NEW CONNECTIONS FOR ENHANCING ROBUSTNESS OF PRECAST CONCRETE FRAME STRUCTURES

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ABSTRACT

The National Institute of Standards and Technology is continuing its research on the mitigation of disproportionate collapse in structural systems typical of U.S. building construction. As part of this research, two ten-story precast concrete moment frame buildings were designed and beam-column assemblies, representing portions of these buildings, were tested at full scale under an interior column removal scenario. The failure of the tested assemblies initiated prior to achieving the nominal flexural capacity of the beam sections and revealed potential vulnerabilities in the beam-to-column connections caused by eccentricities in the tensile force transfer path and reduced ductility of the reinforcement caused by welding. Motivated by the performance of these assemblies, several alternative connection concepts were developed in coordination with an industry review committee of PCI members. Five of these connection concepts were selected for experimental study and will be tested at five-eighths scale under flexural loads. While the beams and columns in the prototype structure would be subjected to a combination of axial, flexural, and torsional forces during a column removal scenario, developing connection details that possesses better flexural performance will represent a major step toward more robust precast concrete building systems. This paper describes the five proposed connection concepts and summarizes the ongoing connection testing program.

Keywords: Disproportionate collapse; Structural integrity; Structural robustness; Moment resisting connections; Column removal.

1 INTRODUCTION

For decades, United States (U.S.) codes and standards have included provisions intended to promote structural integrity in buildings with the objective of preventing local failures from spreading progressively and leading to the collapse of a disproportionately large portion of the structure. However, these codes and standards have generally either lacked specific provisions to achieve this goal or adopted prescriptive requirements that, while providing minimum levels of force continuity, may not ensure resistance to disproportionate collapse. Consequently, vulnerabilities to disproportionate collapse prevail in conventional U.S. construction practice.

Motivated by these considerations, the National Institute of Standards and Technology (NIST), through involvement in national committees of the American Society of Civil Engineers (ASCE), has played a key role in the ongoing development of a new standard for mitigation of disproportionate collapse, along with comprehensive guidelines for performing the alternative load path analyses needed to evaluate the potential for disproportionate collapse in buildings. To inform the development of these documents, NIST tested full-scale beam-to-column assemblies representing portions of steel moment frames,[1] reinforced concrete moment frames,[2] and precast concrete moment frames.[3,4] These tests revealed potential vulnerabilities in welded precast concrete moment connections and motivated the development of new connection details to improve the robustness of precast concrete frame structures.

This paper describes five new precast concrete moment connection concepts that were developed in collaboration with the Precast/Prestressed Concrete Institute (PCI) to provide improved structural robustness and the testing program currently underway at NIST to investigate their performance in flexure. Experimental data are limited on the performance of precast moment frames under loading consistent with disproportionate collapse; it is anticipated that experimental results from the new connection concepts considered in this study will contribute to future editions of the ASCE standard.

2 NIST PHASE I TESTS

NIST has been conducting a multi-year research program with a focus on evaluating the potential for disproportionate collapse in structural systems typical of U.S. building construction. As part of this research, structural designs were developed for ten-story prototype buildings using various structural systems including steel braced frames, reinforced concrete shear walls, steel and reinforced concrete moment frames, and precast concrete moment frames The designs for the two precast concrete buildings incorporated exterior moment frames as the lateral force resisting system in both orthogonal directions. The first building was designed assuming a moderate seismic hazard (SDC B) and the exterior moment frames were detailed as Ordinary Moment Frames (OMFs). The second building was designed assuming a higher seismic hazard (SDC D), and the exterior moment frames were detailed as Special Moment Frames (SMFs). The beams in the two frames were differentiated by their depths and longitudinal reinforcement. Beams of the OMF frames were deeper and had fewer longitudinal reinforcing bars than those of the SMF frames. In both designs, welded link plate connections were used to connect the beams and columns.

To evaluate the performance of the prototype structures under a column loss scenario, two-bay precast concrete moment frame assemblies, one OMF and one SMF, were designed and tested under a center column removal scenario.[3] The precast concrete frame assemblies suffered damage and failed at relatively small vertical displacements. This failure was caused by fracture of welded anchorage bars in the beams that were intended to transfer tension from the welded link plates to the beam reinforcement. These fractures initiated near the face of the column, prior to achieving the nominal flexural capacity of the beam sections. Based on test observations and detailed finite element analyses of the test specimens, the premature fracture of the anchorage bars was attributed to both the eccentricity in the transfer of tensile forces and a reduced ductility of the bars in the heat-affected zone of the welds.

3 CONNECTION CONCEPTS

Motivated by the limited ductility and premature failure of previously tested welded link plate connections,[3] new alternative connection concepts, designed for enhanced robustness, were developed in coordination with an industry review committee of PCI members. Of these new concepts, five were selected for experimental study.

3.1 MODIFIED LINK PLATE CONNECTION

The modified link plate connection is shown schematically in Figure 1. The purpose of the modified link plate connection was to incorporate only the minimal modifications to the prototype configuration necessary to reduce the eccentricity in the transfer of tensile forces and to improve connection ductility. The construction sequence is otherwise similar to the prototype structure, except that the connection interface is grouted. On site, once the beams are placed inside the pockets in the exterior columns, a moment connection is established by welding the steel link plates, protruding from the spandrel beams, to the steel channels embedded in the columns, using packing plates as needed for tolerances, and dry packing cementitious grout into the connection interface.

