Externally applied post-tensioning system restores serviceability and strength

bij120
pr127

Innovative Rehabilitation of a Parking Structure

by Bijan O. Aalami and David T. Swanson

Describes the principal considerations in the evaluation, design, and construction that led to the successful rehabilitation of the multi-level Pier 39 concrete frame parking garage in San Francisco. Gradual deterioration of insufficiently protected heat-seal wrapped post tensioning had rendered the garage unfunctional and necessitated extensive shoring. Rehabilitation options, economic evaluations, structural computations, post-tensioning hardware, and construction highlights are discussed.

When severe slab cracking at the roof level and substantial water leakage from the roof were noted in the Pier 39 parking structure, the owner carried out further inspection. They found top-of-slab cracking adjacent and parallel to many beams (slab negative moment cracks) and beam deflections of up to 1½ in. (38 mm) in many instances. They decided to investigate further, a decision also prompted by several strands popping out of beam ends.

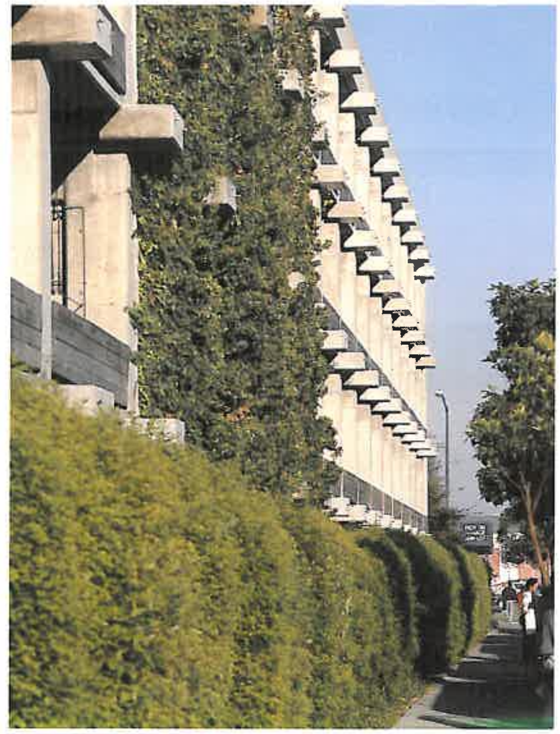
The structure, originally constructed in 1979, has five parking levels including the roof and a rectangular plan with overall dimensions of 370 x 196 ft. (118.9 x 63.0 m). At one corner there is a square recess of 65 x 170 ft (20.9 m x 54.6 m) (Fig. 1).

The structural gravity system consists of post-tensioned beams that frame into columns to form a parallel plane frame in the transverse direction. One-way post-tensioned floor slabs span the longitudinal direction. Shearwalls provide the lateral load resisting structural system.

The 36 in. (91.4 cm) deep beam spans are 65½ ft (21 m), while the 4½ in. (11.4 cm) slab spans are 18 ft (5.8 m). The beams contained seven 0.6 in. (15 mm) monostrand post-tensioning tendons, while the slab had 0.5 in. (12 mm) monostrand tendons at 26 in. (660 mm) on center.

The owner engaged an engineering firm to evaluate the structure. Holes were chipped at midspan of all ramp beams to determine the condition of the beam post-tensioning. All strands examined showed

Fig. 1 — (below, left) View of Pier 39 parking garage on Fisherman's Wharf in San Francisco.
Fig. 2 — (below, right) Prior to rehabilitation, deterioration of the structure had necessitated extensive shoring.



(above) Tendons encased in precast tubes extend from one end of the building to the other between deflectors; (right) end anchorage blocks on the elevation of the building; (bottom, left) a high point deflector at a transfer girder; (bottom, right) the ends of the precast square tubes and the low point deflector are encased in cast-in-place concrete for fire and corrosion protection.

some signs of corrosion and several strands had failed. Slab investigations also revealed severe corrosion with some failures at the roof level.

