POST-TENSIONED PRECAST CONCRETE COUPLING BEAM SYSTEMS

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ABSTRACT

This paper describes an analytical investigation on the nonlinear behavior of a new type of precast concrete coupling beam where coupling of reinforced concrete walls is achieved by post-tensioning the beams and the walls together at the floor and roof levels. The new system offers important advantages over conventional systems with monolithic cast-in-place reinforced concrete coupling beams, such as simpler detailing for the beams and the wall piers (no need for diagonal reinforcement crossing the beam-to-wall joints), reduced damage to the structure, and an ability to self-center, thus reducing the residual lateral displacements of the structure after a large earthquake. Steel top and seat angles are used at the beam-to-wall interfaces to provide energy dissipation. A parametric investigation is conducted on the nonlinear moment versus rotation behavior of floor-level coupling beam subassemblies under lateral loading. The parameters that are varied include beam and wall properties (e.g., beam depth, wall length), post-tensioning properties (e.g., post-tensioning steel area, initial stress), and top and seat angle properties (e.g., angle leg thickness). The results are used to determine how the behavior of the system can be controlled by design.

Introduction

Recent research on steel coupling beams has shown that post-tensioning is an effective method to couple reinforced concrete walls in seismic regions (Shen and Kurama 2002; Kurama and Shen 2004; Kurama et al. 2004, 2005; Shen et al. 2005). This paper extends the use of post-tensioning in coupled wall systems by investigating the seismic behavior of structures with precast concrete coupling beams. As an example, Fig. 1(a) shows an eight-story coupled wall system and Fig. 1(b) shows a post-tensioned coupling beam subassembly consisting of a precast concrete beam and the adjacent concrete wall regions at a floor level. High-strength multi-strand tendons run through the wall piers and the beam to provide the post-tensioning (PT) force. The PT tendons are unbonded over their entire length (by placing the tendons inside ungrouted ducts) and are anchored to the structure only at two locations at the outer ends of the wall piers. The beam-to-wall connection regions include steel top and seat angles. High-performance grout is used at the beam-to-wall interfaces for construction tolerances and for alignment purposes.

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Fig. 1(c) shows an idealized deformed shape of the subassembly as the coupled wall structure is displaced from left to right. The non-linear deformations of the beam occur primarily due to the opening of gaps at the beam ends. Under large displacements, a properly designed subassembly is expected to experience yielding in the top and seat angles, without significant damage to the wall piers or to the coupling beam. The angles, which are designed as sacrificial components during a large earthquake, provide redundancy in support of the beam, as well as energy dissipation. The angles also provide a part of the moment resistance of the system and prevent sliding of the beam against the walls (together with friction induced by post-tensioning). Bonded longitudinal mild steel reinforcement (not shown in Fig. 1) is used at the beam ends to transfer the angle forces into the beam. The mild steel reinforcement is not continuous across the beam-to-wall interfaces, and thus, does not contribute to the coupling resistance of the structure.

As gaps open at the beam ends, large compressive stresses due to post-tensioning are pushed toward the corners of the beam forming a diagonal compression strut. As shown in Fig. 1(d), it is through the formation of this compression strut that the coupling shear force $V_b$ is developed. The amount of coupling between the wall piers can be controlled by controlling the total PT force $P_b$ (which controls the total compression force in the beam, $C_b$), the tension and compression angle forces $T_a$ and $C_a$, the beam depth $h_b$, and the beam length $l_b$. To resist the large compression stresses, concrete confinement is provided in the contact regions at the beam-to-wall interfaces. Unbonding of the PT tendons has two important advantages: (1) it results in a uniform strain distribution in the tendons, thus, delaying the nonlinear straining (i.e., yielding) of the steel; and (2) it significantly reduces the amount of tensile stresses transferred to the concrete, thus reducing the amount of cracking in the wall piers and the coupling beam.

The advantages of precast concrete coupling beams over the post-tensioned steel beams investigated in previous research (see Shen and Kurama 2002) include: (1) simpler beam-to-wall
joint regions since no embedded steel plates are needed in the wall contact regions; (2) central location for the PT tendons, resulting in a reduced number of post-tensioning operations per floor level; (3) better fire and environmental protection for the PT tendons; (4) higher concrete-against-concrete friction resistance to prevent sliding shear failure at the beam ends; (5) simpler construction due to the use of high performance grout instead of steel shim plates at the beam-to-wall interfaces for construction tolerances and beam alignment; and (6) single trade construction.