Moment transfer between the spandrel beam and column is accomplished through steel link plates that are welded to steel channels embedded in the spandrel beams and to steel channels embedded in the columns. To reduce the eccentricity of the tensile force transfer between the beam and column, anchorage reinforcement in the beam is welded to the same face of the embedded channel section as the link plate.

The intent of the modifications of this connection concept are to move inelastic deformations away from the welds and into a more reliable fusing mechanism. As has been observed in several test series,[3,5-7] welding reinforcement can lead to embrittlement and premature fracture in precast concrete frame connections when they undergo significant inelastic deformation. Instead, the ductile element in the modified connection is intended to be the link plate, which is reduced in width at the joint and provided a free length over which elongations due to joint rotation can be spread. The shape of the link-plate, shown schematically in Figure 1, resembles a non-standard tension coupon, and it is anticipated that it would have satisfactory performance in tension under a column removal scenario. Because the compression at the connection is carried through the grout, link plate buckling in compression, that could also occur over this free length, would likely not lead to significant reductions in connection strength under a column loss.

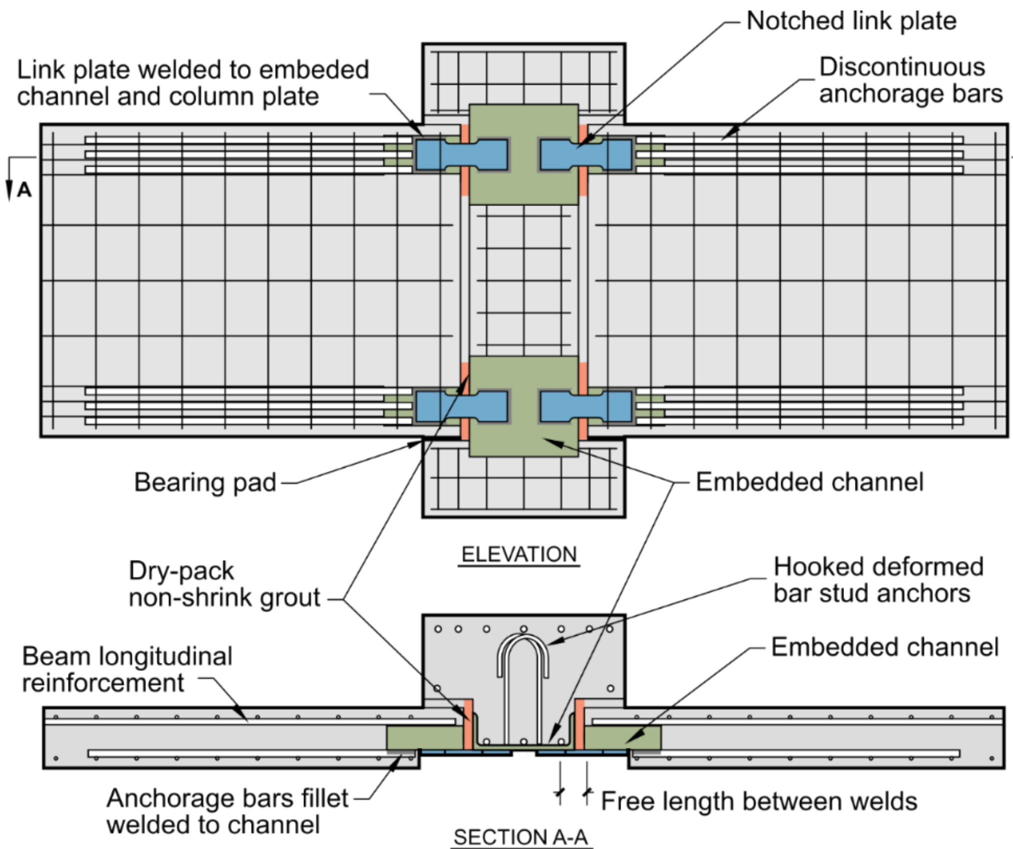


Figure 1. Schematic of modified link plate connection concept

3.2 THREADED ROD CONNECTION

The threaded rod connection concept is shown schematically in Figure 2. The threaded rod connection eliminates welding of critical connection components in the field. The beams and columns are connected via unbonded, high-strength threaded rods. The threaded rods run through corrugated ducts, are anchored by a nut that bears against an embedded rectangular box section in the beam and terminate at a standard steel coupler embedded in the column. Axial forces in the threaded rods are transferred to the beams through bearing of the rectangular box section on the beam concrete and through flexural bars connected to the rectangular box section opposite the threaded rod. These flexural bars are connected to the box section by welding.

On site, once the beams are placed inside the pockets in the exterior columns, a moment connection is established by threading the high-strength rods from the beam into couplers embedded in the columns. After the beam-to-column interface is packed with grout, the threaded rods are tightened to snug tight. The beam moment is transferred to the column by the coupling forces generated by simultaneous bearing of the concrete and elongation of the threaded rod.

