High Performance, Cost Effective Structural Systems for Seismic Hazard Mitigation

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Abstract

The post-earthquake investigations conducted in the recent years have shown serious deficiencies in certain structural as systems that are commonly used in building construction. The investigations also indicate to the engineering profession, as well as the general public, that the level of performance of structures should be substantially improved in future construction. For concrete structures, a ten-year research effort under the PRESSS program, has produced some new precast systems that have shown considerable promise in terms of performance and cost. This paper describes two of these systems, utilizing unbonded post-tensioned tendons, one for moment-frame construction and one for shear-wall construction. The systems have been extensively studied, both analytically and experimentally. The frame system has already been adopted in the construction of multi-story buildings in California.

Introduction

The Chi-Chi earthquake of September 21, 1999, revealed severe structural deficiencies of traditional reinforced concrete structures in resisting seismic ground shaking. Some of the damaged or collapsed buildings were relatively new and, presumably, were designed and constructed with modern technology. The prevalent structural system used is the moment-resisting frames either with or without brick infill. It is therefore opportune time for engineers to consider carefully some alternate structural systems that have shown special promise in recent studies. The purpose of this paper is to introduce two such systems that have been extensively studies under the PRESSS (Precast Seismic Structural Systems) program, which has been an intensive effort to develop high performance, cost effective concrete systems that can withstand strong earthquakes. Both systems utilize the post-tension concept, but with the tendons not bonded to the concrete. One system is for frame type construction and the other is for shear wall construction. They have the common characteristics of being able to (1) undergo large, pseudo-elastic, non-linear deformation without experiencing damage, (2) self center (that is, the structure can return to its original position even after considerable non-linear response) and (3) retain its initial lateral stiffness after a design level earthquake (defined later in the paper). The two systems are briefly described below; more detailed information can be found in Refs. (1), (2), (3), and (4).

Post-Tensioned Frame System

In this system, the precast beams and columns are joined together by post-tensioning of the embedded steel, which is left unbonded through the column and through portions of the beams. Figure 1 shows a beam-and-column subassemblage with a post-tensioned connection. The flexural behavior of connection is characterized by gap opening/closing at the beam-column interface upon loading/unloading. Unlike a cast-in-place connection, the inelastic deformations are concentrated in the connection region where a "crack" already exists between the beam and column. Furthermore, because the post-tensioning steel is unbonded, no additional flexural cracks will form in the beams in the connection region. The unbonded length can be selected to allow the lateral displacement demand of the design level ground motion to be reached without yielding of the post-tensioning steel. Consequently, the prestressing force can be maintained through the loading/unloading cycles. A wide gap is expected at the beam-column interface, and the associated concrete compression strains near the gap are likely to be large. Therefore, spiral reinforcement is necessary to confine the concrete.

The cyclic lateral load vs. deflection response of a post-tensioned subassemblage is shown schematically in Fig. 2. The response provides very limited hysteretic energy dissipation; but the structure has the unique ability to self center. The behavior can therefore be considered as essentially non-linear elastic. Models which can provide accurate prediction of the behavior of the connection have been developed and incorporated into a non-linear structural analysis program (Ref. 2). For design, a trilinear relationship, defined by three limit states, the linear state, the yield limit state, and the ultimate limit state, can be used to define the moment-rotation characteristics of the connection, as shown in Fig. 3. The linear limit (M_{ll}, 2_{ll}), established empirically from results of computer simulation of connection behavior, represents the state at which the moment-rotation response deviates noticeably from the initially linear behavior and begins to show significant softening. The connection reaches the yield limit (M_y, 2_y) when the stress in the post-tensioning steel reaches the proportional limit of the material. The ultimate limit (M_{ult}, 2_{ult}) is the state when the strain in the extreme fiber of the confined concrete reaches its ultimate strain, $,_{cu}$, which is defined by fracture of the spiral reinforcement. 2_{ult} is related to $,_{cu}$ and the failure length of the confined concrete adjacent to the beam-column interface, L_{cr}, shown in Fig. 1. Simple formulas have been developed to calculate the limit state moments and rotations. A design approach for unbonded post-tensioned

