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Precast/Prestressed and Post-Tensioned Concrete

Post-Tensioned Shotcrete Shearwalls

An innovative approach to earthquake safety and concrete construction in buildings

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The seismic rehabilitation of a six-story medical office building (Fig. 1) in Berkeley, CA, presented unique challenges that were met using a variety of innovative solutions. The concrete frame building, with its distinctive exterior masonry cladding, was originally constructed in 1970. Like many buildings of similar age and construction, its nonductile frame would be highly vulnerable in the event of a large earthquake.

When the new owners approached us to develop a voluntary seismic improvement program, they outlined a key set of considerations that were to be integrated into the project. The building's close proximity to the Hayward Fault meant that the rehabilitation project had to be designed to resist large earthquake forces due to intense, near-fault ground shaking. Its proximity to the University of California-Berkeley, considered to be a likely tenant for the building, also meant that the design needed to comply with the university's stringent seismic safety criteria.

Because the seismic improvements were voluntary, the project had a fixed construction budget based on the economics of the office real estate market. A further limitation precluded any scheme that would result in exterior modifications due to the time required to undergo design review and obtain a planning permit from the city. Finally, and most difficult to achieve, the seismic bracing had to accommodate all existing mechanical systems in the building, such as plumbing pipes and heating, ventilation, and air conditioning (HVAC) ducts, to preserve the flexibility and value of the space.

Fig. 1: The voluntary seismic improvement of this six-story office building at 2850 Telegraph in Berkeley, CA, presented some interesting challenges

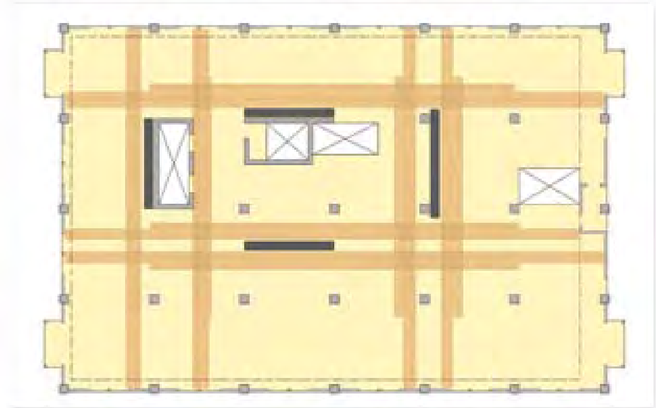

Fig. 2: Floor plan of the retrofitted structure showing new shear-walls in black and new carbon fiber collector elements in tan

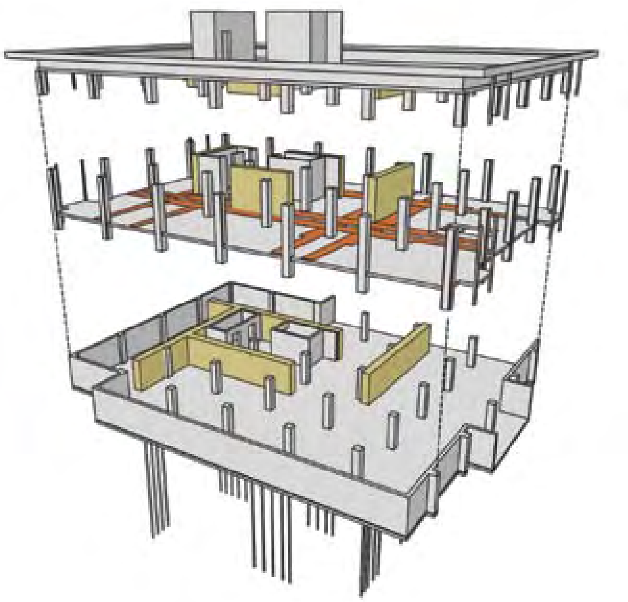

Fig. 3: Cutaway view of the building showing the shearwall and collector element arrangement at the 4th floor and the extended shearwall and micropile foundation arrangement at the basement level