Based on the investigations of the structural consultant, 12 in. square (300 mm square) timber supports were placed at beam quarter points. The load from these supports was distributed to timber mats at ground level. This shoring resulted in a noticeable reduction in the available parking spaces in the garage (Fig. 2).

Rehabilitation options

The central issue to the rehabilitation of the Pier 39 parking structure, besides economy, was that a new system had to be built around or added to the existing structural members while the parking garage remained essentially operational.

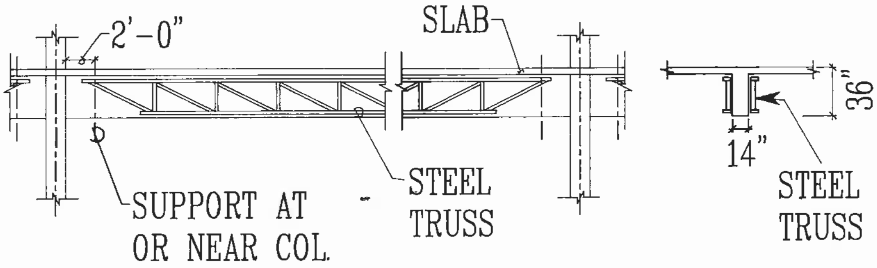
The beams, and to a lesser degree the slabs, were the primary targets of rehabilitation. As the columns were adequate to sustain the subject loading, a second criterion was set to use the existing framing system. Schemes were developed around adding new supporting members on each side of the existing post-tensioned beams to bear the weight of the building. New members could transfer their loads back to the existing beams near the column supports.

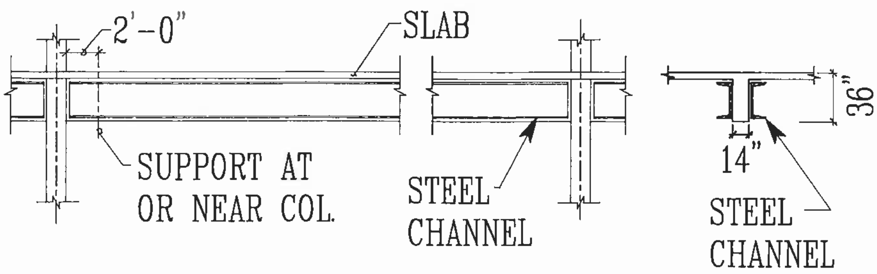
Two major options were reviewed in detail, one using steel and the other concrete. The steel option consisted of several types of tubular truss systems and a steel channel proposal. Providing post-tensioning to the bottom chord of the steel truss proposal was also examined. Basic features of two of the steel options are shown in Fig. 3 and 4.

The concrete alternative consisted of adding post-tensioned tendons on each side of the beams as illustrated schematically in Fig. 5.

Initial structural considerations, in addition to the cost effectiveness of the external post-tensioning, included: (1) whether to pick up the weight of the existing beams at single mid-span points or at one-third points; (2) whether to anchor the tendons at the neutral axis of the beam/slab or to impose an eccentricity at the anchorage points to improve the frame performance of the structure; and (3) resolution of practical details of placing and stressing the post-tensioning hardware, which was problematic due to space and accessibility limitations.

Preliminary studies showed that off-the-shelf hardware could not ef-





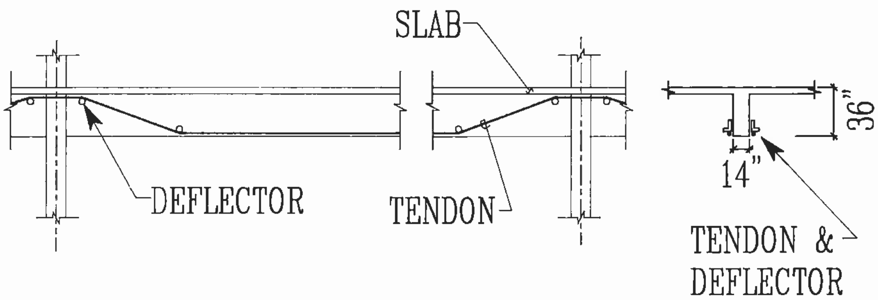


Fig. 3 — (top) Addition of steel trusses was one of the rehabilitation options.

