LATERAL LOAD BEHAVIOR OF UNBONDED POST-TENSIONED HYBRID COUPLED WALLS

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Abstract

This paper describes an ongoing research program at the University of Notre Dame on the seismic behavior and design of unbonded post-tensioned hybrid coupled walls. Concrete walls are coupled using steel beams and unbonded post-tensioning, without embedding the beams into the walls. Analytical results indicate that unbonded post-tensioned steel coupling beams with stiffness and ultimate strength similar to embedded steel coupling beams can be designed to soften and undergo large nonlinear rotations of up to 7.5 percent without significant permanent residual rotations upon unloading. The nonlinear rotations in the beams occur primarily through gap opening along the beam-to-wall connections. Flange cover plates are used to delay the yielding of the beams in compression and top and bottom seat angles are used at the beam-to-wall connections to provide inelastic energy dissipation. An experimental investigation of unbonded post-tensioned hybrid coupled wall subassemblages is currently underway at the University of Notre Dame.

Introduction

A significant amount of research has been recently conducted on hybrid coupled wall systems which use embedded steel beams to couple reinforced concrete walls (Gong and Shahrooz 1997; Harries et al. 1993). The broad objective of the research described in this paper is to develop a new type of hybrid coupled wall system using unbonded post-tensioning, without embedding the beams into the walls. It is expected that unbonded post-tensioned hybrid coupled walls with lateral stiffness and ultimate strength similar to walls with embedded steel coupling beams can be designed to soften and undergo large nonlinear rotations without significant permanent residual rotations in the beams or the walls.

Fig. 1 shows a typical six-story coupled wall and Fig. 2 shows an unbonded post-tensioned hybrid coupled wall subassemblage at a floor level. The post-tensioning force is provided by multi-strand tendons which are placed on both sides of the beam web and which are not in contact with the beam. Inside the walls, the bond between the post-tensioning steel and the concrete is prevented by placing the steel inside oversize ducts which are not grouted. Thus, the post-tensioning steel is anchored to the coupled wall system only at two locations at the ends.
The beam-to-wall connection regions include top and bottom seat angles bolted to the beam flanges and to steel plates embedded inside the walls (using welded headed studs) as shown in Fig. 2(a). Spiral reinforcement is provided in the walls behind the embedded plates to confine the concrete. The purpose of the angles is to yield and provide energy dissipation during an earthquake. The angles also resist sliding of the beams along the beam-to-wall connections (together with friction resistance against sliding which develops as a result of the post-tensioning force).

Fig. 2(b) shows the deformed configuration of the subassemblage under lateral loads acting on the walls. In a properly designed and detailed beam-to-wall connection region, the desired behavior is yielding of the angles due to gap opening between the beam and the wall (Fig. 2(b)). The yielded angles can be replaced after the earthquake.

Gap opening along the beam-to-wall connections allows the beam to undergo large nonlinear rotations with little permanent residual rotations as discussed later. The post-tensioning force controls the length and width of the gaps. As the walls displace laterally, the tensile forces in the post-tensioning steel increase, resisting gap opening. Upon unloading, the post-tensioning steel provides a restoring force that tends to close the gaps. Unbonding of the post-tensioning steel delays the nonlinear straining (i.e., yielding) of the steel and reduces the amount of tensile stresses transferred to the wall concrete.

Analytical Model

This section describes an analytical model which was developed to conduct nonlinear static and nonlinear dynamic time-history analyses of multi-story unbonded post-tensioned hybrid coupled walls using the DRAIN-2DX Program. Fig. 3 shows the analytical model for a six-story coupled wall. Fiber beam-column elements are used to model the nonlinear hysteretic behavior of the beams, walls, and beam-to-wall connection angles, and truss elements are used to model the post-tensioning steel. The analytical model accounts for the axial-flexural interaction in the beams and walls, gap opening along the beam-to-wall connections, and hysteretic behavior of the angles, post-tensioning steel, beam steel, and wall concrete including crushing of concrete. The analytical model is described in detail in Shen and Kurama (2000).