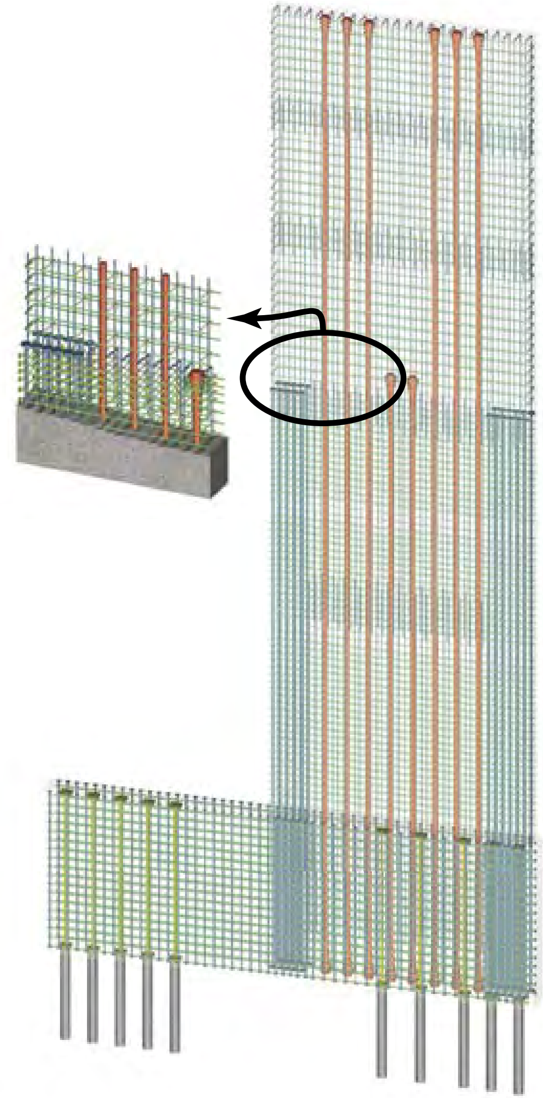

Fig. 4: Schematic illustration of shearwall reinforcement

THE SOLUTION

Often, restrictions can provide the impetus for innovation. By embracing the challenges that were presented, a cost-effective retrofit strategy that incorporated a system of hybrid concrete walls was developed. This system was integrated with the building layout to maximize the flexibility of the space. The primary challenge was to design for the large seismic forces (about 26% of the weight of the building) with a simple, yet effective, scheme.

As shown in Fig. 2, the challenge of reinforcing the lateral system for the building without affecting its outward appearance was met by installing two new shearwalls in each principal direction. To minimize disruption of the interior spaces, these walls were post-tensioned to provide the required strength in a very compact size. The walls blend the inherent advantages of reinforced concrete wall construction, including energy dissipation and stiffness, with the strength and elasticity provided by the vertically arranged post-tensioning tendons. More importantly, the unbonded post-tensioning tendons provide a self-centering capability to reduce seismic damage and minimize permanent deformations following an earthquake.

The compactness of the shearwalls and the large overturning forces required special attention to ensure that the foundations would effectively transfer seismic loads. To reduce these forces, the walls were extended as much as possible at the basement level, as shown in Fig. 3 and 4. High-strength micropiles were also installed

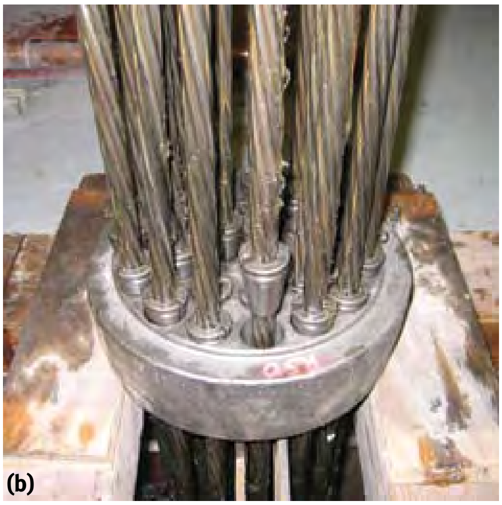

Fig. 5: Anchors for bundled tendons in the shearwalls: (a) fixed (dead) end anchors in the basement surrounded by spiral reinforcement to resist bursting stresses; and (b) fixed (live) end anchor at the roof level

Fig. 6: Due to equipment constraints in the building, each strand for the bundled tendons was individually installed through the floor opening cut in the floor slabs

to support the combined overturning demands from the walls above. In addition to their high axial load capacity, these foundation elements can be installed with minimal headroom and less disruption to the existing facility and foundations.

