludes that the failure of concrete in the column with un-bonded length was significantly less than the column in which the full length of the rebars was bonded and that the un-bonded length can enhance the ductility of the concrete bridge columns. The use of an un-bonded length at the connection interface (Figure 9, Right) helps to mitigate the effect of strain penetration by spreading the total longitudinal deformation of the bar over a larger length, leading to lower levels of strain in the bar. By leaving the un-bonded length, the interface between the column and footing activates a rocking mechanism also known as gap opening.

**Testing Results and Performance Evaluation**

For the top member socket connections, cracks initiated during the 0.2% drift cycle. Further cracking occurred at higher levels of drift with a distribution of cracks along the half height of the column, but larger cracks widths towards the base of column, indicating more distribution of inelastic deformation in the column. Minor spalling of concrete initiated during the 1.5% drift cycle, with the extent of spalling increasing during larger drift cycles. During the 3.4% drift cycle, spalling had extended to approximately 500 mm from the top face of the footing, see Figure 10 (middle row).

For the bottom grouted duct connection, the cracks initiated at similar drifts as MSC. The grouting bed started deteriorating at 1.5% drift cycles. Minor spalling of cover concrete initiated during the 2.8% drift cycle. The extent of spalling increased during the 3.4% drift cycle, reaching a height of approximately 200 mm below the bottom face of the cap beam at the end of test, Figure 10 (top row). The test was stopped following 3.4% drift cycles (1.5 times ULS). It was obvious that the rupturing point for the rebars is greater than 3.4%.

There was no premature failure or bar rupturing. There were few hairline cracks at the panel zones (Figure 10, bottom row). There was no damage or cracking to the footings. The system showed a very stable hysteresis by forming plastic hinges at top and bottom of the columns. There was slight jump in base shear in pulling phase than pushing. This asymmetric behavior can be thought as softening of the specimen following the pulling phase.

The system showed a very stable hysteresis by forming plastic hinges at top and bottom of the columns. The moment distribution and measured crack widths at the plastic hinges for different limit states are shown in Figure 11. Note that the moment capacities of all four connections are approximately the same.
Figure 10. Damage pattern at different drift levels (Top) GDC (Middle) MSC (Bottom) Extent of damage in all four panel zones at the end of testing.

Figure 11. Moment distribution and measured crack widths in (mm) at the plastic hinges.

Using the displacement procedure outlined in Austroads Technical Report [19], the yielding displacement was calculated to be 24 mm (0.82% drift). Using the strain limits Austroads Report, the displacements for the serviceability and ultimate limit states were
calculated to be 38.28 mm (1.31% drift) and 77 mm (2.64% drift) respectively, refer to Figure 12 (Left). These points were plotted on an equivalent multi-linear force-displacement envelope as shown in Figure 12 (Right). At the serviceability limit state, the ductility, $\mu$, was equal to 1.6. At the design level (ULS), the ductility was 3.2 satisfying the initially assumed $\mu = 3$ for the seismic loading. At the end of the test, the ductility was 4.2. It was clear that the ductility was going to be in excess of 4.2 at the failure point for the bent. There was a slight jump in base shear in pulling phase than pushing for bigger drift cycles (Figure 12). This asymmetric behavior can be associated to the location of the displacement controller which was mounted on the right side of the specimen, where the horizontal ram was pulling and pushing the specimen from the left end. In order to get a symmetrical behavior, the point of load application must be shifted following a pull half cycle. Another factor can be softening of the specimen following a push /pull.

As expected, there were four plastic hinges formed in the bent. Figure 13 shows cyclic moment-curvature plots for the grouted duct and member socket connections. For the GDC, it can be observed that the connections have less strength degradation compared to the MSC. One reason for this can be rocking mechanism of the grouted duct connection, the 100 mm unbonded length of the starter bars at the plastic hinging zone have caused less spalling of the cover concrete which has resulted in less strength degradation of the member.

Figure 12. Hysteresis plots: (Left) base shear vs. drift (Right) envelop with performance limits.

Figure 13. Moment-Curvature plots: (Top) top GDC (Bottom) bottom MSC.
Conclusions
The experimental testing showed promising results for using the GDC and MSC for a precast bent in seismic regions. The bent achieved good strength and ductility levels by formation of four plastic hinges similar to what can be expected from a cast-in-place construction. This type of bent construction provides the potential for significant time savings (precast cap beam, columns, and possibly footings) through avoiding the need for pouring of concrete on the site.

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