Prototype Subassembly

The parametric investigation described in this paper is based on a prototype coupling beam subassembly as shown in Fig. 2. The prototype subassembly (referred to as Subassembly 1) has a wall pier length of $l_w=120$ in., uniform wall thickness of $t_w=15$ in., beam width of $b_b=15$ in., beam depth of $h_b=28$ in., and beam length of $l_b=90$ in., resulting in a beam length to depth aspect ratio of 3.21. The PT force is applied using a single tendon with twelve 0.6 in. diameter high-strength PT strands with a total area of $A_{bp}=2.6$ in$^2$. The strands are post-tensioned to an initial stress of $f_{bpi}=0.5f_{pu}$, where $f_{pu}=270$ ksi is the design ultimate strength of the strands. Four L8x8x3/4 top and seat angles are used at the beam-to-wall interfaces, each with a length equal to the beam width of 15 in. The gage length for the angle-to-wall connections (i.e., the length measured from the heel of the angle to the center of the innermost angle-to-wall connectors) is equal to $l_{gv}=5$ in. The design strength of unconfined concrete is $f'_c=6$ ksi, with an assumed ultimate strain of $\varepsilon_{cu}=0.003$. Closed hoops with cross-ties (No. 4 bars at 1.5 in. spacing) are provided in the beam-to-wall contact regions to confine the concrete. The strength of the beam confined concrete, estimated using a model developed by Mander et al. (1988), is equal to $f_{cc}=16.8$ ksi with an ultimate strain at crushing of $\varepsilon_{ccu}=0.047$.

![Fig. 2. Prototype subassembly](image)

Analytical Modeling

The analyses of the coupling beam subassemblies are conducted using the model in Fig. 3(a), with the DRAIN-2DX program (Prakash et al. 1993) as the analytical platform. The model includes fiber beam-column elements to represent the in-plane behavior of the wall piers and the coupling beam, zero-length spring elements to represent the top and seat angles, and truss elements to represent the unbonded PT tendon. Out-of-plane behavior of the subassembly is not considered. The post-tensioning of the structure is simulated by initial tensile forces in the truss elements, which are equilibrated by compressive forces in the fiber elements.

Each wall region is represented using two sets of fiber beam-column elements. The first set consists of elements, referred to as the “wall-height” elements, to model the axial-flexural and shear behavior of the wall region along its height. The second set of fiber elements, referred to as the “wall-contact” elements, is used to model the local compression behavior of the wall contact
regions to the left and right of the coupling beam. The Y-translational degrees-of-freedom (DOF) of Nodes B and C are kinematically constrained to Nodes A and D, respectively. The rotational and X-translational DOFs of Nodes B and C are not constrained. It is assumed that no slip occurs at the beam-to-wall interfaces. It is also assumed that the beam is designed with an adequate amount of transverse steel reinforcement to prevent diagonal tension failure and an adequate amount of bonded longitudinal mild steel reinforcement to transfer the angle forces into the beam. Note that the diagonal tension reinforcement requirements for unbonded post-tensioned precast concrete coupling beams are significantly less than the requirements for monolithic cast-in-place concrete beams as a result of the development of a diagonal compression strut along the span of the precast beam. This is described in detail in Weldon and Kurama (2005).

Each angle is modeled using two zero-length spring elements in the X- and Y-directions. The first spring element, which is referred to as the “horizontal angle element,” has a trilinear tension force versus deformation relationship in the horizontal (i.e., X) direction as shown in Fig. 3(b). The second spring element, referred to as the “vertical angle element,” uses a bilinear hysteretic model [Fig. 3(c)] in the vertical (i.e., Y) direction. The contribution of the vertical angle element is small with respect to the horizontal angle element, and thus, can be ignored. Both angle elements are connected to the same pair of nodes with identical coordinates located at the centroid of the bolt group connecting the angle horizontal leg to the beam and at the same elevation as the middle of the horizontal leg thickness. It is assumed that the angle-to-wall and angle-to-beam connections are properly designed and detailed for the maximum angle forces and deformations. Based on this assumption, one of the angle nodes is kinematically constrained to a wall-height element node at the same elevation and the other angle node is kinematically constrained to a corresponding beam node. More information on the modeling of the angles, as well as other aspects of the model (e.g., gap opening at the beam ends) and the verification of the model, can be found in Weldon and Kurama (2005) and Shen et al. (2005). Analytical models of multi-story wall systems can be constructed by combining the subassembly models for the floor and roof levels.