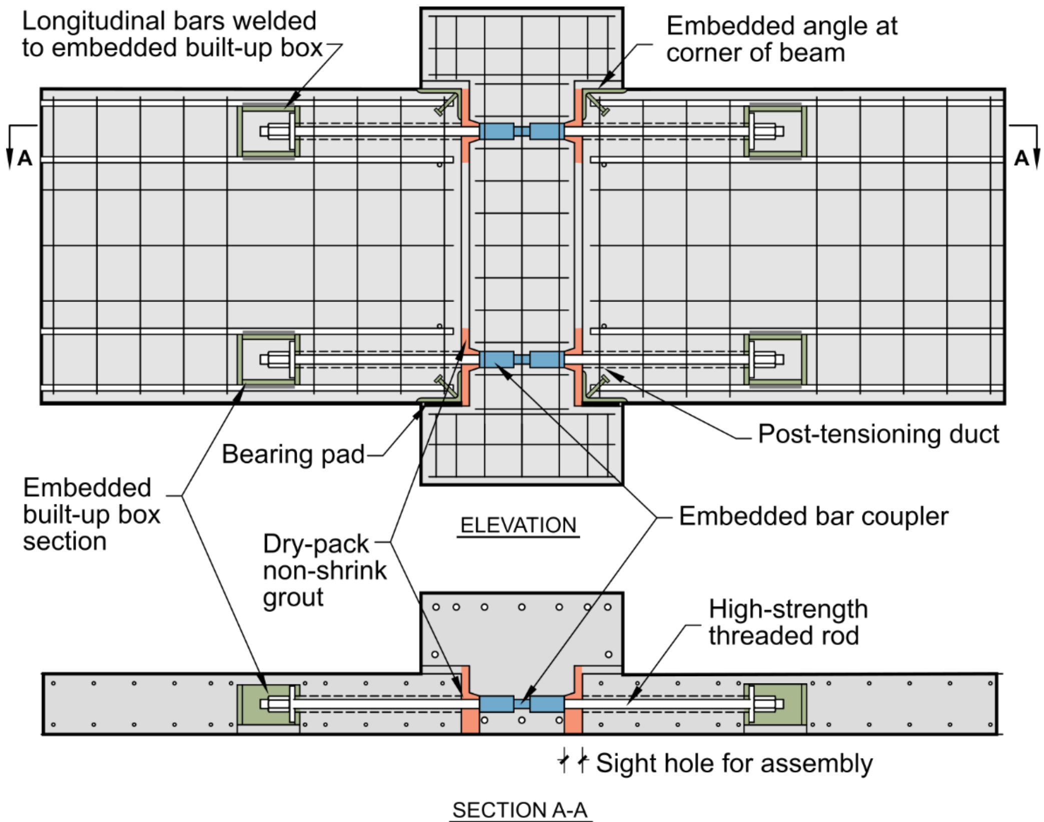


Figure 2. Schematic of threaded rod connection concept

Similar beam-to-column connection concepts have been investigated for seismic loads. French et al.[8,9] tested four beam-to-column connection subassemblies connected using threaded bars under cyclic loads. The threaded rods in these specimens were grouted in ducts, terminated in blockouts in the beam, and designed to yield and dissipate energy during a seismic event. The threaded bars in one specimen were post-tensioned, while in the other three, they were left unstressed. Palmieri et al.[10] also investigated a threaded bar connection concept. In that connection subassembly the post-tensioning rods were lightly tensioned prior to testing and unbonded through the column and beam ends, which were wider than the remainder of the beam to allow the rods to be passed from one beam end, through the column, to the adjacent beam. These tests all exhibited stable cyclic behavior up to interstory drift ratios of roughly 4 %, except in the case of the post-tensioned connection that suffered significant damage in the blockout region of the beam at an interstory drift ratio of 3.3 %. As expected, the connection subassembly with unbonded bars dissipated very little energy but had excellent deformation capacity, maintaining its strength under a monotonic displacement to an interstory drift ratio of 9 %.

In concept, the threaded rod assembly proposed here closely resembles the connection tested by Quiel et al.[11] also specifically intended for improving the performance of exterior precast frames with ledges under a column removal scenario. In that connection concept, the beam-to-column connection was completed by passing a threaded rod through two "connection corbels" and mating ducts within the column. Three tests were performed with two diameters and two types of ASTM A722 Type II reinforcement.[12] The specimens were tested under four-point loading and were fabricated so that each subassembly could be tested twice, utilizing both ends of the precast beams. In all three tests, the subassemblies behaved similarly; concentrated rotations occurred at the beam-to-column interface, the connections resisted the computed plastic strength of the spandrel beams in Main et al.[3], and eventually compression failure of the top connection corbel in the beam occurred. In the first test specimen, the bearing plate in the bottom (tension) connection corbel deformed inelastically and led to bearing failure of the corbel. In the later specimens, this undesirable failure mode was eliminated by increasing the thickness and extent of the bearing plate.

3.3 GROUTED BAR CONNECTION

The grouted bar connection concept is shown schematically in Figure 3. This connection concept avoids the use of welded link plates, in favor of grouted deformed bars in ducts. The beams and column are connected using several high-strength continuity bars that run through corrugated steel ducts in both the beams and columns. The bars are positioned on the centerline of the beam cross-section to further reduce eccentricities in the transfer of tensile forces.

On site, once the beams are placed inside the pockets in the exterior columns, a moment connection is established by passing the continuity bars from the beam on one side of the column into the adjacent beam, through ducts embedded in the column. Hand holes are provided at several locations along the beam to assist in sliding the deformed bars through the ducts. After sliding the continuity bars through the column, the ducts, hand holes, and beam-to-column interface are grouted.