The total construction budget for the renovation was approximately $4 million. To help meet this target, formwork, labor, and material expenses were reduced by constructing the shearwalls using shotcrete rather than traditional cast-in-place concrete. This method is becoming increasingly widespread on the U.S. West Coast for constructing shearwalls in mid-rise buildings. On this project, an unusually high-strength mixture was used for the concrete walls to take better advantage of the resistance provided by the post-tensioning. The improved workability of the mixture resulting from the addition of a large proportion of slag cement also made it well suited for shotcreting. The post-tensioning also significantly reduced the overall quantity of vertical reinforcement, thereby facilitating shotcrete placement. In the boundary elements of the lower portions of the walls, large-diameter bars, mechanical splices, and headed reinforcement were also used to minimize congestion and ensure void-free concrete.

HYBRID POST-TENSIONED WALLS

The hybrid wall system consists of the four post-tensioned concrete walls clustered about the central stair and elevator cores shown in Fig. 2. The walls vary in thickness, measuring 34 in. (860 mm) thick from the foundation to the first floor slab, 30 in. (760 mm) thick up to the 4th floor, and 22 in. (560 mm) thick from the 4th floor to the roof. In the upper stories, the walls are typically 20 to 24 ft (6.1 to 7.3 m) in length. In the basement, the walls were interconnected and extended with outriggers to decrease overturning forces on the micropiles.

The walls were post-tensioned using 0.6 in. (15.2 mm) diameter seven-wire strand with an ultimate tensile strength of 270 ksi (1860 MPa). As shown in Fig. 4, each wall contains six bundled tendons, each consisting of 27 individually greased and sheathed strands extending the entire height of the wall. Two additional bundles extend from the foundation to the 4th floor, for a total of 216 strands at the base of each wall. To guard against yielding or rupturing the tendons as the walls deform inelastically, the strands were stressed to only 60% of their tensile strength. This provides a reserve of elastic elongation sufficient to accommodate the anticipated seismic deflections.

Still, the large number of strands resulted in a relatively high 1000 psi (6.9 MPa) post-tensioning stress in the wall. The tendons were provided with fixed multi-strand anchors at the base of the walls and stressing anchors at the upper end of the bundles. Figure 5 shows the installation of these tendon anchorages in the field. As shown in Fig. 6, the full-length tendons were installed through openings cut in the existing floor slabs prior to placing the mild-steel reinforcement. After the concrete was placed, the strands were individually stressed.

Capacity design principles were rigorously applied to ensure a controlled flexural mechanism at plastic hinge zones located at the base of the walls. The purpose of this approach is to prevent undesirable failure modes in

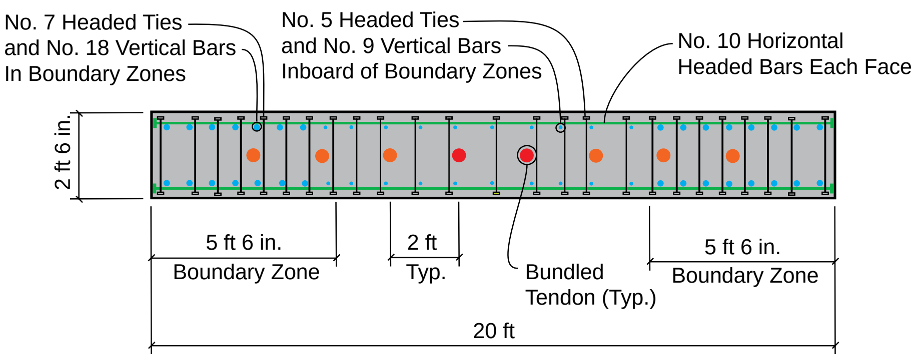

Fig. 7: Typical section in the plastic hinge region of a shearwall (1 in. = 25.4 mm; 1 ft = 0.3 m)