Fig. 4 — (center) Steel channels or I-sections were also considered as alternatives.

Fig. 5 — (bottom) Externally applied post-tensioning proved to be the most economical alternative. Initial schemes were based on tendon deflectors at third points.

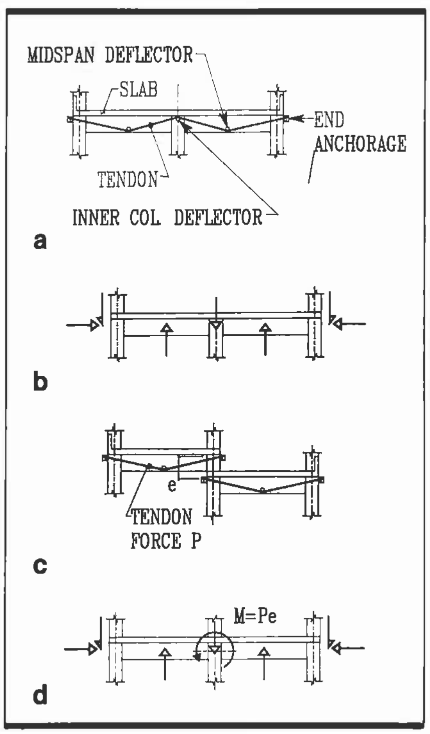


Fig. 6—For the analysis of the beam frames, the externally applied post-tensioning was substituted by the forces it exerts on the structure: a) elevation; b) balanced loading; c) elevation at split level; d) modelled balanced loading at split level.

fectively meet the stringent corrosion protection and fire resistance requirements together with limitations encountered in maintaining minimum clearances for parking. Working around the geometries of the existing structural members was another major restriction. A completely new hardware system was devised for this project. Due to high development costs, hardware design proceeded with the objective of producing a standard system for use in similar rehabilitation projects.

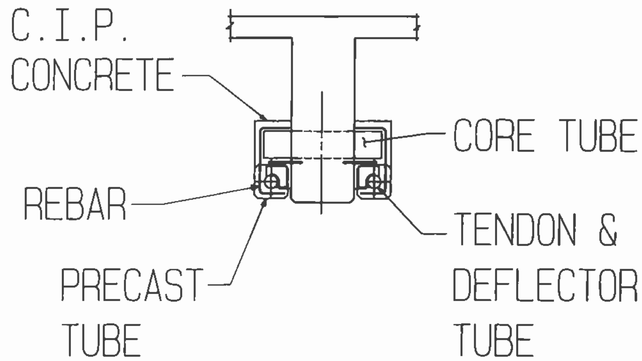
Structural design considerations

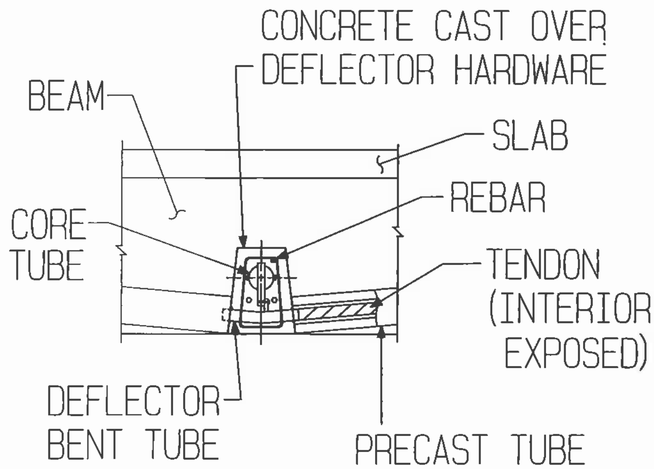
The structural design followed UBC 1982.[1] Two tendons per beam were added, one on each side of the stem. Based on the actual details of each beam regarding the pick-up points, method of anchorage selected, and geometry of connections, the most suitable practical tendon layout was selected. The analysis and design was aimed at determining the number of strands in each tendon to meet the stipulated serviceability and strength requirements.