The beam moment is transferred to the column by the coupling forces generated by compression of the grouted interface and elongation of the continuity bars. Tension forces from the continuity bars are transferred to the beam through bond. Previous bond tests of bars grouted in ducts have shown that a bonded length of roughly 10 bar diameters is sufficient to reach the ultimate capacity of the bar under monotonic loads.[13,14] The reinforcement is locally debonded near the grouted beam-to-column interface to promote distributed yielding and delay the fracture of the continuity bars, thus increasing the rotational capacity of the connection.

A similar connection was tested by Palmieri et al.[10] under cyclic loads. The beam longitudinal reinforcement was made continuous through the column by passing deformed bars through block-outs in the beam and corrugated ducts in the column. After assembly, the block-outs and ducts were grouted to complete the connection. The connection achieved its peak story shear at an interstory drift ratio of roughly 4 %, and the specimen maintained over 80 % of this peak strength up to an interstory drift ratio of roughly 6 %. The yielding frame concept was also included in the PRESSS Phase III test building[15] to serve as a baseline level of performance, to assess the performance of the prestressed frames. During testing, the headed continuity bars embedded in the exterior column slipped through the ducts within the column. This occurred after the bars first yielded in tension on a previous load cycle. This behavior was attributed to improperly placed grout and was presumed responsible for the low level of damage sustained by the frame during testing, since the loss of bond resulted in a simple connection at the beam end.[16] During a column removal scenario, it is unlikely that the structure would experience such a moment reversal, but this result does highlight the critical nature of the grouting procedure with this type of connection concept.

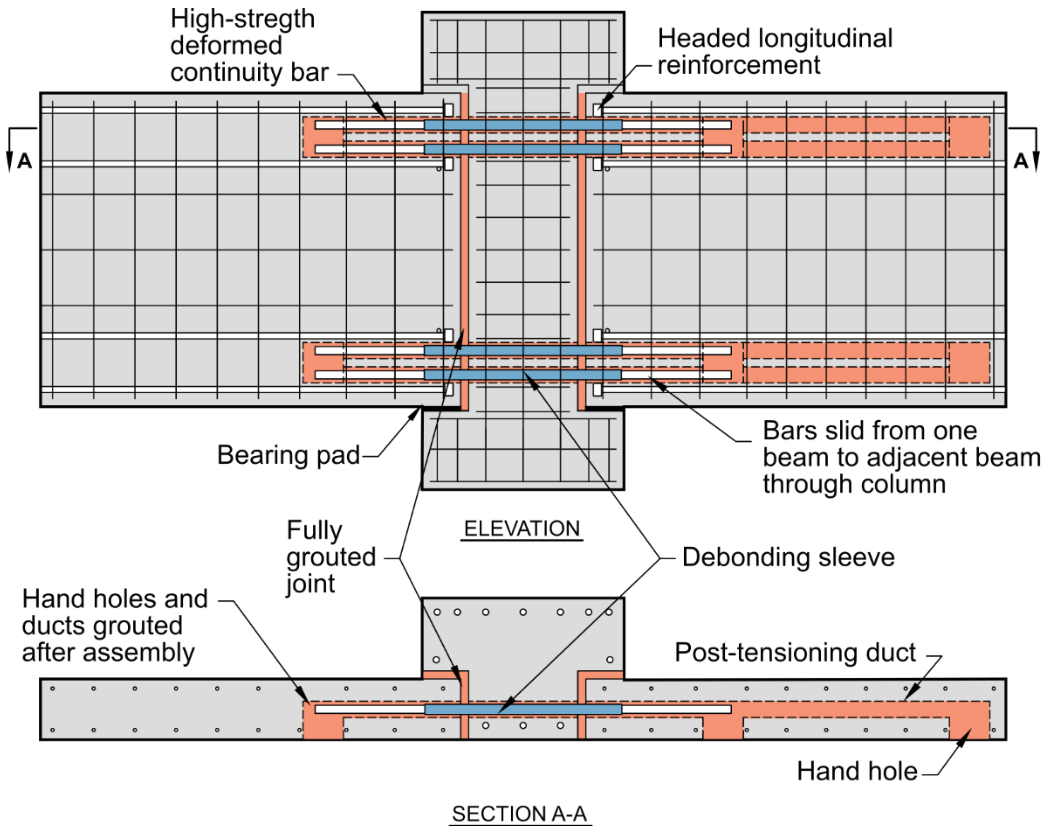


Figure 3. Schematic of grouted bar connection concept

3.4 BAR COUPLER CONNECTION

The bar coupler connection concept is shown schematically in Figure 4. This connection concept avoids the use of welded link plates, in favor of mechanically coupled reinforcing bars. The beams and column are connected using several continuity bars, which are discontinuous in the beams and are joined, using mechanical couplers, within a pocket in the precast column. The continuity bars are joined using loose "link bars," which consist of a short segment of bar with a coupler on either end. After assembly on-site, the column pockets and the beam-column interface are grouted to complete the connection.

Deformations of the frame are designed to be concentrated at the beam-to-column interface. In this connection concept, beam moment is transferred to the column by the coupling forces generated by simultaneous bearing of the concrete and elongation of the continuity bars. The continuity bars are positioned on the centerline of the beam cross-section, to eliminate eccentricities in the transfer of tensile forces. Tension forces from the continuity bars are transferred to the beam reinforcement through a non-contact splice.