Behavior of Unbonded Post-Tensioned Coupling Beams Under Monotonic Loading

This section describes an analytical investigation of the moment-rotation behavior of unbonded post-tensioned hybrid coupling beams under monotonic loading. The subassemblage shown in Fig. 4 is used for this purpose. The subassemblage includes two wall regions and one coupling beam as shown in Fig. 1. The wall length, \( l_w = 3 \text{ m} \) (10 ft) and the beam length, \( l_b = 3 \text{ m} \) (10 ft). The beam has a W18×234 cross-section and is post-tensioned to the wall using six post-tensioning tendons. Each tendon has an area of \( a_p \).
The outer pair of tendons (see Fig. 4(b)) are prestressed to 0.70\(f_{pu}\) and the inner pair of tendons are prestressed to 0.50\(f_{pu}\), where \(f_{pu} = 1860\) MPa (270 ksi) is the ultimate strength of the post-tensioning steel. Top and bottom angles with L8x8x3/4 cross-section are used in the beam-to-wall connections. Flange cover plates with thickness, \(t_c = 50\) mm (2 in.) are used near the connections to delay the yielding of the beam flanges in compression. The length of the cover plates is, \(l_c = 762\) mm (30 in.) to prevent yielding of the beam away from the connection regions. The yield strength of the beam steel is \(f_y = 362\) MPa (52.5 ksi).

Fig. 5(a) shows the moment-rotation behavior of the beam under a point load applied as in Fig. 4 to represent the conditions near the beam-to-wall connections during lateral loading of the coupled wall system. The beam is modeled as described above. The left wall region is assumed to be rigid and fixed. The right wall region can move along the horizontal and vertical directions but cannot rotate. The beam moment, \(M_b\), is equal to the moment at each beam end and the beam rotation, \(\theta_b\), is equal to the vertical displacement of the right beam end divided by the beam length (i.e., the chord rotation).

Fig. 5(a) shows that as the beam rotates, it goes through six states of response: (1) decompression (i.e., initiation of gap opening) along the beam-to-wall connections; (2) yielding of the flange cover plates in compression near the connections; (3) yielding of the angles in tension; (4) yielding of the beam flanges in compression near the connections; (5) yielding of the outer pair of post-tensioning tendons; and (6) yielding of the inner pair of post-tensioning tendons.

The first state in the moment-rotation response of the beam is decompression along the beam-to-wall connections. This state identifies the beginning of a nonlinear behavior of the beam due to gap opening. The stiffness of the beam before the decompression state is similar to the stiffness of an embedded beam with similar properties. Fig. 5(a) shows that the effect of gap opening on the stiffness of the beam is small until the gap opening extends over a significant portion of the beam depth, typically more than half the depth of the beam.

Fig. 5(a) shows that a significant reduction in the stiffness of the beam occurs between 0.3-0.7 percent rotation due to a combined effect of increased gap opening, yielding of the flange cover plates in compression, and yielding of the angles in tension. This combined state is called the softening state. The
beam moment and rotation at the softening state are referred to as $M_{\text{sof}}$ and $\theta_{\text{sof}}$. Estimation of $M_{\text{sof}}$ and $\theta_{\text{sof}}$ is discussed later.

The softening state is followed by yielding of the flanges in compression near the beam-to-wall connections. The beam moment and rotation corresponding to the yielding of the beam flanges in compression are referred to as $M_{\text{bfy}}$ and $\theta_{\text{bfy}}$.

The beam is designed such that yielding of the outer pair of post-tensioning tendons occurs at approximately 4.0 percent rotation. This state corresponds to the first yielding of the PT tendons and is called the PT-yielding state. The beam moment and rotation at the PT-yielding state are referred to as $M_{\text{pty}}$ and $\theta_{\text{pty}}$.

Yielding of the inner pair of post-tensioning tendons occurs at approximately 7.5 percent rotation. It is noted that the inner tendons yield after the outer tendons yield because the inner tendons are prestressed to a smaller value as described above. The elongations of the inner and outer tendons due to the rotation of the beam are the same due to symmetry. Fig. 5(a) shows that the moment resistance of the beam continues to increase after yielding of the inner post-tensioning tendons. However, as described later, this state corresponds to a significant reduction in prestress under cyclic loading. Thus, yielding of the inner post-tensioning tendons is considered to represent the rotation capacity of the beam and is called the ultimate state. The beam moment and rotation at the ultimate state are referred to as $M_{\text{ult}}$ and $\theta_{\text{ult}}$.