Fig. 8: Boundary element region of shearwall showing mechanically spliced No. 18 (No. 57) vertical bars, headed shear and crosstie reinforcement, and post-tensioning tendon

the wall, such as diagonal shear failure or crushing failure of the concrete, and reduce the sensitivity of the structural response to those characteristics. The philosophy of capacity design is concisely summed up by Paulay, "…if we don't like a shear mode of failure in the wall, then we must tell the wall what to do, so we use capacity design to make the wall have a shear capacity that is in excess of its flexural capacity."[1] In practical terms, the desirable flexural yielding of the wall at a well-defined plastic hinge location is achieved through several important detailing considerations:

- Horizontal reinforcement was designed for larger shear forces resulting from the effects of higher modes as well as material overstrength to ensure that these demands can be met elastically. The design equations for shear capacity described in FEMA 306[2] were used to proportion these critical regions. The plastic hinge regions between the 1st and 2nd floors were typically reinforced with No. 9 or No. 10 (No. 29 or No. 32) bars at 8 in. (200 mm) on center. Headed deformed bars were used for the horizontal reinforcement to maintain a stable response during large cyclic flexural deformations;
- Mechanical couplers were used to splice vertical reinforcement. This ensures that the full strength of the bars can be developed and eliminates the possibility of cyclic lap splice failure. The use of mechanical couplers also reduces congestion;
- Headed crossties were used at the boundary zones to provide the confinement necessary to maximize concrete compressive strength and prevent buckling of the No. 18 (No. 57) vertical boundary bars. In the plastic hinge regions between the 1st and 2nd floors, the transverse reinforcement consists of No. 7 (No. 22) headed crossties spaced at 8 in. (200 mm.) on center; and
- Outside the boundary zone, No. 5 (No. 16) headed crossties were used at each intersection of the vertical and horizontal reinforcement to enhance the concrete compressive strength through confinement and preclude diagonal compression failure of the concrete.

A typical section of the wall at the plastic hinge region is shown in Fig. 7. Figure 8 is a photo a typical wall boundary zone showing the post-tensioning tendons, the vertical and horizontal reinforcement, and the headed crossties.

DYNAMIC RESPONSE

The response of a hybrid post-tensioned wall is characterized by the superposition of two essential components of its behavior: an inelastic yielding component and an elastic restoring component.[3] The nonlinear elastic restoring component, idealized in Fig. 9(a), represents the response of the wall solely with post-tensioning and without the effect of the mild steel reinforcement. The yielding component, idealized in Fig. 9(b), represents the strength and energy dissipation response of the mild-steel reinforced wall without the effect of the post-tensioning. As seen in Fig. 9(c), the hysteresis loop of a hybrid wall containing mild steel and post-tensioning exhibits a flagpole shape resulting from the superposition of two component curves. The design on this project provides an elastic restoring component that is slightly larger than the yielding component. This means that slightly more than half the total resistance of the wall is derived from the post-tensioning. At this point, the hybrid system exhibits a favorable "rocking" response and a tendency to center itself following an earthquake.

DETAILING FOR CONSTRUCTIBILITY

Once the conceptual approach for the seismic retrofit was sufficiently developed, but prior to the completion of the design, we engaged the general contractor and the major subcontractors in