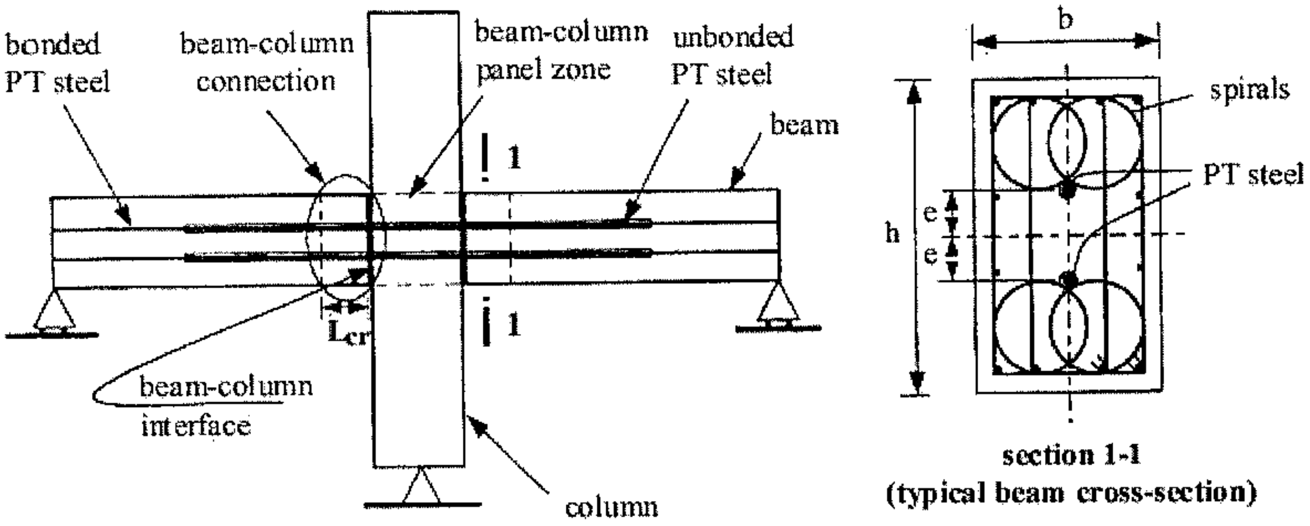


Fig.1 Unbonded Post-Tensioned Beam-and-Column Subassemblage

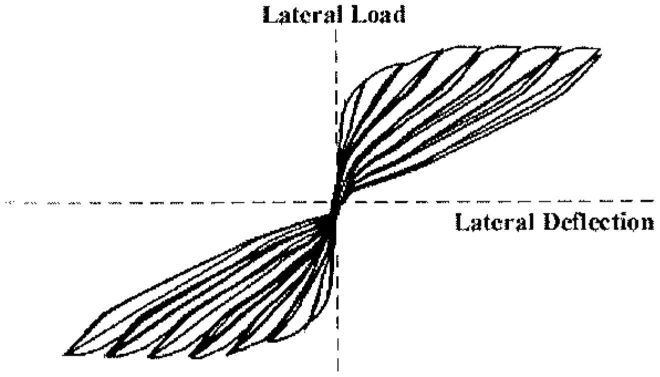


Fig.2 Lateral Load-Deflection Relationship of Beam-and-Column Subassemblage

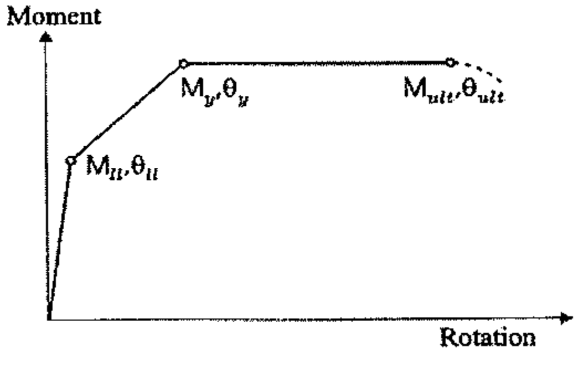


Fig.3 Trilinear Idealization of Moment-Rotation Relationship

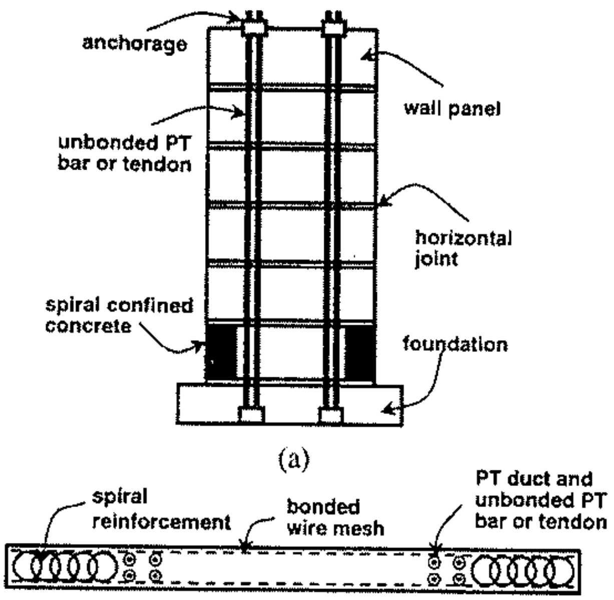
frames, based on the capacity design concept, has been proposed (Ref. 1). It considers two levels of earthquake ground motion, the design level (500-year return period) and the survival level (2500-year return period). The design level ground motion may cause only minor, easily repaired damages to both structural and nonstructural components, while survival level ground motion may cause damages to the structure that can not be repaired, but should not cause the structure to collapse. The NEHRP provisions (Ref. 5) can be followed for the first level design. The post-tensioned frame system is considered to be a special moment-resisting frame system with ductile connections. The response modification factor R and the deflection amplification factor C_d are, respectively, 8 and 5.5. The R factor reduces the first level base shear to the base shear to be used in structural design. The reduced base shear is called the structural design base shear. The general behavior of a well-designed post-tensioned frame is essentially controlled by the moment-rotation behavior of its beam-column connections. Therefore, the behavior of the frame can also be idealized using a trilinear base shear vs. roof displacement relationship by adapting the trilinear idealization of the moment-rotation relationship of the connections. Thus, in frame design, there