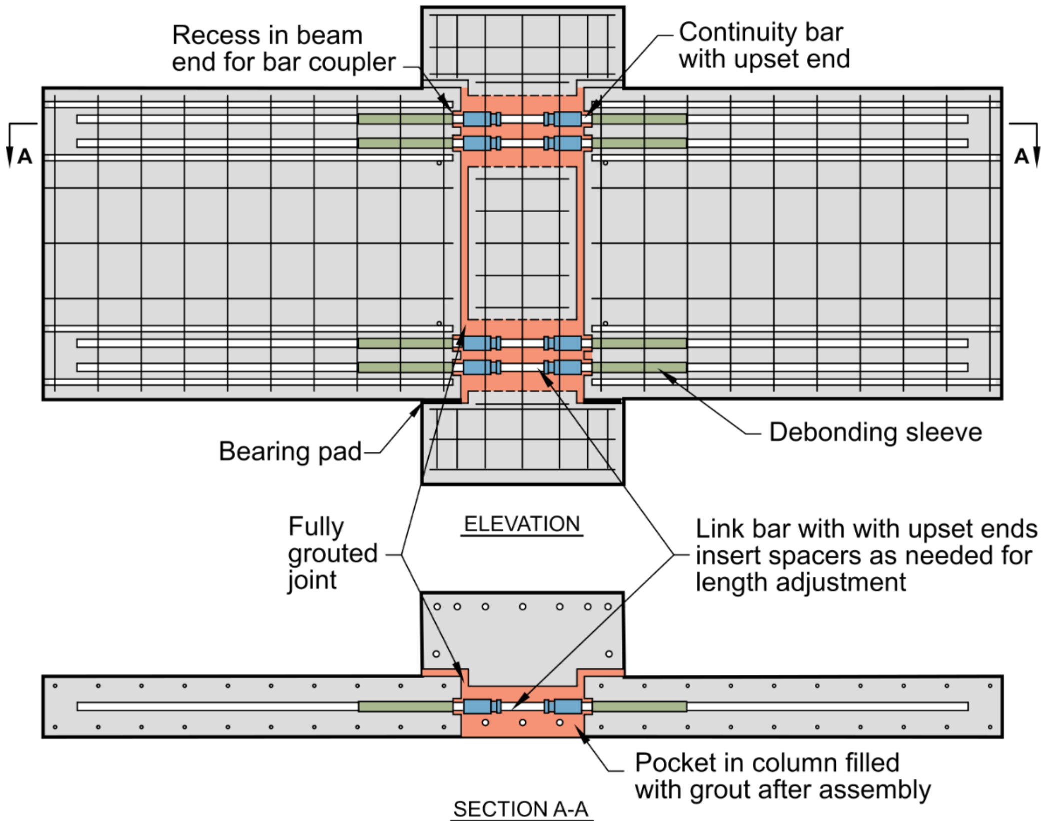


Figure 4. Schematic of bar coupler connection concept

The structural performance and ease of assembly of this connection concept is determined by the coupler that is used and the detailing of the bar connection at the beam-to-column interface. Some potential bar coupler candidates were tested by Rowell et al.[17] under uniaxial tensile loading at low, intermediate, and high strain rates (ranging from 0.001 sec^{-1} to 3.5 sec^{-1}). The test series compared the performance of coupled bars (using taper thread, upset ended, grouted sleeve, shear screw, and threaded bar couplers) to that of uncoupled bars subjected to the same loading protocol. Overall, all five of the bar coupler types performed well at low strain rates; however, only the threaded bar coupler performed satisfactorily at high strain rates. Of the tested coupler types, only the upset ended coupler and grouted splice sleeve can accommodate the variations in length necessary to complete the proposed connection concept, and the grouted splice sleeve would be difficult to assemble on-site because bars protruding from either the beam or the column would interfere with erection. Therefore, the upset ended coupler, shown in Figure 5, was selected for the proposed connection assembly. At the intermediate and high strain-rate tests, the bars joined using upset ended couplers maintained over 90 % of the dynamic ultimate strength of the control bar but had significant reductions in maximum strain and ductility (up to 40 %). One individual test achieved nearly the 100 % of the maximum strain of the control bars, but the reason for this large discrepancy between tests was not determined.

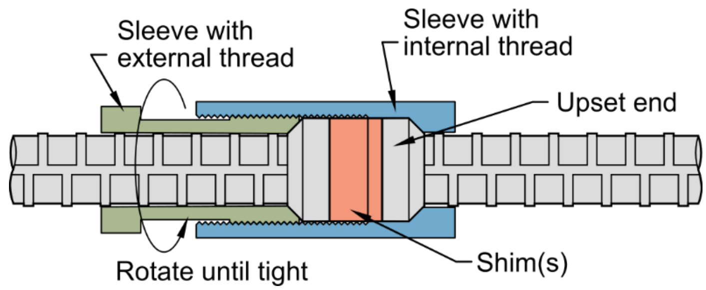


Figure 5. Schematic of upset ended bar coupler

Similar coupler connections have been used in recent tests of column-to-cap-beam and column-to-foundation bar coupler connections for accelerated bridge construction. Haber et al.[18] tested two precast column-to-foundation connections that utilized upset ended bar couplers in the hinge region. The column and foundation reinforcement were coupled using "transition bars," similar to the link bars in the proposed connection, which were either located at the column-to-foundation interface or half of one column diameter above the top face of the footing. Results of the tests showed that the connections utilizing mechanical bar splices exhibited comparable behavior to cast-in-place connections up to a 6 % drift ratio, and the location of the couplers within the column (at the interface or half a column diameter above the interface) had little influence on the performance.