The dashed horizontal lines in Fig. 5(a) show the yield moment, $M_y$, and the plastic moment, $M_p$ of the W18x234 section used for the beam. The unbonded post-tensioned beam reaches the softening state at approximately $M_{\text{sof}} = 0.72M_y$ and the ultimate state at approximately $M_{\text{ult}} = 0.90M_p$ (or $M_{\text{ult}} = 1.1M_y$).

**Behavior of Unbonded Post-Tensioned Coupling Beams Under Cyclic Loading**

Fig. 5(b) shows the moment-rotation behavior of the beam under cyclic loading. The thick line represents the behavior under monotonic loading. The hysteresis loops show that the beam has a self-centering capability which means that upon unloading from a large nonlinear rotation, the beam returns towards its original position with little permanent residual rotation. The self-centering capability of the beam indicates that the nonlinear rotation occurs with little damage to the beam and that the post-tensioning force provides a sufficient amount of restoring force to close the gaps upon unloading.

Fig. 5(b) shows that there is a reduction in the softening moment, $M_{\text{sof}}$ under cyclic loading. The reduction in $M_{\text{sof}}$ occurs because of, to a large extent, yielding of the post-tensioning steel, and to a smaller extent, yielding of the angles in tension. This is described below.

Fig. 6 shows the post-tensioning force ratio, $\alpha_p$, plotted with respect to the beam rotation, $\theta_b$ during cyclic loading. The post-tensioning force ratio, $\alpha_p$, is equal to the total force in the post-tensioning steel divided by $A_{bg}f_y$, where $A_{bg}$ is the gross cross-section area of the beam including the cover plates and $f_y$ is the yield strength of the beam steel. Fig. 6 shows that there is a reduction in $\alpha_p$ during cyclic loading. The reduction in $\alpha_p$ occurs due to the yielding of the post-tensioning steel after the PT-yielding state is exceeded.

The post-tensioning force ratio at zero rotation, $\alpha_{p0}$ represents the prestress in the beam upon unloading and the initial post-tensioning force ratio, $\alpha_{pi}$ represents the initial prestress. Thus, the reduction in $\alpha_{p0}$ during cyclic loading is a measure of the reduction in prestress. Fig. 6 shows that $\alpha_{p0} = \alpha_{pi} = 0.22$ before
the loading of the beam and $\alpha_{p0} = \alpha_{pu} = 0.18$ after the loading of the beam to $\theta_{ult} = 7.5$ percent (corresponding to the ultimate state). The reduction in $\alpha_{p0}$, and thus, the reduction in prestress results in a reduction in $M_{sof}$ due to an earlier gap opening along the beam-to-wall connections. Because of unbonding, the maximum inelastic strains in the tendons remain small even during the large rotations of the beam, and thus, the self-centering capability of the beam is preserved (Fig. 5(b)).

As shown in Fig. 5(b), the beam is loaded to $\theta_b = \pm 1.875, 3.75, 5.625$ and 7.5 percent rotation in successive cycles. A reduction in prestress occurs during the cycles to $\theta_b = +5.625$ and 7.5 percent (Fig. 6) because the PT-yielding state is exceeded and the inelastic strains in the post-tensioning tendons increase during the successive cycles. This results in a reduction in $M_{sof}$ as shown in Fig. 5(b). It is noted that most of the reduction in prestress occurs during the loading of the beam in the positive direction (see Fig. 6) because the maximum tendon strains are not exceeded during the loading of the beam to the same rotation in the negative direction.

Fig. 6 shows that there is no reduction in prestress after the first cycle to $\theta_b = +1.875$ and 3.75 percent because the PT-yielding state is not exceeded (i.e., $\theta_b = 1.875$ and 3.75 percent are smaller than $\theta_{py} = 4.0$ percent). Fig. 5(b) shows that there is a reduction in $M_{sof}$ after the first cycle to $\theta_b = +1.875$ percent even though there is no reduction in prestress. This reduction occurs as a result of a degradation in the axial stiffness of the angle after yielding in tension during cyclic loading (Shen and Kurama 2000).