are three sets of limit state criteria to be satisfied, which are associated with limit state of linear response $)_{ll}$, the limit state of yielding $)_y$, and the ultimate limit state, $)_{ult}$, defined by connections reaching the rotation 2_{ult}. The limit states lateral loads and displacements can be established, using the limit states moments and rotations of the connections and certain modifying factors (Ref. 1). The limit states design criteria for frames are as follows:

1. The structure should respond linearly, $) <)_{ll}$, at the structural design level base shear.
2. At the first (design) level earthquake, the structure's response should be within the non-linear elastic range, with the lateral displacement less than the yield limit displacement, that is $) <)_y$.
3. At the second (survival) level earthquake, the goal is to prevent excessive damage or collapse, that is, the displacement should be less than $)_{ult}$.

The procedure outlined above has been used in designing several prototype frames for both high and moderate seismicity zones. The response of these frames has been studied by performing pushover and time-history analyses, the latter with both natural and artificially generated (spectrum compatible) ground motion records. In these studies, for the high seismicity zone, the peak ground acceleration of the design level earthquake is set at 0.4g and that of the survival level earthquake at 1.0g. The earthquake records are scaled so that their peak ground accelerations are equal to either 0.4g or 1.0g. The performance of the frames at these two levels of ground motion is generally in line with the expected performance and the design goals are consistently met.