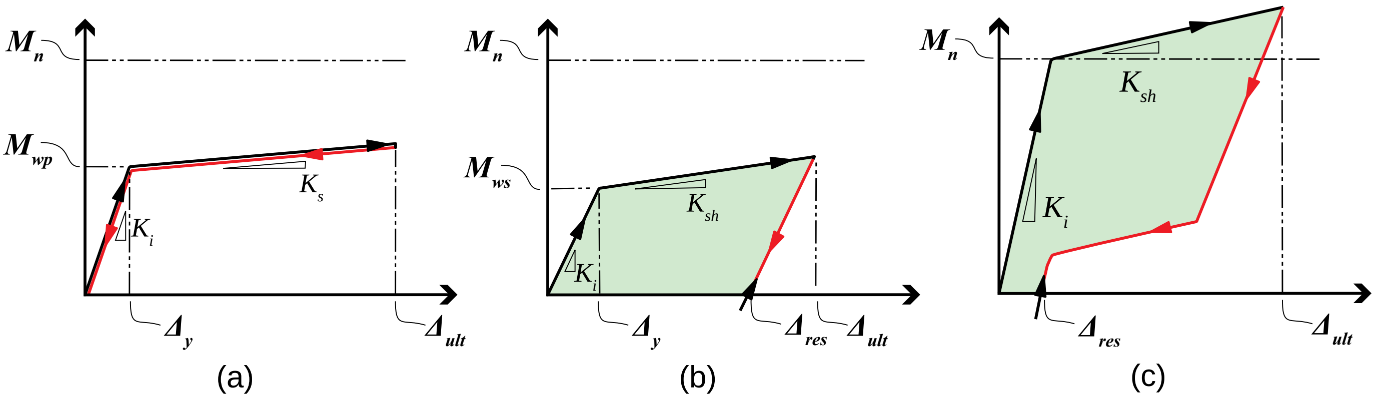

Fig. 9: Response of a hybrid post-tensioned wall: (a) post-tensioned wall response; (b) mild-steel reinforced wall response; and (c) combined response

extensive discussions to establish the key parameters that would affect the cost and complexity of construction. This involved all aspects of the work, including access, sequencing, and shoring considerations, from the beginning to the end of the process. Early collaboration allowed the design team to optimize the design and simplify the construction process on the basis of contractor input.

One outcome of this collaboration was that the reinforcement at the boundary zones was reconfigured to take advantage of larger diameter headed bars. The overall spacing was shifted from 6 in. (150 mm) to 8 in. (200 mm) to coordinate the placement of the horizontal and transverse reinforcement, simplifying assembly and minimizing labor. The reconfiguration also simplified the shotcrete work by reducing congestion. Ties and stirrups using bars larger than No. 5 (No. 16) are typically impractical due to large bend radii and limited anchorage provided by standard hooks. The plates anchoring each end of a No. 7 (No. 22) headed crosstie are about 2-1/2 in. (65 mm) square and 3/4 in. (19 mm) thick, allowing them to maintain effective confining pressure at the increased spacing.

Another outcome of the extensive collaboration with the contractors involved the post-tensioning tendons. In a typical application, multi-strand tendons would be preassembled, installed, and stressed as units in an effort to minimize their installation time. On this project, the initial concept was no different. The unbonded tendon bundles were initially conceived as being sheathed within a single duct; however, this was identified as a difficulty due to sequencing and access considerations. Specifically, there was no room to install a crane without major disruption to the adjacent streets or parking lot, mechanical equipment on the roof would be in the way of some tendons if they were dropped in from overhead, and there would be no way to slip a 90 ft (27 m) long, relatively rigid duct through the windows on the top floor and down through the narrow slots in the floor.

Both the general contractor and the post-tensioning supplier saw the flexibility of installing each bundled tendon's 27 strands individually as greatly outweighing the work it would take to install them pre-assembled. The strands could be individually sheathed (like slab tendons), loaded up elevators individually, dropped into place by crews without heavy equipment, and stressed individually with much smaller monostrand jacks. Although the construction was not as rapid as the initial preassembled scheme, the installation, especially the strand tensioning, proceeded more rapidly than expected. A two-person crew was able to tension all four walls in less than a week. In fact, all of the 864 strands in the walls were satisfactorily stressed to the design load.

Early collaboration with the contractors resulted in workable solutions that ultimately made this innovative project feasible and cost-effective. As a consequence, construction was streamlined and was completed with minimal field change-orders.

SHOTCRETE

A key feature of the shearwalls was a specialized concrete mixture developed to provide 8000 psi (55 MPa) compressive strength at 56 days, while allowing the concrete to be "shot" or pneumatically applied. Targeting a later point for establishing the design strength of the mixture provided the concrete supplier with the flexibility to replace portland cement with slag cement, primarily to improve workability. The resulting mixture incorporated a roughly 50% replacement of the portland cement with slag cement. In addition to the expected strength gain, the inclusion of slag resulted in good consolidation properties that made it optimal for shotcrete application. This substitution carried with it the added environmental benefits of reducing the embodied energy and the carbon footprint of the concrete.