**Parametric Investigation of Beam Moment-Rotation Behavior**

This section describes a parametric investigation of the moment-rotation behavior of unbonded post-tensioned hybrid coupling beams under monotonic loading. For this purpose, selected structural properties of the subassemblage shown in Fig. 4 were varied. Then, a monotonic lateral load analysis of each subassemblage was conducted using the analytical model.

The structural properties which were varied include: (1) beam cross sectional properties (e.g., depth, flange thickness, etc.); (2) beam length; (3) wall length; (4) post-tensioning steel properties (i.e., area, initial stress, location, bars versus strands); (5) angle properties (i.e., length, cross-section, gage length); and (6) cover plate properties (i.e., length, thickness).

Figs. 7(a)-(d) show selected results from the parametric investigation. Each figure shows the moment rotation behavior of three parametric beams. Beam 1 is the same as the beam in the Fig. 4. The properties of Beams 2 and 3 are determined as described below.

In Fig. 7(a), the area of the post-tensioning steel, $a_p$ and the initial stress in the post-tensioning steel, $f_{pi}$ are both varied such that the initial post-tensioning force, $P_i = \sum a_p f_{pi}$ remains constant. The results indicate that $M_{sof}$ remains constant when $P_i$ is constant.
In Fig. 7(b), only a_p is varied. The results indicate that an increase in a_p results in: (1) an increase in M_{soft}, M_{pty}, and M_{ult}; and (2) a decrease in \( \theta_{pty} \).

In Fig. 7(c), only the wall length, l_w is varied. The effect of l_w on the subassembly shown in Fig. 4 is a change in the length of the post-tensioning tendons (note that the wall deformations are not modeled). The results indicate that an increase in l_w results in an increase in \( \theta_{pty} \) and \( \theta_{ult} \).

In Fig. 7(d), only f_{pi} is varied. The results indicate that an increase in f_{pi} results in: (1) an increase in M_{soft}; and (2) a decrease in \( \theta_{bty}, \theta_{pty}, \) and \( \theta_{ult} \).

**Trilinear Estimation of Beam Moment-Rotation Behavior**

This section describes a trilinear estimation for the nonlinear moment-rotation behavior of unbonded post-tensioned coupling beams under monotonic loading. The purpose of the trilinear estimation is to develop a simplified method for the seismic design and analysis of the beams. As an example, Fig. 8 shows a trilinear estimation for the smooth moment-rotation behavior in Fig. 5(a).

The trilinear moment-rotation behavior in Fig. 8 is identified by the softening state (at M_{soft}, \( \theta_{soft} \)), the PT-yielding state (at M_{pty}, \( \theta_{pty} \)), and the ultimate state (at M_{ult}, \( \theta_{ult} \)). Design equations were developed to estimate M_{soft}, \( \theta_{soft} \), M_{pty}, \( \theta_{pty} \), and M_{ult}, \( \theta_{ult} \) (Shen and Kurama 2000). The design equations were verified by comparing the estimated moment and rotation values with values determined using the analytical model as described below.

For example, Fig. 9 shows comparisons between the M_{pty} and M_{ult} values determined using the analytical model and the values estimated using the design equations for the parametric beams in Fig. 7(b)-(d). Similarly, Fig. 10 shows comparisons between the \( \theta_{pty} \) and \( \theta_{ult} \) values determined using the analytical model and the values estimated using the design equations for the parametric beams. The results indicate that the estimated moment and rotation values are close to the values determined using the analytical model for a wide range of parameters. Thus, the design equations can be used to conduct approximate, simplified analyses of unbonded post-tensioned coupling beams with different properties.
Behavior of Multi-Story Hybrid Coupled Walls under Monotonic Loading

This section describes the behavior of multi-story unbonded post-tensioned hybrid coupled walls under monotonic lateral loading (Shen and Kurama 2000). A six-story prototype coupled wall system is used which includes two walls and six coupling beams as shown in Fig. 1. The wall length, $l_w = 3$ m (10 ft) and the beam length, $l_b = 3$ m (10 ft). The coupling beams are the same as described in Figs. 4 and 5. The wall height, $h_w = 25$ m (82 ft) and the wall thickness, $t_w = 457$ mm (18 in.). Precast concrete walls post-tensioned vertically along horizontal joints are used (Kurama et al. 1999).