Post-Tensioned Wall System

An unbonded post-tensioned wall is constructed by post-tensioning wall panels across horizontal joints at the floor levels using post-tensioning steel, which is not bonded to the concrete (Fig. 4). Dry pack or grout may be used between the



(a)

(b)

Fig.4 Unbonded Post-Tensioned Wall: (a) Elevation and (b) Cross Section Near Base (Enlarged) panels for alignment and for construction tolerance. The behavior of the wall is very different from that of a cast-in-place wall. The lateral load resistance is provided by the post-tensioning steel (bars or tendons), located inside ducts which are not grouted. Spiral reinforcing steel is used to confine the concrete in the wall panel near the base of the wall. Wire mesh is used as bonded reinforcement in the panels.

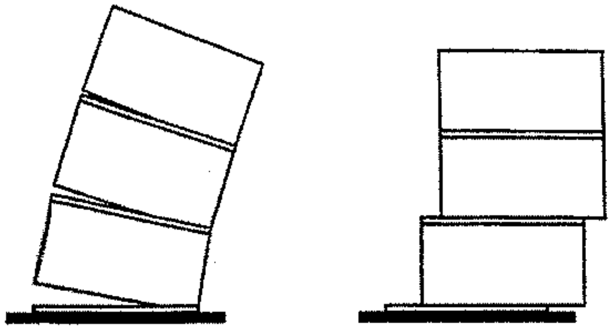

(a) (b)

Fig.5 Behavior Along Horizontal Joints:
(a) Gap Opening and (b) Shear Slip

The behavior of an unbonded post-tensioned wall under lateral load is governed by the behavior along the horizontal joints. Figure 5 shows the two types of behavior that can occur along the joints, namely, gap opening and shear slip. In the case of gap opening, the post-tensioning force and axial force due to gravity load provide a restoring force that tends to close the gaps upon unloading. In the case of shear slip, however, there is no restoring force to reverse the slip. Shear slip should, therefore, be prevented by proper design and detailing of the wall (Refs. 3 and 4).

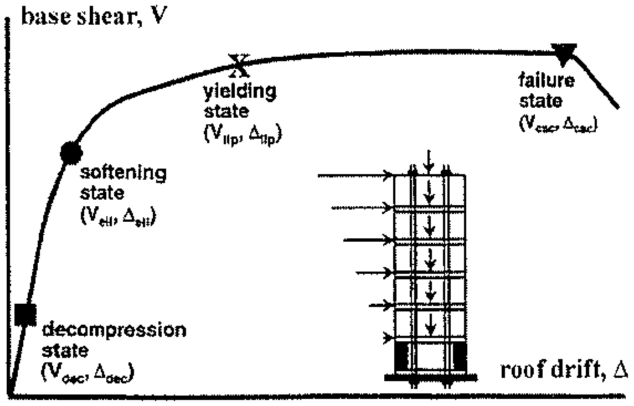


Fig.6 Base Shear vs. Roof Drift Relationship
of Unbonded Post-Tensioned Wall

The base shear (equal to the sum of all the lateral loads) vs. roof drift,), relationship of the wall is shown in Fig. 6. This relationship can be established by considering the axial-flexural behavior (i.e. behavior under combined axial force and flexure) and gap opening. As the wall displaces, it goes through four limit states. The first is the decompression state, identified by V_{dec} and $)_{dec}$, which represents the initiation of gap opening along the horizontal joint between the wall and the foundation and is the beginning of non-linear behavior. However, the effect of this non-linear behavior on the lateral stiffness of the wall is small until the gap opening extends over a significant portion of the joint. The second is the softening state (solid circle in Fig. 6), which signifies the beginning of an appreciable reduction in the lateral stiffness of the wall due to gap opening and non-linear behavior of the concrete in compression. The reduction in the lateral stiffness of the wall occurs in a smooth and continuous manner. Therefore, an effective linear limit, denoted by V_{ell} and $)_{ell}$, is used to identify this state. The third limit state (identified by a X) is related to the beginning of yielding of the post-tensioning steel. A properly designed wall does not reach the yielding state (denoted by V_{llp} and $)_{llp}$) until a large non-linear drift has occurred. The final state is the failure or ultimate state (identified by a solid triangle) when axial-flexural failure of the wall occurs as a result of crushing of the spiral confined concrete (at V_{csc} and $)_{csc}$). Sufficient spiral reinforcement is provided in the wall panels such that the failure state is reached at a drift significantly larger than the drift at the yielding state.

