Quasi-static cyclic testing of half-scale fully precast bridge substructure system in high seismicity

M. Mashal & A. Palermo

Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch.

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ABSTRACT: Accelerated Bridge Construction (ABC) can be defined as any type of 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. Past earthquakes have shown vulnerability of the precast connections in seismic areas. Recently, there have been several emulative concepts proposed to achieve similar seismic behaviour from a precast connection in high seismicity as expected from a conventional cast-in-place construction. As part of the project Advanced Bridge Construction and Design (ABCD) funded by New Zealand Natural Hazards Research Platform (NHRP) at the University of Canterbury (UC), several of the proposed emulative concepts were previously tested for cantilevered bridge columns.

In this paper, two half scale fully precast bridge bents are developed for a typical highway bridge with 16m span length in New Zealand. The first specimen incorporates ductile emulative connections which aim to limit the damage only in the columns by formation of plastic hinges at the design earthquake level. Therefore, it can be called "ABC High Damage".

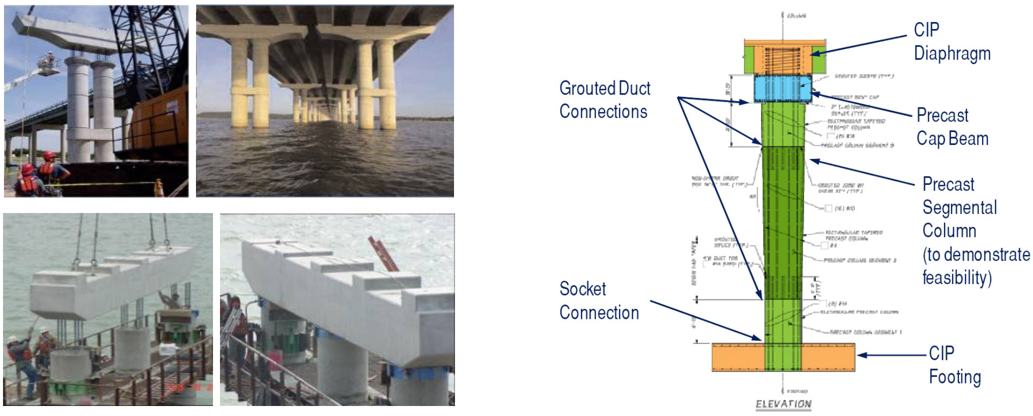
1 INTRODUCTION

In the past several decades, the cast-in-place (CIP) construction of the bridge substructure systems has been the traditional and preferred method of construction in many countries around the world. However, there are a number of challenges identified over the years with the CIP practice. These challenges include but are not limited to, traffic disruption in urban areas, construction safety, higher life cycle costs, construction quality control, and environmental impacts. Accelerated Bridge Construction (ABC) is intended to provide solutions for the aforementioned challenges. ABC can be defined as any method of construction that can accelerate the construction time of a bridge structure. In the recent years, the use of prefabricated elements for the bridge superstructure system has been popular among several nations around the world such as the United States, New Zealand, Taiwan, and Japan. However, the CIP construction has still remained the primarily method of constructing the bridge substructure system in a moderate to high seismic region.

Palermo and Mashal (2012) presents general background on ABC from many countries around the world. More information on development and application of ABC over the past several years can be found in Billington et al. (1999), Khaleghi (2010), Ralls et al. (2004), Burkett et al. (2004), and The Federal Highway Administration (2011). So far, most applications of precast substructure system in context of ABC in countries such as United States have been limited to areas with low seismicity. This has been due to uncertainty and doubts in seismic performance of precast connections in high seismicity. Buckle (1994) has studied vulnerability of the precast connections in highway bridges following the Northridge Earthquake. Therefore, using precast substructure system for ABC in regions with moderate to high seismicity requires development and testing of ductile and reliable connections. The National Cooperative Highway Research Program (NCHRP) Report 698 (2011) proposed several concepts for the connection of the precast members for ABC in high seismicity. These connections include, the member socket, coupler rebar, grouted duct, integral, pocket, and hybrid (controlled dissipative rocking). A majority of the proposed connections are intended to emulate the traditional