The solid line in Fig. 11(a) shows the base-shear-roof-drift relationship of the prototype coupled wall system under combined gravity and lateral loads. The distribution of the lateral loads over the height of the wall is triangular with the maximum load at the roof (applied from left to right). The coupled wall is modeled as described in Fig. 3.

Fig. 11(a) shows that the coupled wall system goes through six states of response under lateral loads: (1) decompression of the first beam along the height of the wall (beam decompression state in Fig. 5(a)); (2) decompression of the left wall at the base; (3) softening of the left wall at the base; (4) softening of the first beam along the height of the wall (beam softening state in Fig. 5(a)); (5) softening of the right wall at the base; and (6) failure of the right wall due to crushing of the confined concrete at the base.

The decompression and softening states for the beams are described in Fig. 5(a). In Fig. 11(a), only the first occurrence of the beam decompression and softening states over the height of the walls are shown. The decompression state for the walls corresponds to the initiation of a gap opening along the horizontal joint between the precast wall and the foundation (Kurama et al. 1999). The softening state for the walls is defined as the state at which the gap opening between the wall and the foundation reaches half the length of the wall. The failure state for the walls is defined as the state at which the confined concrete near the base of the wall crushes. Heavy spiral confinement is provided near the base of the walls to delay the crushing of the concrete (Kurama et al. 1999).

Fig. 11(b) shows the base-moment-roof-drift relationship of the prototype coupled wall system (solid line) and of the left and right walls in the coupled wall system (dashed lines). The coupled wall base moment is calculated as the moment from the applied lateral forces and is larger than the sum of the base moments for the left and right walls because of the coupling effect. The results indicate that the softening states for the left and right walls identify a significant reduction in the lateral stiffness of the walls.

The dashed line in Fig. 11(a) shows the sum of the base-shear-roof-drift relationship of the two walls without coupling (i.e., uncoupled system). The coupling of the walls results in a significant increase in the initial lateral stiffness and the base shear strength of the walls. The initial lateral stiffness of the coupled wall system is approximately 7 times the initial stiffness of the uncoupled system (with two walls). The degree of coupling between the walls (calculated as the difference in the base shear strength of the coupled and uncoupled systems divided by the base shear strength of the coupled system) at the right wall softening state (at a roof drift of 0.33 percent) is equal to, approximately, 65 percent.

The results for the prototype wall indicate that significant levels of coupling in hybrid wall systems can be developed by using unbonded post-tensioned beams (similar to the levels of coupling that can be
developed by using embedded beams). The degree of coupling can be controlled by changing the amount of post-tensioning force in the beams (Shen and Kurama 2000). In Fig. 11, the PT-yielding state is not reached for any of the six coupling beams. Thus, the coupling of the walls is achieved without causing significant damage in the beams during the lateral displacements of the walls.

**Current Work**

The current work for the research includes an experimental evaluation of the beam-to-wall subassemblage shown in Fig. 4 and an analytical investigation of the seismic behavior and design of multi-story coupled walls. A set of prototype coupled wall buildings are designed using the results of the analytical and experimental research. The seismic behavior of the prototype buildings is assessed based on nonlinear static and nonlinear dynamic time-history analyses.

**Summary and Conclusions**

An analytical model is developed to investigate the behavior of unbonded post-tensioned hybrid coupled walls under earthquakes. Simplified methods are developed for the design and analysis of the coupling beams. These methods are verified based on a parametric investigation of the beams using the analytical model. Lateral load analyses of multi-story walls indicate that significant levels of coupling in hybrid wall systems can be developed by using unbonded post-tensioned beams, without embedding the beams into the walls. The coupling of the walls is achieved without causing significant damage in the beams during large lateral displacements of the walls.

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**References**