The behavior of the wall under cyclic lateral load is illustrated in Fig. 7. Fig. 7(b) shows the loading/unloading behavior during a load cycle with a drift equal to $)_{llp}$. Fig. 7(c) shows the behavior during a subsequent cycle with a maximum drift between $)_{llp}$ and $)_{csc}$. The hysteresis loops indicate that the behavior of the wall is nearly non-linear elastic, characterized by loading and unloading curves that are very close to each other. This behavior results in a self-centering capability.

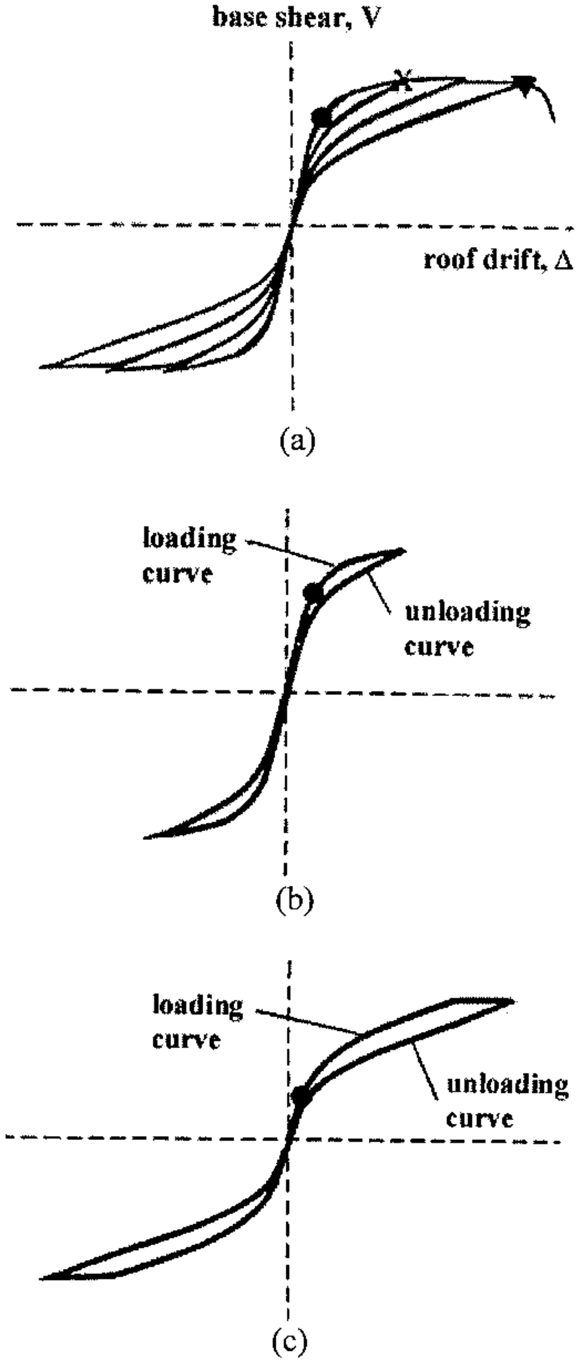


Fig.7 Hysteretic Behavior of Wall under Lateral Load: (a) Entire Behavior, (b) Loading Cycle just Reaching Yielding State, and (c) Loading Cycle Beyond Yielding State