CIP behaviour during a severe seismic event. This means, the connections should be designed and detailed to match at least the level of ductility and strength that can be expected from a CIP construction. Thus, in this case the proposed connections should be sufficient enough to form plastic hinges in the precast element without suffering any damage or deformation in the panel zones (column to footing or column to cap beam), NCHRP Report 698. However, the formation of plastic hinges would result into cracking, yielding of flexural rebars, spalling of cover concrete, and ultimately rupturing of the rebars in the precast member during a strong earthquake. While the bridge may suffer structural damage as explained above, but it should remain functional and open to traffic following an earthquake. This type of prefabricated solution is termed "ABC High Damage" in this paper. High Damage solution offers no supplementary self-centring capability to the structure, meaning there will be residual displacement in the structure following a seismic event, as shown by Palermo and Pampanin (2008). The bridge system is offering the advantages of prefabrication, but would need repair or possible replacement after an earthquake.

As part of ABCD project at University of Canterbury, four half-scale precast segmental cantilever columns for a typical prototype highway bridge with 12 m span supporting a duo hollow core superstructure system from NZTA Research Report 364 (2008) were tested earlier. The type of connection between column to footing and segment to segment were member socket and grouted duct connections respectively. The NCHRP Report 698 also presents a concept for Highways for LIFE precast bent for seismic regions as shown in Figure 1(b). In this bent structure, the precast column is connected to the foundation using member socket connection, the pier to cap beam connection is grouted duct. Some applications of grouted duct connection for the precast bents in the United States are shown in Figure 1(a). However, most of the applications have been limited to low seismicity areas.



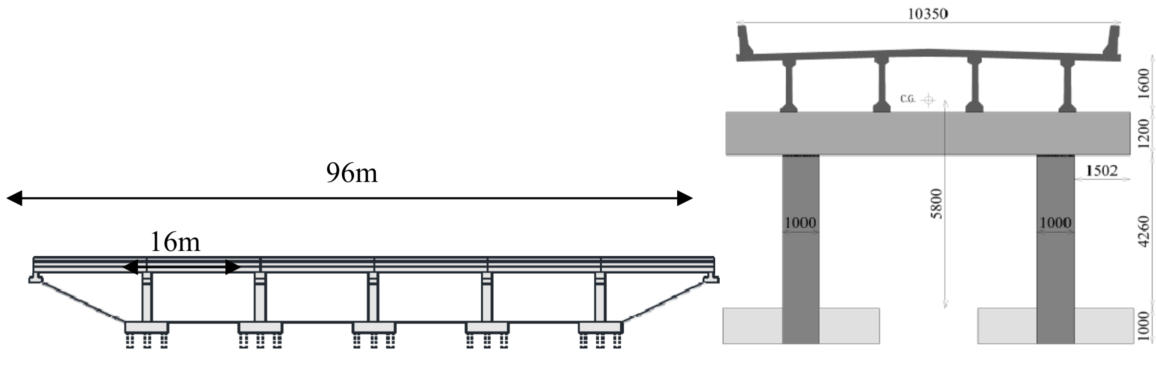
(a) (b)

Figure 1. (a) Examples of Precast bent in the United States (b) Highways for LIFE Precast bent concept for seismic regions after NCHRP Report 698 (2011)

This paper presents prototype development, design detailing, construction, assembly, and experimental testing of a half-scale fully precast bridge bent. The prototype is similar to concept for Highways for LIFE bent but the seismic connection details have never been proof tested before.

2 PROTOTYPE STRUCTURE

The prototype structure is a representative of a typical highway bridge pier support with low to medium span in New Zealand. The design of the connection and test specimen was based on the prototype shown in Figure 2. A span length of 16 meters 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). The columns are circular cross section with a diameter of 1 meter. For simplicity, it is assumed the piers are of an equal height of 5.8 meters to the centre of mass of the superstructure. The footings shown are for indicative purposes. For testing purposes, it is assumed that the footings are fully fixed. According to NZTA Bridge Manual (2013) 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 be 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 structure.