Concrete strength measured in the existing structure averaged more than 5000 psi (34.48 MPa), but the rehabilitation design was based on 4000 psi (27.58 MPa), which was consistent with the original design. Several questions were addressed in handling the existing post-tensioning, which had diminished in many beams, yet was available with full strength in others.

The original intent for the rehabilitation was to detension the existing unbonded strands, but the benefits of leaving the strends outweighed the potential damages detensioning could incur. In this regard, two factors that could adversely affect the performance of the present design are noteworthy.

First, the precompression from the existing strands would be additive to the compression provided by the new design. Thus the added post-tensioning would have to be kept low to limit the total average compression in the beams to 650 psi (4.48 MPa), a limit considered acceptable in the rehabilitation to mitigate shortening distresses in the existing concrete frame. The available average compression, if fully





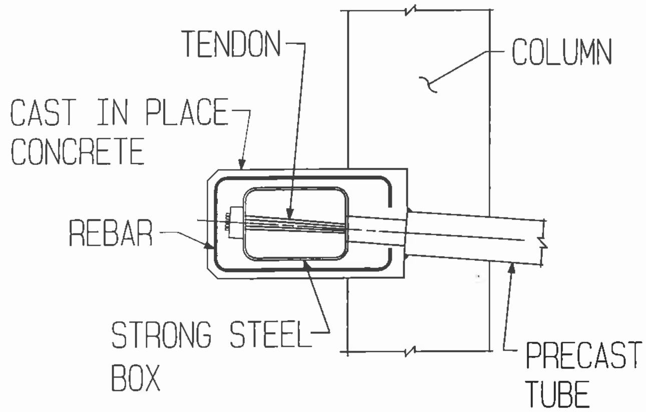


Fig. 7 — (above) The mid-span tendon deflectors consisted of bent tubes anchored in the stem of the beam through a chore tube.

Fig. 8 — (left) The tendons react behind the columns against strong steel boxes that were subsequently encased in cast-in-place concrete for corrosion and fire protection.

effective, was estimated at 250 psi (1.72 MPa), leaving a maximum of 400 psi (2.76 MPa) to be accommodated in the rehabilitation work. For most cases, the actual design concluded with average compressions well below the set limit.

Second, if the existing post-tensioning is considered active, its area of steel would add to the new post-tensioning. When combined with the existing mild reinforcement, it would result at a number of locations in a combined reinforcement index of greater 0.3, the maximum recommended by UBC 2618i. This index is a parameter intended to avoid overreinforcement, thus averting nonductile compression failure. A higher reinforcement index was accepted after it was demonstrated that a compression failure could not occur for the conditions encountered.

The contribution of the existing plastic-wrapped unbonded post-tensioning to the ultimate strength of the beams was disregarded.

Serviceability requirements were set as follows:

1. Maximum fiber stresses under working conditions were 380 psi (2.62 MPa) for tensile stresses and 1800 psi (12.41 MPa) for compressive stresses.

2. Long-term deflections had already taken place. After completion of rehabilitation, deflections due to live loading were set to be less than $l/350$, where l is the span length.

3. Typically, about 65 percent of total shortening in post-tensioned structures is due to shrinkage of concrete. Creep, elastic shortening, and temperature changes account for the balance. The existing concrete, having aged several years, had essentially undergone its total shrinkage. The immediate elastic shortening of the longest beams at the location of the end columns and due to the added average precompression of 400 psi (2.76 MPa) was estimated at 0.9 in. (2.30 mm). The long-term creep associated with this shortening is approximately 0.18 in. (4.57 mm). Because the differential shortening between successive levels is the major cause of column distresses, it is primarily at the lowest level that the columns had to be checked for consequences of shortening in beams.