Ameli et al.[19,20] tested precast column connections utilizing grouted splice sleeves and investigated the location of the coupler, either within the column or the foundation/cap beam, and the influence of debonding the longitudinal reinforcement at the coupler location. They found that placing the couplers within the body of the column led to an undesirable concentration of inelastic deformations at the ends of the couplers and debonding the reinforcement at the connection interface led to less distributed cracking up the height of the column and less damage at the joint between elements.

In the bar coupler connection concept in the present study, locating couplers within the beam would require bars protruding from the column, which would impede erection of the beams on-site. Therefore, the couplers were located within the column. The resulting link bars within the column are similar to the transition bars used in the columns tested by Haber et al.[19] To improve the connection performance and concentrate rotations at the connection interface, the bars will be locally debonded within the beam.

3.5 GROUTED BAR CONNECTION

The bolted connection concept is shown schematically in Figure 6. This connection concept is based on Figure B.7.2 in the PCI MNL-120-17 *Design Handbook 8th Ed.*[21] The bolted connection eliminates welding of critical connection components in the field. The beams and columns are connected via threaded anchor bolts. The anchor bolts run through an embedded HSS section in the beam and terminate at a standard steel coupler embedded in the column. In concept, the design is similar to that of the threaded rod connection; however, the anchor bolts are shorter and therefore a lower-strength ductile steel must be used to achieve the target connection rotation capacity.

On site, once the beams are placed inside the pockets in the exterior columns, a moment connection is established by threading the anchor bolts from the beam into couplers embedded in the columns. After the beam column interface is packed with grout, the anchor bolts are tightened to snug tight.

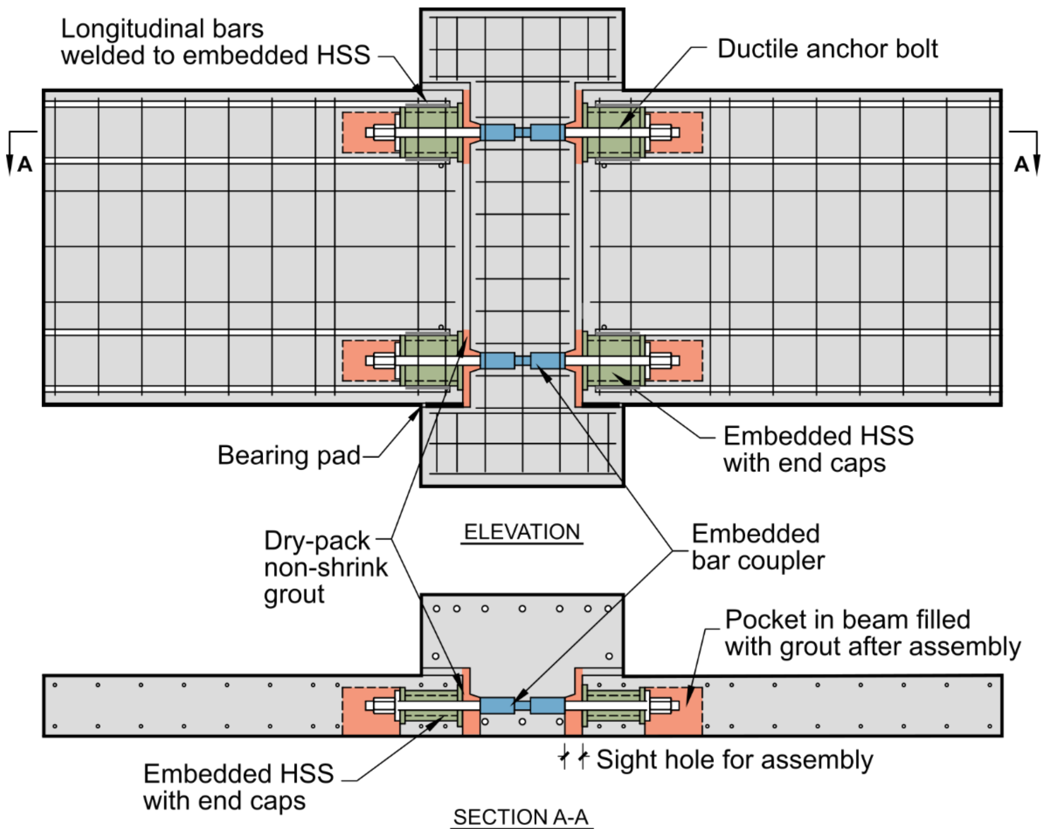


Figure 6. Schematic of bolted connection concept

Deformations of the frame are designed to be concentrated at the beam-to-column interface, and the anchor bolts are designed to be the ductile element in the connection. Forces from the anchor bolts are intended to be transferred to the beam longitudinal reinforcement through bearing of the anchor bolts on the embedded HSS section and through welds between the HSS section and the longitudinal reinforcement of the beams.