(a) **(b)**

Figure 2. Prototype bridge: (a) longitudinal profile (b) transverse section

3 DEVELOPMENT OF THE SPECIMEN

3.1 Gravity and seismic loadings

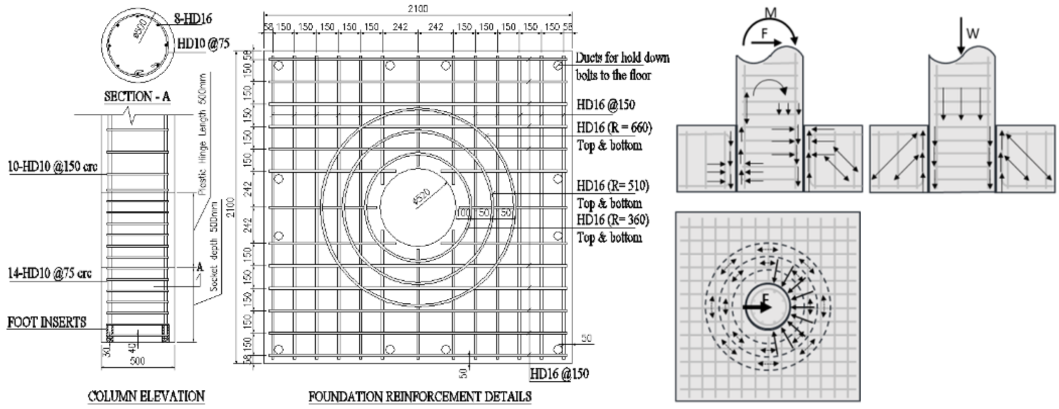
For the half-scale specimen, the gravity loads include self weight of the bent and dead load of the superstructure (390 kN). For this specimen, only dead load of the superstructure is used during testing, this is consistent with the Ultimate Limit State (ULS) combinations of NZTA Bridge Manual (2013). Other loads such as live, breaking, wind, etc have not been considered. The seismic loading for the specimen was calculated 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 3.0 at ULS. A ductility of 3.0 means, there will be significant nonlinear deformation at locations where the plastic hinges will be formed. For a bent structure, the plastic hinges are likely to happen at the top and bottom of the columns where there are higher moment demands. The zone factor (Z) was chosen to be of 0.3 which represents a region with high seismicity. 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.

3.2 Column to footing connection

The column to footing connection is a member socket connection. 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. This type of connection 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) using conventional design methods. Minimum specified strength for concrete is 40 MPa, steel yielding 500 MPa, and grout strength 50 MPa. 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, see Figure 4(c). Column and footing details are shown in Figure 3.

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 in order to prevent possible punching shear failure. Foot inserts can be used at the base of column to achieve the full development length of the bars, Figure 3(a). The internal actions for the applied loads for MSC are shown in Figure 3(c).

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 10 mm gap is sufficient for adequate grout flow. However, a larger gap may be required on-site 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. Figure 4 presents the construction sequence for the MSC.



(a) (b) (c)

Figure 3. (a) Column section (b) footing reinforcement (c) internal actions under: (1) lateral loading (2) vertical loading (3) plan view showing radial compressive and tensile hoops stresses under lateral loading

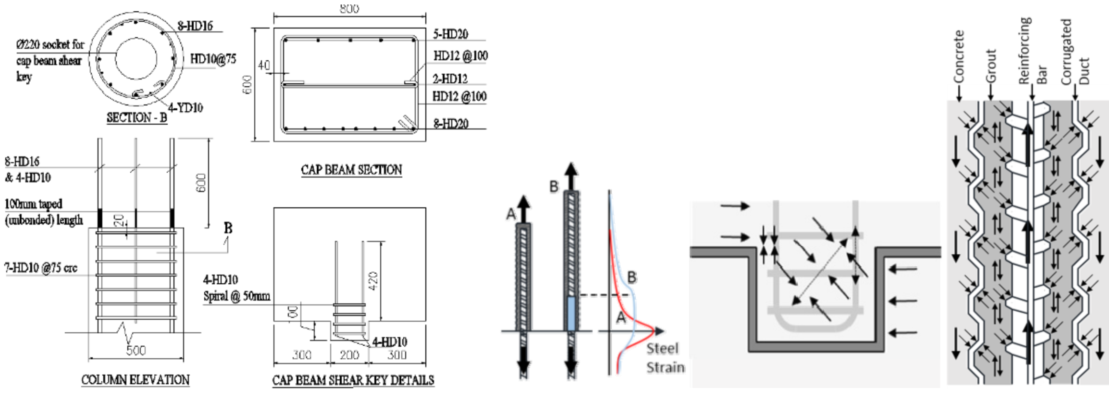
(a) (b) (c) (d) (e)

