SUMMARY

This paper summarises the findings from the first phase of testing at the University of Canterbury into the precast concrete elements using wet connections to emulate cast-in-place (CIP) performance. The emulative solution is called Accelerated Bridge Construction (ABC) High Damage. Three connections were tested through uni and bi-directional loading; two feature the Grouted Duct Connection (GDC) as the primary connection type, while the third features the Member Socket Connection (MSC). This paper gives an introduction to each connection type followed by specific design and detailing considerations. The construction method for each connection is also outlined and followed by a discussion of the performance of the connections during experimental testing.

INTRODUCTION

The use of precast concrete for construction of bridge substructures offers a number of advantages over the conventional use of cast in place (CIP) concrete. These advantages include increased construction speed and quality, minimised traffic disruption, reduced life cycle costs of the structure, and increased construction safety. General background has been summarized in (Palermo and Mashal 2012). The use of precast concrete to rapidly construct concrete bridges is commonly known as Accelerated Bridge Construction (ABC).

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). There has also been significant interest into ABC by the United States Departments of Transportation including Washington (Khaleghi 2010), Texas (Ralls et al. 2004), Utah (Burkett et al. 2004) and The Federal Highway Administration (U.S. FHWA 2011).

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. The use of precast concrete in regions of high seismicity has been limited mainly due to concerns regarding the performance of connections between the precast components.

Current research at the University of Canterbury (UC) as part of the research program, funded by the New Zealand Natural Hazards Research Platform (NHRP) and titled “Advanced Bridge Construction and Design” (ABCD) aims to address these concerns through testing of a range of precast connection types for use in bridge substructures. A range of connection types are being investigated, these are characterised as either “High-Damage”, “Controlled Damage” or “Low Damage” connections with each category corresponding to the type and extent of damage that is to be expected during seismic loading. This paper focuses on the use of High Damage connections also known as...
emulative connections for bridge substructures. 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.

High Damage connections are designed to emulate conventional cast in place connections in performance while offering the advantages of prefabrication. They are termed “High Damage” as they are detailed such that the plastic hinges form in the structure. 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). The first phase of testing at the University of Canterbury looks at three variations of High Damage connection. Two specimens feature grouted duct connections and one with member socket connection. The grouted ducts have already been used in non-seismic and seismic regions. There is a significant amount of research done on this type of connections (NCHRP Report 698 2011).

PROTOTYPE STRUCTURE AND TESTING ARRANGEMENT

Prototype Structure

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 and is based on the geometry of the Port Hills Overbridge located in Christchurch (-43.573° S, 172.695° E). This bridge represents a typical low to medium span highway bridge structure in New Zealand. Figure 1 gives an overview of the prototype structure used for the first phase of the research.

![Figure 1](image)

A span length of 12 metres is considered for the prototype bridge. The bridge consists of single column piers supporting a hammerhead pier cap and double Hollowcore precast superstructure sections as given in NZTA Research Report 364 (2008). The columns are of either circular or square cross section with a section width of 1 metre. For simplicity, it is assumed the piers are of an equal height of 5 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. According to
NZTA Bridge Manual (2003) for earthquake resistant design of the prototype shown in Figure 1, 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 pier above the ground level. The maximum allowable design displacement ductility is 6 for this type of structure.

Testing Arrangement

Half scale specimens were constructed offsite and transported to the lab for quasi-static testing. Figure 2 shows the testing arrangement and loading history (uni and bi-directional) from ACI T1-01 (2001) loading protocol for the first phase of the research.

Hydraulic actuators were used to apply lateral load to the columns. Lateral load was applied in one direction only for the uni-directional tests, while two actuators were used for the bi-directional test. In addition to the lateral loads, an axial load was applied to the specimens to simulate the gravity load in the structure. This was applied through the use of a Macalloy bar running down the centre of the columns. A hollow hydraulic cylinder was located on top of the column to stress the bar during testing, with the load being held constant (to within approximately ±5%) during testing.

Figure 2. (Left) Elevation view of testing setup; (Middle and Right) Displacement history for uni and bi-directional loading respectively

GROUTED DUCT CONNECTION

Connection Overview

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). 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.
This testing carried out as part of this research is considering the Grouted Duct connection for use in the connection of footing to column and the connection of column segments. In particular, this research is investigating whether this type of connection allows for plastic hinge formation in the column element without failure of the connection itself.

**Design and Construction**

Two segmental columns featuring the Grouted Duct connection were tested. These columns were half scale specimens based on the prototype structure using square cross sections of 500mm depth. Grouted Duct Connections were located at both the footing-column interface and the interface between column segments, see Figure 3.