**Behavior Under Monotonic Loading**

Fig. 4 shows the expected moment versus rotation ($M_b-\theta_b$) behavior of the prototype coupling beam subassembly under monotonic loading. The left wall region is fixed at Node A.
(ignoring the deformations in the wall-height elements), and the right wall region at Node D is allowed to translate in the horizontal and vertical directions, but not allowed to rotate, resulting in displacements similar to the displaced shape with respect to the “reference line” in Fig. 1(c). A vertical force $V$ is applied at Node D in displacement control. The beam moment $M_b$ is equal to the coupling moment at the beam ends determined as $M_b = V l_b/2$ and the beam rotation $\theta_b$ is equal to the chord rotation, calculated as the relative vertical displacement between the two ends of the beam divided by the beam length.

Note that the subassembly analyses do not include the wall pier shear forces that develop in a multi-story structure, and thus, do not capture the axial forces that would be introduced into the coupling beams from the wall shear forces as the structure is displaced laterally. As discussed in Kurama and Shen (2004), these additional axial forces may be significant in the lower floor beams [2nd and 3rd floor beams, see Fig. 1(a)] in a multi-story structure; however, they are negligible for the coupling beams in the upper floor and roof levels. Thus, the results described below are more representative of the behavior of upper level beams in a multi-story structure.

As the prototype coupling beam subassembly is displaced, it goes through six response states as follows (see Fig. 4):

1. Decompression ($\Delta$ marker) – This state represents the initiation of gap opening at the beam-to-wall interfaces when the precompression due to post-tensioning is overcome by the applied lateral load. Before the decompression state, the PT force creates an initial lateral stiffness in the beam similar to the initial uncracked linear elastic stiffness of a monolithic cast-in-place reinforced concrete beam with the same dimensions. Gap opening at the ends of the precast beam results in a reduction in the lateral stiffness, allowing the system to soften and undergo large nonlinear rotations without significant damage (except in the angles and cover concrete at the beam corners). Note that the effect of gap opening on the subassembly stiffness is small until the gap extends over a significant portion of the beam depth.

2. Cover concrete crushing (◊ marker) – This state identifies the crushing of the cover concrete when the assumed ultimate strain of $\varepsilon_{cu}=0.003$ is reached in the unconfined concrete at the compression corners of the beam. The stiffness of the subassembly continues to decrease due to increased gap opening and deformation of the wall and beam concrete in compression.

3. Tension angle yielding (□ marker) – This state is reached when the first reduction occurs in the stiffness of the assumed tri-linear tension angle force versus deformation relationship in Fig. 3(b).

4. Tension angle strength (○ marker) – This state is reached when the second reduction occurs in the stiffness of the assumed tri-linear tension angle force versus deformation relationship in Fig. 3(b), representing the full plastic capacity of the tension angles. A large increase in the beam end moment resistance is observed between State 3 and State 4, after which the lateral stiffness of the structure is significantly reduced.

![Fig. 4. Behavior under monotonic loading](image)
(5) PT tendon yielding (X marker) – This state identifies the initiation of nonlinear straining (i.e., “yielding”) of the beam PT tendon. Note that the yielding of the beam PT tendon results in a reduction of prestress under cyclic loading, and thus, is not desirable. The use of unbonded tendons significantly delays the yielding of the PT steel.

(6) Confined concrete crushing (∨ marker) – This state identifies the desired failure mode of the subassembly due to the crushing of the confined concrete at the beam ends, resulting in a drop in the moment resistance of the structure. Note that other failure modes can also limit the nonlinear behavior of a subassembly, such as: (i) fracture of the top and seat angles; (ii) failure of the angle-to-beam or angle-to-wall connections; (iii) shear slip at the beam ends; (iv) diagonal tension failure of the beam; and (v) failure of the PT tendons or anchorages. These failure modes should be prevented by design, and thus, are not represented in Fig. 4.

Behavior Under Cyclic Loading

Fig. 5(a) shows the hysteretic moment versus rotation (M_b-θ_b) behavior of the prototype subassembly under cyclic loading. The thick curve represents the behavior under monotonic loading as described above. The subassembly is stable under large nonlinear cyclic rotations. The hysteresis loops show a self-centering capability (i.e., ability of the structure to return towards its undisplaced position upon unloading from a large nonlinear rotation) as compared to systems with monolithic cast-in-place reinforced concrete beams, while also providing a considerable amount of energy dissipation. The large self-centering capability of the subassembly indicates that the beam PT tendon provides a sufficient amount of restoring force to yield the tension angles back in compression and close the gaps at the beam ends. The total force in the PT tendon, P_b (normalized with A_bpfbpu) corresponding to the hysteretic behavior in Fig. 5(a) is shown in Fig. 5(b). Almost all of the initial prestress is maintained throughout the analysis since the yielding of the PT steel is prevented due to the use of unbonded strands.