Figure 4. MSC assemblies and grouting sequence: (a) precast footing with the socket (b) placing the precast column (c) sliding the precast column with roughened ends for bond improvement in footing socket (d) grouting the interface (e) columns erected

3.3 Column to cap beam connection

The column to cap beam connection is grouted duct connection (GDC). In this type of connection, 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 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. The extra 4HD10 bars at top of the column compared to the base (MSC) was due to a slightly higher moment demand, Figure 5(a). The longitudinal bars were grouted into corrugated steel ducts of 50 mm 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. A 15 mm grouting bed was left at the column to cap beam interface. The grout can be poured from the top of the cap beam as shown in Figure 6.

There was a 100 mm un-bonded length at the connection interface between the column and cap beam, refer to Figure 5(a). The un-bonded length can be calculated using the NZCS PRESSS Design Handbook (2010) and Priestley and Park (1984). 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. 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 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, Figure 5(c). By leaving the un-bonded length, the interface between the column and cap beam activates a rocking mechanism or gap opening which forms a single crack and yields unbonded reinforcement. Shear keys were located at the cap beam to transfer shear loads across the connection interface. 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 as shown in Figure 5(d).



(a) (b) (c) (d) (e)

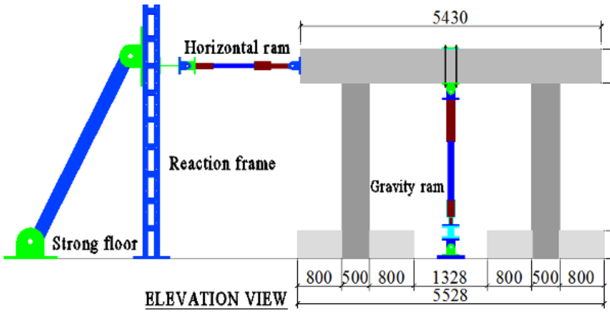
Figure 5. (a) Column details (b) cap beam sections (c) effect of debonding on strain concentration at the interface (d) internal actions in the shear key (e) primary stress transfer mechanism in corrugated ducts

For design purposes, it was assumed that the shear load is transferred only through the shear keys. In this instance, the dowel action of the rebars was neglected. The shear key was designed using the methods outlined in NZS 3101 (2006) treating the shear key as a corbel. Figure 5(e) 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 column. 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. More details can be found in Brenes et al. (2006) which investigated the effects of different type of duct materials on the overall 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. 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. Figure 6 presents construction sequence for GDC.

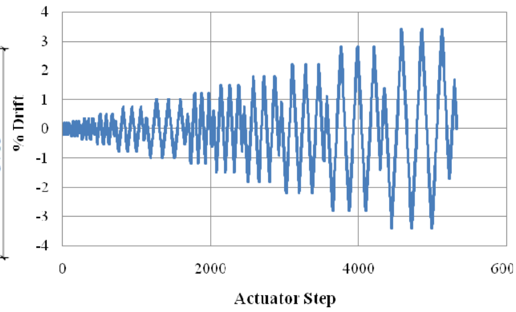
Figure 6. GDC assemblies and grouting: (a) lowering the cap beam on columns and aligning the starter bars into the ducts (b) grouting bed at the interface (c) pouring grout from the top and filling the ducts (d) assembled bent

4 TESTING ARRANGEMENT

A half scale specimen of the prototype shown in Figure 2 was constructed offsite and transported to the lab for quasi-static testing. Figure 7(a) shows the testing arrangement and loading history (uni - directional) from ACI T1-01 (2001) loading protocol for the specimen, Figure 7(b). Two hydraulic actuators, each 1000 kN capacity, were used to apply gravity and lateral loads to the bent structure.



(a)



(b)

Figure 7. (a) Testing setup (b) displacement history for cyclic testing from ACI T1-01(2001)

5 TESTING RESULTS AND OBSERVED PERFORMANCE

For the MSC, 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 8(middle row).