A total of 16 HD16 longitudinal bars were located at the lower connection while only 8 bars of the same type were used at the upper connection due to reduced flexural demand. The longitudinal bars were grouted into corrugated steel ducts of 40mm diameter which were cast into the base of each column segment. HD10 stirrups at 50mm spacing were used to provide shear, confinement and anti-buckling capacity in the plastic hinge regions of the columns. A spacing of 100mm was used above the plastic hinge regions. Shear keys were located at both connections 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.

A 2.1 metre square footing with depth of 500mm was used at the base of the columns and was anchored to the floor using hold-down bolts. Footing reinforcement consisted of a top and bottom grid of HD16 bars at an average spacing of 150mm. Additional reinforcement was included in the shear key which was used to transfer shear loads from the footing to column.

SQ-1 featured longitudinal bars which were fully bonded, with no armouring used to protect and confine the cover concrete at the connection interface. SQ-1 was intended for unidirectional testing. Figure 4 (Left) shows components for SQ-1.

The second specimen (SQ-2) was intended for the bi-directional testing. It featured a 120mm un-bonded length at the connection interface between the column and footing. 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. Steel armouring was also used to confine the concrete at the connection interface preventing spalling. The armouring consisted of 100x100x5 equal angle around each side of the column (Figure 4, Right). Figure 5 and 6 show construction of SQ-1 and SQ-2.
Detailing Considerations

Figure 7 (Left) shows the internal actions in the grouted duct connection under vertical loading, while Figure 7 (Middle) shows the internal actions under lateral loading. 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 key. The shear key was designed using the methods outlined in Section 16 of NZS3101 (2006) treating the shear key as a corbel.

Figure 8 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 or grout. It is for this reason that a corrugated duct must be used in this application. More details can be found in Brenes et al. (2006) which investigates the effects of different type of duct materials on the overall bond strength. The use of a straight pipe with no corrugations would mean no interlock of the grout and concrete keys leading to greatly reduced ultimate bond strength.

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 (2006) was allowed for.

![Figure 7. Internal actions under: (Left) Vertical loading; (Middle) Lateral loading; (Right) Strain distribution in longitudinal bars with and without an un-bonded length](image)

![Figure 8. (Left) Primary stress transfer mechanism in corrugated ducts; (Right) Internal actions through shear key under lateral loading](image)

The increased bond strength leads to a lower length of strain penetration at the connection interface. The strain penetration length can be defined as the distance of dowel debonding on each side of the interface.

As the column displaces during lateral loading, the largest cracks will occur at the interface of the column and footing. Since no tension load is carried across these cracks by the concrete, the steel must carry the load, leading to stress concentrations in the steel at the crack locations. A shortened length of strain penetration means a shorter length of bar is accommodating the total deformation of the bar caused by the crack opening. This leads to strain concentrations in the bar at the crack location. This is illustrated by Line A in Figure 7 (Right). Higher strains mean the bar is more likely to fail due to low cycle fatigue, or the column will undergo a lower ultimate drift before bar failure occurs. Kawashima et al. (2001) 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 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. This is illustrated by Line B in Figure 7 (Right). By leaving the un-bonded length, the interface between the column and footing activates a rocking mechanism also known as gap opening.

Lower strains mean premature low cycle failure of the bar is less of an issue and that the column is able to undergo higher ultimate levels of drift. It must be noted that providing un-bonded length also increases the yield drift of the column. However, it has been observed that the increase in yield drift is generally less than the increase in ultimate drift capacity, which overall means a higher level of displacement ductility at the failure point of the column is achieved. The required un-bonded length for the bars in specimen SQ-2 was estimated from the results of the testing of specimen SQ-1.

The plastic hinge length, $L_p$, for a conventional monolithic column can be estimated using:

$$L_p = 0.08L_{cant} + l_{sp}$$  \hspace{1cm} (1)

Where $L_{cant}$ is the distance from the footing face to the point of contraflexure in the column and $l_{sp}$ is the strain penetration length. In a conventional column where the bars are not enclosed in ducts, the strain penetration length is defined by Priestley and Park (1984) as follows:

$$l_{sp} = 0.022d_b f_y$$  \hspace{1cm} (2)

where $d_b$ is the bar diameter and $f_y$ is the yield strength of the bar. For a 2.5 metre tall column with 16mm diameter, grade 500 bars, we would expect a strain penetration length of 176mm and a plastic hinge length of 376mm. Re-arranging equation 1 gives:

$$l_{sp} = L_p - 0.08L_{cant}$$  \hspace{1cm} (3)

This allows the strain penetration length to be estimated from a plastic hinge length observed during testing.

Testing of the first grouted duct connection with no un-bonded length left at the connection interface showed the plastic hinge length to be approximately 250mm. This gives a length of strain penetration of just 50mm which is considerably less than the 176mm that would be expected of the same longitudinal bar without the confinement provided by the duct. Using these values, it was estimated that an un-bonded length of 120mm would give an effective length of strain penetration of approximately 170mm length, leading to similar behavior from the grouted duct connection as would be expected in a conventional monolithic connection.