Figs. 5(c) and 5(d) investigate the effect of the top and seat angles on the hysteretic behavior of the subassembly. The moment-rotation behavior in Fig. 5(c) is for a system with thicker angles having an L8x8x1 cross-section, which results in increased strength and energy dissipation with slightly reduced self-centering capability. Similarly, Fig. 5(d) shows the behavior of the prototype subassembly with the angles removed. The cyclic behavior of the subassembly without angles is very close to nonlinear-elastic, indicating that the angles provide most of the energy dissipation and that significant damage is limited to the angles only, which are replaceable after an earthquake. The angle size and post-tensioning force can be determined to achieve a good balance between the amount of energy dissipation and self-centering in the
structure. It is important that the angles provide a significant amount of energy dissipation; however, they should not prevent the closing of the gaps at the beam ends upon unloading. Further investigation in this area is beyond the scope of this paper.

**Parametric Investigation of Subassembly Behavior**

This section presents a parametric investigation on the nonlinear moment-rotation behavior of post-tensioned precast coupling beam subassemblies under monotonic loading. Selected structural properties of the prototype subassembly described previously (Subassembly 1) are varied, and then, a monotonic analysis of each subassembly is conducted. The results are used to determine how the behavior of the system can be controlled by design. The varied properties are: (1) thickness of the top and seat angles, $t_a$; (2) initial stress in the PT steel, $f_{bpi}$; (3) total area of the PT steel, $A_{bp}$; (4) $f_{bpi}$ and $A_{bp}$ varied simultaneously, with the total PT force kept constant; (5) wall length, $l_w$; (6) beam width, $b_b$; (7) beam depth, $h_b$; and (8) beam length, $l_b$.

The subassembly moment-rotation relationships from the parametric investigation are given in Figs. 6(a-h). For each of the eight parameters investigated, two variations from the original prototype subassembly are made, while keeping all other parameters constant.

The markers in Fig. 6 represent the states of behavior identified in Fig. 4. The beam end moment and chord rotation corresponding to the following selected response states are shown in Fig. 7 as functions of the varied parameters: (1) tension angle yielding (□ marker); (2) tension angle strength (○ marker); (3) PT tendon yielding (X marker); and (4) confined concrete crushing (△ marker). The other response states identified in Fig. 4 – decompression and cover concrete crushing – are not shown in Fig. 7.

Figs. 6(a) and 7(a) show the effect of the angle thickness, $t_a$ on the subassembly moment-rotation behavior. It is observed that, for the parameter range investigated, an increase in the angle thickness results in: (1) a large increase in the beam end moment resistance; (2) a small increase in the chord rotation at the PT tendon yielding state; and (3) a modest decrease in the chord rotation at the confined concrete crushing state.

Figs. 6(b) and 7(b) show the effect of the initial stress in the beam PT tendon, $f_{bpi}$ on the moment-rotation behavior. It is observed that, for the parameter range investigated, an increase in the PT steel stress results in: (1) a modest increase in the beam end moment resistance, without much change in the ultimate strength at confined concrete crushing; (2) a large decrease in the chord rotation at the PT tendon yielding state; and (3) a considerable decrease in the chord rotation at the confined concrete crushing state. Note that an initial PT steel stress that is too high can result in a loss of prestress under cyclic loading and in the fracture of the PT tendon.

Figs. 6(c) and 7(c) show the effect of the total area of the beam PT tendon, $A_{bp}$ on the moment-rotation behavior. It is observed that, for the parameter range investigated, an increase in the PT steel area results in: (1) a modest increase in the beam end moment resistance; (2) a small increase in the chord rotation at the PT tendon yielding state; and (3) a large decrease in the chord rotation at the confined concrete crushing state.