For the GDC, 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 8(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 of the joints and columns. No damage or cracking to the footings was observed. However, there were few hairline cracks at the panel zones, Figure 8 (bottom row).

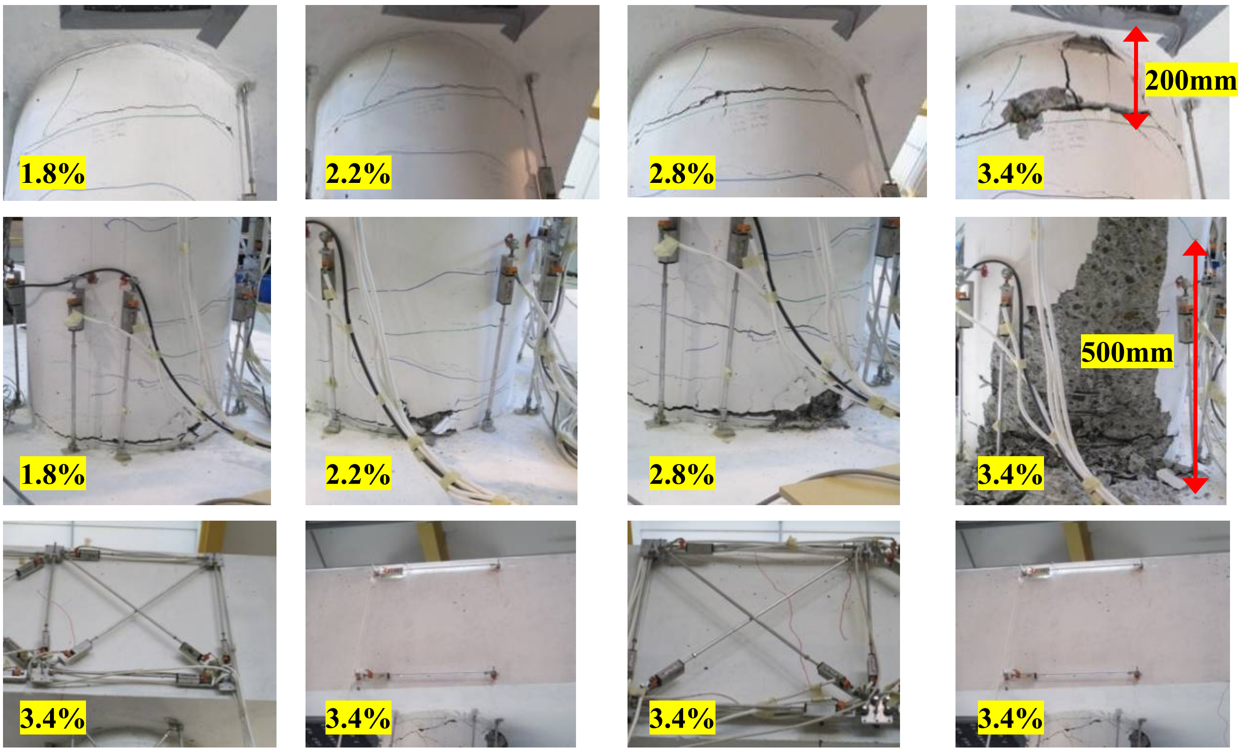


Figure 8. Damage pattern at different drift levels (top) grouted duct connection (middle) member socket connection (bottom) extent of damage in all four panel zones at the end of testing

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 9. Note that the moment capacities of all four connections are approximately the same. Table 1 presents a summary of material strain limits with the quantitative performance description (crack widths) for each performance level from Austroads Technical Report.