Maintaining an aggressive construction schedule was a balancing act to ensure sufficient early strength to expedite the stressing operation and maximize the slag content of the mixture. Stressing was performed when the concrete had reached 4000 psi (27.5 MPa) compressive strength, which took about 7 days to develop. In an effort to accelerate the stressing operation, the contractor coordinated with the testing agency to have extra cylinders available to verify early strength gain characteristics and minimize the curing period prior to stressing. Ultimately, the concrete mixture resulted in, on average, a 56-day strength well in excess of the specified minimum, with some samples indicating a compressive strength above 10,000 psi (69 MPa). This truly high-performance shotcrete mixture sets a new standard for design and provides an opportunity to improve the quality and cost-effectiveness of concrete construction in a wide variety of applications.

CARBON FIBER COLLECTORS

To collect and transfer the inertial forces of the floors to the post-tensioned walls, carbon fiber-reinforced polymer (CFRP) strips were applied to the existing slabs on Levels 2 through 5 as shown in Fig. 2 and 10. The CFRP was applied directly to the top of the slab, so the existing HVAC ducts and most of the suspended ceilings could remain in place during the renovation. While comparable in cost, traditional under-slab collectors would have required the removal and replacement of the mechanical systems and ceilings at substantial additional expense.

The CFRP strips were applied in layers to achieve the required fiber area at each critical location. A strut-and-tie approach was used to design the collectors and minimize the extent of the CFRP overlay. The resulting CFRP overlay was sufficiently thin to be concealed below typical floor finishes with no additional fire protection. At the roof slab, the carbon fiber collectors were replaced with reinforced concrete collector beams that were anchored to the top of the slab and followed the perimeter of an existing mechanical penthouse.

MICROPILE FOUNDATIONS

High-capacity micropiles were installed to transfer the large over-turning forces generated by the post-tensioned walls into the supporting soil. While shear forces could be directly transferred and resisted with new grade beams and the existing basement slab, deep foundations were required to resist overturning effects. Due to the limited headroom in the basement—less than 10 ft (3 m)—micropiles proved to be the most efficient and practical deep foundation alternative.

The geotechnical design of the micropiles was established on the basis of performance criteria that specified an ultimate capacity of 600 kips (2670 kN) in tension and compression. To ensure adequate stiffness in the foundation system, the performance criteria limited pile extension to 3-1/2 in. (90 mm). To meet the performance criteria, the micropiles were extended to a depth of about 65 ft (20 m) with an effective bond diameter of 8-1/2 in. (220 mm), developing frictional resistance against the surrounding clayey soils to transmit both tensile and compressive forces. The micropiles rely on post-grouting, which involves the injection of pressurized grout along the bond zone after installation, to enhance the effective skin friction of the pile. The micropiles use a single 2-1/2 in. (65 mm) diameter reinforcing bar with an ultimate strength of 150 ksi (1000 MPa), and the top 25 ft (8 m) were cased in an 8 in. (200 mm) diameter steel pipe. The high-strength pile reinforcing bars were extended above grade and integrally cast into the basement walls.

Fig. 10: Carbon fiber collector elements installed on the floor slab and later concealed under the floor covering

A detailed testing program was carried out to verify that the design loads would be satisfactorily met and to ensure adequate quality control during installation.

At the onset of construction, the first two piles were tested to an ultimate tension capacity of 600 kips (2700 kN) to verify the design. For quality control, to ensure consistent construction practices, four additional piles were tested to the design load of 400 kips (1800 kN).

Shear forces from the new post-tensioned walls were transmitted to the supporting soil through a combination of friction and passive soil pressure. A broad base of the existing structure was engaged by doweling the long basement foundation walls to the adjacent basement slab, and connecting the walls to existing spread footings through a network of grade beams.

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PROJECT CREDITS

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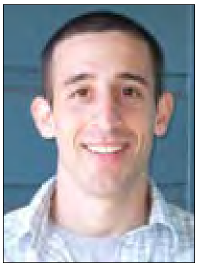

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