Note that the total beam PT force varies as the initial stress in the PT steel, $f_{bpi}$ is varied in Figs. 6(b) and 7(b) and as the area of the PT steel, $A_{bp}$ is varied in Figs. 6(c) and 7(c). In order to investigate this effect, the area $A_{bp}$ and initial stress $f_{bpi}$ of the PT tendon are varied simultaneously in Figs. 6(d) and 7(d) such that the initial PT force, $P_{bi}=A_{bp}f_{bpi}$ remains constant. It is observed that the beam end moment resistance up to the tension angle strength state is similar for the three subassemblies when $P_{bi}$ is kept constant.
The next four parameters in Figs. 6 and 7 investigate the beam and wall geometry. Figs. 6(e) and 7(e) show the effect of the wall length, \( l_w \). It is observed that, for the parameter range investigated, an increase in the wall length results in: (1) a small decrease in the beam end moment strength at the confined concrete crushing state, with almost no effect on the moment resistance up to the tension angle strength state; (2) a large increase in the chord rotation at the PT tendon yielding state; and (3) a modest increase in the chord rotation at the confined concrete crushing state. Note that the parametric subassemblies in Figs. 6(e) and 7(e) show no yielding of the PT tendon, except for Subassembly 1 for which PT tendon yielding occurs right before the confined concrete crushing state. The dashed line in Fig. 7(e) depicts the effect of the wall length on the rotation at the PT tendon yielding state, assuming that the crushing of the confined concrete is prevented.

Figs. 6(f) and 7(f) investigate the effect of the beam width, \( b_b \), on the behavior of the subassembly. For the parameter range investigated, an increase in the beam width results in: (1) a small increase in the beam end moment strength at the confined concrete crushing state, with almost no effect on the moment resistance up to the tension angle strength state; (2) a considerable increase in the chord rotation at the PT tendon yielding state; and (3) a large increase in the chord rotation at the confined concrete crushing state.

Figs. 6(g) and 7(g) show the effect of the coupling beam depth, \( h_b \), on the moment-rotation behavior. It is observed that, for the parameter range investigated, an increase in the beam depth results in: (1) a large increase in the beam end moment resistance; (2) a large decrease in the chord rotation at the PT tendon yielding state; and (3) a modest decrease in the chord rotation at the confined concrete crushing state.
Finally, Figs. 6(h) and 7(h) show the effect of the beam length, $l_b$ on the moment-rotation behavior. For the parameter range investigated, an increase in the beam length results in: (1) a small decrease in the beam end moment strength at the confined concrete crushing state, with almost no effect on the moment resistance up to the tension angle strength state; (2) a small increase in the beam chord rotation at the PT tendon yielding state; and (3) a modest decrease in the rotation at the confined concrete crushing state. Note that the effect of the beam length on the rotation at the PT tendon yielding state is smaller than the effect of the wall length, since the wall length represents a larger component of the total unbonded length of the tendon.

Fig. 7. Response states: (a) angle thickness; (b) initial PT stress; (c) PT area; (d) initial PT stress and PT area; (e) wall length; (f) beam width; (g) beam depth; (h) beam length

Summary and Conclusions

This paper investigates a new method to couple concrete structural walls using unbonded post-tensioned precast concrete beams. The investigation is based on monotonic and cyclic lateral load analyses of floor-level coupling beam subassemblies. The results show that the post-tensioning force creates an initial lateral stiffness in the beam similar to the uncracked linear elastic stiffness of a monolithic cast-in-place reinforced concrete coupling beam with the same
dimensions. Gap opening at the ends of the precast beam results in a reduction in the stiffness, allowing the system to soften and undergo large nonlinear rotations without significant damage.

The analysis results show that unbonded post-tensioned precast concrete coupling beams can be designed to provide stable levels of coupling over large reversed cyclic deformations, with considerable energy dissipation through the yielding of steel top and seat angles at the beam-to-wall interfaces. The post-tensioning force creates a restoring effect that closes the gaps and pulls the wall piers and the beams back towards their undisplaced position upon unloading (i.e., self-centering capability). Unbonding of the post-tensioning tendons ensures that the strains in the tendons remain small, thus delaying the yielding (i.e., nonlinear straining) of the post-tensioning steel and maintaining the initial prestress under cyclic loading.

The coupling moment resistance of a subassembly can be controlled by varying the beam depth, the top and seat angle strength, and the total post-tensioning force. The amount of energy dissipation is governed by the angle strength. The yielding of the post-tensioning tendons can be delayed by reducing the initial stress in the post-tensioning steel and the crushing of the confined concrete at the beam ends can be delayed by reducing the total post-tensioning force. Current work at the University of Notre Dame is using the results from this analytical investigation to conduct a large-scale experimental evaluation of post-tensioned precast concrete coupling beam subassemblies and to develop a simplified procedure to estimate the nonlinear moment-rotation behavior of the system. Analytical investigations on the seismic behavior and design of multi-story coupled wall structures are also being carried out.

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