4 PHASE II TESTING PROGRAM

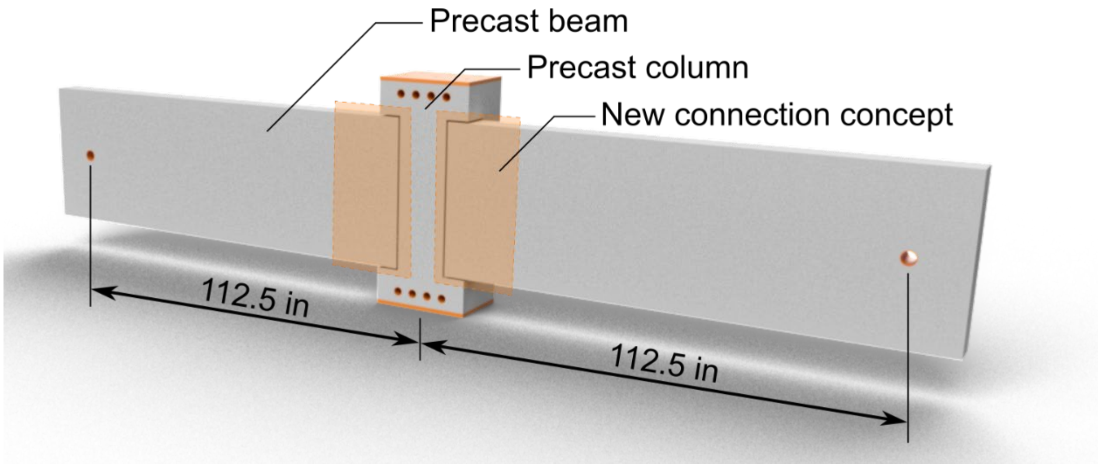
Results of high-fidelity computational modeling of the new connection details have suggested that they would provide improved strength and deformation capacity in comparison with the prototype configuration.[22] However, experimental data is needed to evaluate the potential performance benefits of the five new connection types. In the upcoming experiments, each connection will be tested in flexure at five-eighths scale. Each subassembly specimen consists of a central stub column connected to two spandrel beams, as shown in Figure 7a. The spandrel beams extend half the distance to the adjacent column in the scaled prototype building and are pinned at their ends, thus enforcing a zero-moment boundary condition at the midspan location, which corresponds approximately to the inflection point under a central column removal scenario (assuming a symmetric plastic mechanism).

Figure 7b shows the experimental setup that will be used in the test program. The test setup constrains the center column to move vertically in the plane of the frame, while permitting in-plane rotation. Vertical loads are applied by a hydraulic actuator aligned with the centerline of the spandrel beams, with horizontal movement of the actuator prevented by horizontal bracing. In this configuration, the connections and the beams will be subjected to flexural forces, while torsional and axial forces will be minimized. In a column loss scenario in an actual building, the beams and columns in the perimeter moment frame would be subjected to additional axial and torsional forces, depending on the configuration of the surrounding framing. However, the flexural performance of the beam-to-column connections will play a dominant role in preventing collapse. Therefore, providing alternative connection details with enhanced flexural performance will represent a major step toward developing precast concrete building systems with improved robustness against disproportionate collapse.

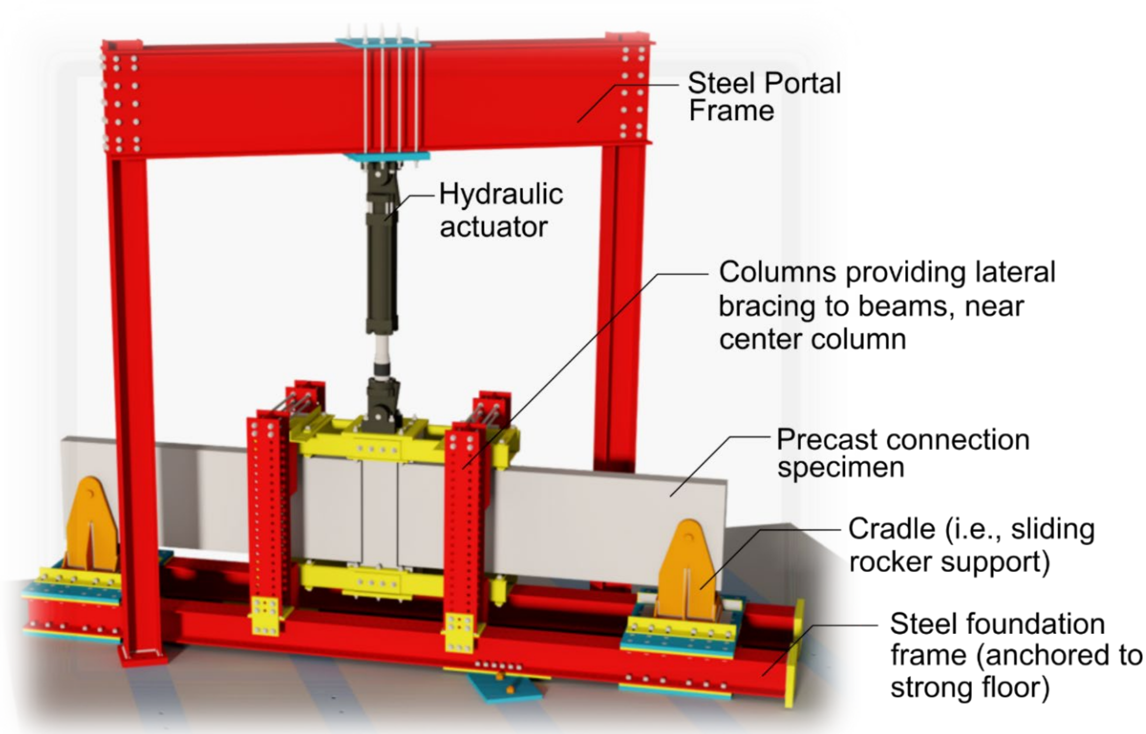
The five connection specimens have been designed, and fabrication will commence shortly. The design of the Phase 2 specimens followed the guidelines of UFC 4-023-03 *Design of Buildings to Resist Progressive Collapse*,[23] which are similar to those used in the forthcoming ASCE disproportionate collapse mitigation standard. The strength and detailing of the specimens was determined in accordance with ACI 318-19 *Building Code Requirements for Structural Concrete and Commentary*,[24] PCI MNL-120-17 *Design Handbook 8th Ed.*,[21] and AISC 360-16 *Specification for Structural Steel Buildings* (AISC, 2016).[27]