A more accurate method for determining the required un-bonded length is to consider the connection as a rocking interface as outlined in the NZCS PRESSS Design Handbook (2010), where sufficient unbonded length is provided to limit the strain in the reinforcing bars to less than 5%.

**MEMBER SOCKET CONNECTION**

**Connection Overview**

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.

**Design and Construction**

Two segmental columns (CR-1 and CR-2) featuring the member socket connection were tested. Both columns featured a member socket connection between the footing and column and a grouted duct connection between the column segments. Both columns were identical in order to investigate the differences in behavior under uni-directional and bi-directional loading. Figure 9 shows section details for the specimens.

The column itself was designed using NZS3101 (2006) using conventional design methods. The lower segment contains 16 HD16 bars while the upper segment contains 8HD16 bars. Transverse reinforcement consists of HD10 bars at a spacing of 50mm in the socket and plastic hinge region of the column, with the spacing increasing to 100mm above the plastic hinge region.

2.1 metre square footings with a depth of 500mm were used for CR-1 and CR-2. The footing was reinforced using a top and bottom grid of HD16 bars at an average spacing of 150mm. A socket of 500mm depth and 520mm 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. Figure 10 and 11 show construction, assembly, and grouting procedure for MSC.

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**Detailing Considerations**

The main considerations that are required for this type of connection are the socket depth, column diameter, 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. 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. Lateral loads also contribute to shear in the grouted interface as shown in Figure 12 (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.

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 12 (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. This is illustrated by Figure 13 where a reduction in socket depth of 25% leads to increase in bearing forces of about 40%. If insufficient socket depth is available, a partial socket could be used where the socket does not extend all the way through the footing could be used. Alternatively, a shear key in the socket could be used to provide interlock between the footing and column. Increasing the column diameter also increases the area of grout available for bearing and shear stress transfer, reducing the depth of socket required.

The bearing stresses induced in the footing by lateral loads are shown in Figure 12 (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 straight bars 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. The gap should not be too large. However, as this will reduce the effectiveness of the grout interface to transfer shear between the precast elements. 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.

RESULTS AND DISCUSSION

For SQ-1, spalling of concrete occurred during the 3% drift cycle. The extent of spalling increased during the 4% drift cycle reaching a height of approximately 200mm above the top face of the footing. Bar rupture occurred at a drift of 5.0%. By the end of the test, spalling occurred to a height of approximately 250mm. From the load-displacement plot (Figure 14, Left) it can be seen that the column yielded at a displacement of 19mm corresponding to a drift of 0.75%. The ductility at failure point was > 6. The moment-curvature plot is shown in Figure 14 (Right). The post-stiffness behavior was due to the grouted Macalloy bar which has influenced the capacity of the section not the ductility.
SQ-2 behaved better than SQ-1. There was less spalling of the concrete in plastic hinge zone. The specimen had a similar ductility as SQ-1. The un-bonded length of the starter bars and armouring at the base were very effective for the post yielding behavior. The bar rupture occurred at 6% drift after running a significant number of large cycles (>5% drift) with a bi-directional loading scheme. A comparison of the damage at the end of tests for SQ-1 and SQ-2 is shown in Figure 15.

![Figure 15. (Left and Middle) SQ-1 at the end of test; (Right) SQ-2 at the end of test](image)

For CR-1, cracks initiated during the 0.25% drift cycle. Further cracking occurred at higher levels of drift with a similar distribution of cracks as the first segment but larger crack widths towards the base of the column indicating more distribution of inelastic deformation in the column. Minor spalling of concrete initiated during the 3% drift cycle with the extent of spalling increasing during larger drift cycles. During the 6.0% drift cycle, spalling had extended to approximately 500mm from the top face of the footing, see Figure 16 (right).

![Figure 16. Crack propagation in CR-1](image)

The load-displacement plot (Figure 17, Left) shows that the column yielded at a drift of 1%. The ductility at failure point was equal to 6. A good correlation between the theoretical and experimental moment-curvature plots was observed in the second column indicating the Macalloy bar performed as desired in simulating the axial load (Figure 17, Right).
CONCLUSION

The experimental testing showed promising results for both the GDC and MSC with both achieving good strength and ductility levels similar to what can be expected from a CIP construction. Use of armouring and un-bonded length at the starter bars can significantly enhance the ductility of GDC. However, further research on strain penetration effects might be required. Both GDC and MSC provide the potential for significant time savings through avoiding the need for assembly of formwork and pouring of concrete on site.

ACKNOWLEDGEMENTS

The authors would like to express their gratitude to the Ministry of Science and Innovation (MSI) – NHRP for supporting this research as part of the project ABCD. The authors are thankful of technician Gavin Keats and Russell McConchie for helping with the testing.

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