4. Corrosion protection and fire resistivity stipulations were treated through the selection of post-tensioning strands and hardware design. These are not strictly structural engineering issues.

Strength design

The ultimate strength of the new system was checked for gravity and seismic loadings. For the gravity condition, moments and shears were factored using the following relationship

$$U = 1.4D + 1.7L + Sec$$

where U is the minimum strength to be provided by the beams and columns and Sec is the secondary effects. Secondary effects are the moments and shears due to post-tensioning that are induced in the structure as a result of the restraining effects of the supports on unobstructed displacements of the post-tensioned beams.

The beam-column frames were assigned to resist that portion of the lateral loading that is due to the weight rehabilitation adds to the building. The balance of lateral

Fig. 9 — Most beams were stressed from outside the building.

Fig. 10 — Transfer girders were stressed from within the parking garage.

loading is taken by the existing shearwalls. The following formula was used to check the frame strength

$$U = 0.75\ (1.4D + 1.7L + 1.81E) + Sec$$

where E is the seismic force.

Structural modeling of beam frames

Beam frames were modeled using the load-balancing concept of post-tensioning design in which a force diagram is constructed from the tendon layout, such as the one shown in Fig. 6. The magnitude of the concentrated forces shown is related to the post-tensioning to be applied. The post-tensioning forces required to satisfy the serviceability and strength criteria described in the preceding were computed using the ADAPT Post-Tensioning software system.*

Hardware and details

Several different schemes were studied for the beam post-tensioning hardware. The following criteria determined the final choice:

1. Use of only one pickup point per span. This reduced hardware expense as well as coring costs.
2. Transfer of the post-tensioning force to the column or beams by cored holes and pipes (Fig. 7). No reduction of vertical clearance was allowed.
3. Since the columns were 20 in. (508 mm) wide and the beams only 14 in. (356 mm) wide, the required 2 in. (51 mm) concrete tendon protection had to be either precast or grouted into a pipe surrounding the tendon.
4. Multistrand tendons were selected to minimize their eccentricity at support points.

The beam hardware consists of two components. First, the end brackets that wrap around the columns and transfer the applied post-tensioning load to the structure incorporate 8 x 12 x ½ in. (20.3 x 30.5 x 13 mm) tube sections with stiffeners at the tendons (Fig. 8). The design was governed by shear.

The second component is the deflector, which transfers the vertical post-tensioning forces to the beams and columns. The deflectors were made of 4½ in. (114 mm) extra-heavy pipe. They were connected to the column and beam by slipping through a 4⅝ in. (119 mm) cored hole, which was located so it would miss the existing beam post-tensioning (Fig. 7).

The tendons were protected by 6½ in. square (165 mm square) precast concrete beams with a 2 in (51 mm) corrugated PVC pipe running along the sides of the beam. The precast concrete provides permanent fire cover and is connected to deflectors and end brackets by pipes. The basis of the design was to arrive at the most economical system that would result in the least disturbance to the continuous use of the garage.

The slab tendon replacement was done by removing the corroded ½ in. (12 mm) strands and replacing them with 7⁄16 in., 270 k (11 mm, 1.2 MN) strand coated with epoxy. This is a relatively new product. The epoxy coating is 30 mils thick and very durable. The size of the 7⁄16 in (11 mm) strand with the epoxy coating is approximately ½ in. (12 mm).

Construction

The work was carried out to allow continuous use of the garage by the public. All deflectors, end brackets, and 32 ft (10.3 m) precast members were erected on a shift from midnight to 8:00 a.m. The crew erected as many as 20 members a shift, using forklifts equipped with frames. Prior to the installation of the post-tensioning, the precast members were held in place at midspan by 2 x 4 (48 x 95 mm) supports in the area of the 12 in. square (300 mm square) temporary supports.