Unlike the prototype frame system, the alternative connection concepts rely on the development of concentrated rotations at the spandrel beam-to-column connections rather than relying on distributed curvature in the beams. The connections were designed for a target rotation of 0.04 rad. To achieve this level of ductility, the connection components (i.e., the link plates, threaded rods, or deformed bar reinforcement) were locally debonded from the concrete. Because the connection components are unbonded, the design provisions of ACI 318-19[24] that rely on strain compatibility could not be used to determine the strength of the connections. Instead, the connection components were sized according to procedures developed as part of the PRESSS program.[16,26] These design guidelines were developed for jointed precast connections with both prestressing strands and deformed bar reinforcement and provided a reasonable basis for the design of the Phase 2 precast concrete connection assemblies.

Additional detailing considerations were based on the results of recent tests of precast concrete assemblies under column removal[3,11] and previously tested precast concrete connections under cyclic loads.[10,16,18,27-29]



(a) Precast concrete specimen



(b) Experimental setup

Figure 7. (a) Five-eighths-scale precast concrete subassembly specimen, and (b) experimental setup used to apply vertical loading to center column, while restricting out-of-plane motions

The connection specimens were designed for the scaled, estimated moment demand under a column loss scenario for the SDC D prototype structure. In order to estimate the forces applied to the beam-to-column connection under a notional column removal, the loads acting on an exterior column in the prototype building were calculated under the extraordinary events load case from ASCE 7-16,[30] P = 1.2 D + 0.5 L. A multiplier for the gravity loads above the removed column, $\Omega_N = 1.2$, was computed using the dynamic amplification factors provided in UFC 4-023-03,[23] which are also incorporated in the ASCE disproportionate collapse mitigation standard currently in development. An idealized plastic beam mechanism was analyzed under the amplified gravity loads to estimate the required strength of the beam-to-column connections under sudden loss of a column in an exterior moment frame of the prototype structure. It was assumed in this analysis that plastic hinges form in the beams at the beam-to-column connections. Following the design of the connections under these idealized conditions, it is anticipated that a non-linear static analysis of the building would be conducted, with the as-designed connections, to determine whether the connections were sufficient to arrest disproportionate collapse. The estimated moment demand under an exterior column loss was roughly 36 % larger than that under the action of seismic loads but was still less than the estimated capacity of the beam sections in the Phase I specimens. This supports the conclusion that the performance of the Phase I assemblies was limited by the insufficient ductility of the welded connections.

The design procedure for the connections began with sizing the ductile fuse (i.e., the link plate, threaded rod, or deformed bar reinforcement) at the beam-to-column interface for the moment capacity necessary to resist collapse under a column removal scenario. The flexural strength of the beam-to-column connection was determined using the expected material strengths[31] and applicable resistance factors,[24,27] in accordance with UFC 4-023-03.[23] After the ductile fuse was sized, the remaining elements in the tensile load path (connection components intended to transfer tensile forces from the ductile fuse into the beams) and the longitudinal and shear reinforcement of the beam were designed for the probable strength of the connection, using the expected material strengths of the elements and applicable resistance factors. The probable strength of the connection was determined assuming a tensile stress in the ductile fuse of 1.25 times the expected yield stress of the material.

5 SUMMARY AND STATUS OF ONGOING WORK

To meet a previously identified need for alternative precast concrete moment connections to provide enhanced robustness, this paper described five new connections that were developed by NIST in coordination with an industry review committee of PCI members. The concept for each new connection type was summarized, and a schematic drawing was provided to illustrate the connection between the precast beams and the precast columns. Unlike the design of the prototype frame system,[3] which relied on distributed curvature in the beams, each of the five new alternative connection was designed to rely on the development of concentrated rotations at the spandrel beam-to-column connections. To achieve the required level of ductility, the connection components (i.e., the link plates, threaded rods, or deformed bar reinforcement) were designed as ductile fuses locally debonded from the concrete, and the beam longitudinal and shear reinforcement was designed for the probable strength of the connection.

The paper also described a testing program that is currently underway to evaluate the flexural performance of these five alternative connections at five-eighths scale. To date, the design and detailing of each connection type has been finalized, fabrication of the connection specimens will commence shortly, and fabrication of the experimental setup is nearly completed, with the portal frame and most of the experimental setup installed in-place on the strong floor of the NIST laboratory. Testing of the first connection is expected to commence in 2021.

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