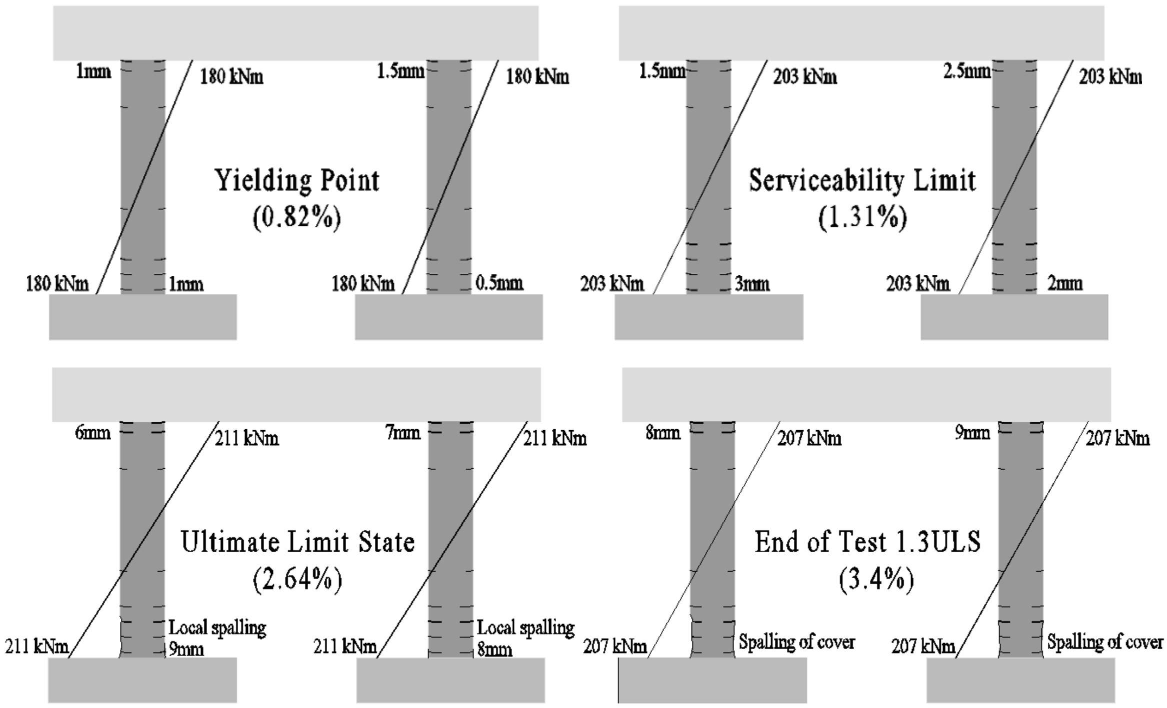
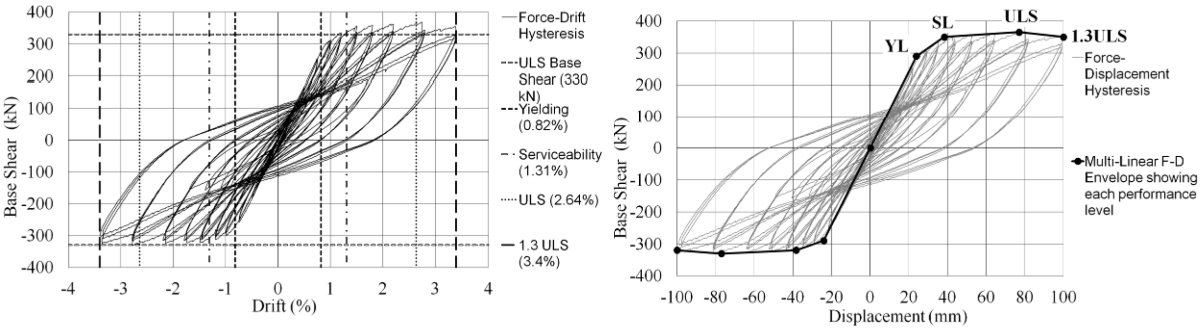


Figure 9. Moment distribution and measured crack widths in (mm) at the plastic hinges for limit states

Table 1. Summary of strain limits and associated crack widths for different performance levels.

Limit States	Reinforcing Steel Tensile Strain	Concrete Compressive Strain	Crack Width in (mm)	Operational Performance Level	Repair
Yielding	0.00275	<0.004	<1 mm	Fully operational	No repair / limited epoxy injection
Serviceability	0.015	0.004	1-2 mm	Delayed operational	Epoxy injection / concrete patching
Ultimate Limit State	0.0448	0.0176	>2 mm	Delayed operational	Extensive repair / reconstruction

Using the displacement procedure outlined in Austroads Technical Report, the yielding displacement was calculated to be 24 mm (0.82% drift). Using the strain limits from Table 1, 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 10 (a). These points were plotted on an equivalent multi-linear force-displacement envelope as shown in Figure 10 (b). At the serviceability limit state, the ductility, μ, 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 10). This asymmetric behaviour 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 behaviour, 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.