The post-tensioning tendons were precut and pulled into their final position during the day shift. Most stressing was accomplished by jacking from the outside of the building (Fig. 9). Transfer girders were stressed from within the building (Fig. 10).

*Bijan O. Aalami, ADAPT Post-Tensioning Software System, 1985, BFL, 1601 El Camino Real, Mountain View, CA, 94040.

Fig. 11 — Closure strips in slabs were opened up for inspection and repair of floor slab strands.

Fig. 12 — After stressing operations, the shoring was removed and the garage was returned to full operation.

The slab tendons were inspected and replaced by removing 4 ft (1.3 m) closure strips located at approximately one-third points along the 370 ft (119 m) side of the structure (Fig. 11). Approximately 65 ft (21 m) of closure were removed at a time, then the tendons were inspected and the corroded ones replaced. The concrete closure was replaced prior to removal of the next 65 ft (21 m) to provide the least disruption to traffic. Approximately 10 percent of the tendons were replaced. Work on the garage started in August 1986 and was essentially complete by early April 1987. Fig. 12 is an interior view of the completed project.

Conclusion

During the early days of post-tensioning, when design experience, post-tensioning hardware, and codes of practice were in development, many buildings — parking garages in particular — were constructed using insufficiently protected strands and other details that are now considered unacceptable practice. Gradual deterioration of inadequately protected unbonded strands inevitably leads to a need for rehabilitation, but through novel engineering schemes and design of special hardware it is now practical to retrofit such structures at reasonable cost. Furthermore, the Pier 39 project demonstrated that such major rehabilitation procedures can be carried out with practically no interruption in the regular functioning of a garage.

It should be emphasized that shortcomings encountered in early application of unbonded strands in commercial buildings have long been recognized and fully rectified. Today, post-tensioning of parking structures is commonly the most economical and performance-healthy mode of design and construction.

ACKNOWLEDGMENTS

The efforts of project manager Manly Jackson and project coordinator Shane Korfike were of particular value in the timely and smooth completion of the project. John Harrison was responsible for engineering evaluation of the project. Bakhtiar Zundi, the resident engineer, meticulously oversaw and inspected the progress of the work. His contribution was crucial in maintaining a high standard of quality control of the construction. Alexander Burga is credited for his contribution in development of rehabilitation options.

Reference

1. *Uniform Building Code,* International Conference of Building Officials, Whittier, 1982, 780 pp.

Keywords: corrosion; **parking facilities; post-tensioning;** prestressed concrete; prestressing; **renovating;** repairs; structural design.

Owner: Pier 39 Associates, San Francisco, Calif.
Evaluation Engineer: Harrison Teasley, San Jose, Calif.
Redesign Engineer: Bijan, Florian & Assoc., Inc., Mountain View, Calif.
Contractor: VSL Corporation, Los Gatos, Calif.

ACI member **Bijan O. Aalami** is the principal consultant to Bijan, Florian & Associates, Inc., a structural consulting firm in Mountain View, Calif., specializing in the design of post-tensioned structures, and a professor of civil engineering at San Francisco State University. He received his PhD from the University of London, England, and is the author of the ADAPT Post-Tensioning software system. He is a member of ACI-ASCE Joint Committees 421, Design of Reinforced Concrete Slabs, and 423, Prestressed Concrete.

David T. Swanson is vice president and corporate engineer of VSL Corporation, Los Gatos, Calif., where he has been involved in the engineering and construction of post-tensioned concrete, transit system, heavy transportation, and lifting projects. He is a graduate of the University of California at Berkeley and is a frequent lecturer on design and construction of post-tensioned concrete structures.

Authorized Reprint From
February 1988, issue of
Concrete International: Design & Construction
Published by American Concrete Institute