A design approach for unbonded post-tensioned walls, similar to the approach described previously frame systems, has been developed. The design approach is a performance-based approach and considers again two levels of seismic ground motion: the design level and the survival level. The objectives of the design approach are (1) the building should experience very limited damage and can be immediately occupied after the design level ground motion and (2) the building should not collapse under the survival level ground motion. The first objective can be achieved if $)_{llp}$ is not exceeded under the design level ground motion. The wall behavior is nearly elastic, but non-linear, and the post-tensioning steel remains linear-elastic. The second objective, which is collapse prevention, can be achieved if the maximum roof drift of the wall is less than $)_{csc}$ under the survival level ground motion. Detailed design criteria to achieve these objectives are available (Refs. 3 and 4) and prototype walls have been designed using the design procedure and criteria. The dynamic response of the prototype walls has been studied in detail for various recorded ground motion inputs with scaled peak ground accelerations of 0.4g and 1.0g. The behavior of the walls is close to the behavior assumed in design. As observed previously in the post-tensioned frame system study, an unbonded post-tensioned wall has significantly smaller residual drift at the end of the ground motion than a cast-in-place wall.

A separate study conducted at Lehigh University has examined the behavior of unbonded post-tensioned walls with ductile vertical connections (Ref. 6).

Experimental Validation

A 0.6 scale five-story precast concrete building comprised of four ductile post-tensioned frames in one direction and a post-tensioned wall in the orthogonal direction was tested at the University of California, San Diego. The wall units were joined horizontally at the mid-height and vertically at the mid-width. The structure was tested separately in two directions (i.e. the frame direction and the wall direction). Seismic input levels equivalent to at least 50 percent higher than those required for UBS Seismic Zone 4 were applied and drift levels up to 4.5 percent were reached during the tests. The following is excerpted from a preliminary report of the investigation (Ref. 7).

1. At the end of the tests, damage to the building in the wall direction was minimal... . Cracking and minor crushing developed at the wall base at each end over a height of 150 mm above the foundation. These damages can be easily repaired.
2. Damage to the building in the frame direction was much less than could be expected for an equivalent cast-in-place structure, subjected to the same drifts. The damage was limited to minor spalling of cover concrete in the beams immediately adjacent to the columns... . Cracks in the beam-to-column joints due to shear were extremely small..... .
3. The residual drifts of the structure were very low. After the application of the design level (UBC) excitation, which produced a peak drift of 1.8 percent, the residual drift in the wall direction was only 0.06 percent. The low residual drift is a characteristic of the unbonded prestressing system... and is a significant advantage over conventional cast-in-place construction where very high residual drifts are possible. The negligible damage and the very low residual drift indicate that immediate occupancy of the building is possible.

Application

The post-tensioned frame system has been adopted recently in two construction projects: a 4-story building in Los Angeles and a 39-story building in San Francisco (Ref. 8). The 4-story building is structurally complete. The connections used in the building are the hybrid type, which include both bonded mild steel reinforcement and unbonded post-tensioning cables. The post-tensioning of all the connections at each floor level can be carried out in one operation, resulting in substantial savings in construction cost. It is reported that for the 39-story building a total of $4 million to $5 million can be saved by using the hybrid post-tensioned system.

Summary

The research, experimental testing, and application of unbonded post-tensioned precast concrete systems have been presented. Two systems have been described: a frame system and a wall system. Both have been developed specifically for seismic-resistant building construction. The load vs. deformation behavior of the two systems has been discussed and the proposed seismic design approaches (performance-based) have been outlined. The design approach considers two levels of seismic ground motion input and for each level a design goal is specified. The experimental studies indicate that the performance of the unbonded post-tensioned precast systems are considerably better than the cast-in-place systems from the points of view of ductility, damage control, and residual displacement. The frame system has already been adopted in two building projects and found to be cost effective. The new systems require much less on-site work than the cast-in-place systems and their use will help improve the quality of construction. These systems are recommended to the engineering profession as alternatives to some of the cast-in-place systems which performed poorly in the past earthquakes.

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