(a) **(b)**

Figure 10. Hysteresis plots for the bent structure: (a) base shear vs. drift (b) cyclic envelop showing performance limits

As expected, there were four plastic hinges formed in the bent. Figure 11 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.

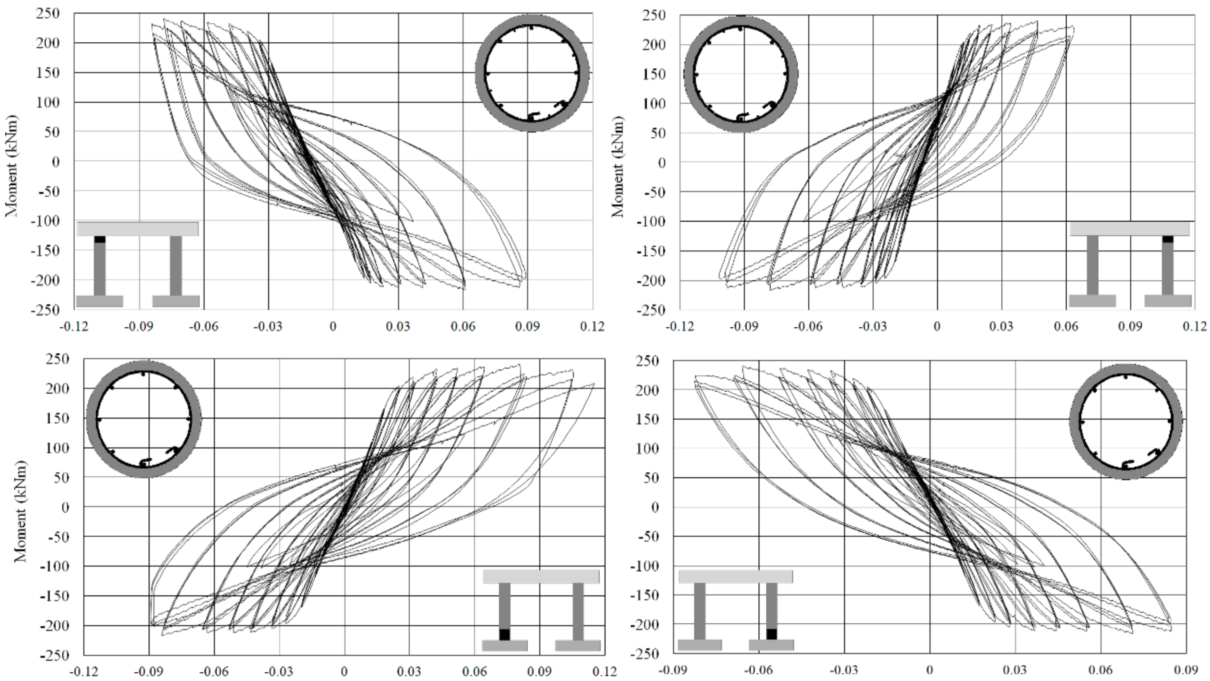


Figure 11. Moment-Curvature plots: (Top) top grouted ducts connections (Bottom) bottom member socket

6 CONCLUSIONS

The experimental testing showed promising results for using the grouted duct and member socket connection 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 CIP construction. The length of plastic hinges for the MSC measured to be similar to values prescribed in the codes, such as diameter of the column (500 mm). The unbonded length of the starter bars in GDC was very effective in distributing the strain over a longer length of the bar and to prevent excessive strength degradation which was obvious from having less concrete spalling at the top connections. The opening of a single gap or rocking mechanism was clear when the specimen was at the maximum push or pull. Overall, there was no damage to cap beam or footings. The precast 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. The specimen satisfied the criteria for operational performance levels shown in Table 1. However, the cost of repairing and downtime will be an issue for ABC High Damage. Therefore, the second phase of testing or ABCD Low Damage (2011-2014) is the evolution for a better seismic design which minimizes the cost of repair, in the same time eliminating downtime.

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