Design and Construction of a Modern Post-tensioned Parking Structure

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Structure Description

The parking structure on the California State University, Sacramento, campus comprises two connected six-story units, totaling approximately 1 million square feet and 3,000 stalls. Figure 1 is a recent photograph of the structure, looking east. International Parking Design was the architect of record, and Miyamoto International was the structural engineer for this university-owned building project.

Both units are trapezoidal in plan, with footprints of 100,000 square feet and 60,000 square feet, respectively. Figure 2 shows a schematic of the floor plan. Six-level ramps provide access between the floors of the units. A 6-inch gap allows for differential seismic movement between the units. A typical structural bay measures 17 feet, 0 inches in the north-south direction and 61 feet, 10 inches in the east-west direction. The structure has a uniform story height of 10 feet.

This parking structure is a signature building for the campus. It is visible from the freeway and provides much-needed parking for the university. As such, it was necessary to design and construct a structurally robust and aesthetically pleasing building, as Figure 3 shows.

During the structural system selection process, steel, precast, and cast-in-place concrete framing and concrete shear wall alternatives were considered. The cast-in-place design proved to be the most economical option that met the university's requirements. Unbonded post-tensioning was added to the east-west beams to make the design even more cost-efficient. All floors are 5-inch-thick, normal-weight concrete slabs with one-way post-tensioning.

Concrete special moment-resisting frames (SMRFs) provide resistance to lateral loading in both directions. In the north-south direction, only the frames along the perimeter of the two structures are SMRFs. The rest of the north-south frames were designed to resist gravity loads only. The north-south beams are 28 inches wide and 36 inches deep. In the east-west direction, all the frames are SMRFs, with Cunningham (trapezoidal) beams that are 35 inches deep and an average of 15 inches wide. East-west beams are reinforced with a combination of unbonded post-tensioning tendons draped in a parabolic profile along the span and mild steel top and bottom reinforcement. All frames have 28-inch square columns. Only mild steel reinforces the columns and the north-south beams.

The design uses normal-weight concrete with a nominal compressive strength of 4 ksi, ASTM A706 mild reinforcement, and ½-inch 270-ksi stress-relieved tendons. Cast-in-place drilled piers, or caissons, make up the foundation system. The diameter of the shafts varies from 3 to 6 feet, and the caissons have a typical depth of 22 feet. A number of staircases and elevators are located along the perimeter of the two units.

Post-Tensioning Design

For the east-west beams—with area, *A*, and section modulus, *S*—subjected to gravity moment, M_g, and post-tensioning force, *P,* with eccentricity, *e*, the maximum stress at the top or bottom fiber can be computed as:

$$\sigma = \pm \frac{M_g}{S} + \frac{P}{A} \mp \frac{Pe}{S}$$

By placing the tendon profile in a parabolic diagram approximating the gravity moment, the net effect of prestressing is to counteract a significant percentage of the gravity loading. For this structure, the post-tensioning force was selected to counterbalance 80% of the total dead load acting on the beams. The post-tensioning force is approximately equal to 10% of the product of the beam area (*A*) and the concrete compressive strength (f'_c). American Concrete Institute (ACI 318) limits the maximum compressive stress to 0.45 f'_c and tensile stress to $6\sqrt{f'_c}$ for post-tensioned members. The tendon layout and the level of prestressing force were selected to meet the stress limits after both stressing and prestressing losses. The tendon profile for a typical east-west beam is presented in Figure 4.

Figure 5 shows the bending moment diagrams for 80% dead load and post-tensioning load. The bending moments from prestressing almost counteract the member bending stress resulting from application of 80% of the dead load. Figure 6 shows a typical east-west beam during construction. Note the location of tendons near the top at the joint and near the bottom at close to midspan.

ADAPT software was used to design prestressing for the one-way slabs. The 2001 *California Building Code* (CBC) service load combinations were used to design the prestressing forces and to bound the slab tensile and compressive stresses to ACI 318 limits. The CBC strength load combinations were then used to check the adequacy of the design for factored loads. Temperature and shrinkage mild reinforcement were also added. Figure 7 presents the post-tensioned design for a typical interior bay span.

Lateral-Load Design

The three primary objectives of the lateral design were to: (1) ensure that member sizes and reinforcement area and detailing comply with ACI 318 requirements; (2) verify that the seismic gap is adequate; and (3) verify that the life-safety performance level at the 475-year event is met. To achieve these objectives, a comprehensive and detailed three-dimensional model of the structure was prepared and analyzed. Provisions of the 2001 CBC equivalent lateral-load procedure and ACI 318 were used to design the member sizes and reinforcement, and to detail the members. FEMA 356 nonlinear static and dynamic procedures were then used to check the adequacy of the seismic performance and the seismic gap. This model is shown in Figure 8.

The analytical model included all pertinent features of the parking structure, including openings in the floor for staircase areas. The model also included ramps because they affect the building period and reduce the clear height of columns. Member dimensions were based on centerlines, and member connections were assumed concentric. Floor slabs were modeled as rigid diaphragms. A secondary geometric, P-Δ effect was also included in the analysis. The flexural rigidity of beams and columns was modified to account for cracked sections. The base of the columns was supported by springs in one analysis and modeled as fixed in an alternate analyses

Conservatively, the envelope of the two analysis cases was used in computing the maximum demands for member forces and floor displacements. The properties of support springs were based on soil-structure interaction parameters. The structure was preloaded with gravity loading and then components of acceleration history were applied along the two orthogonal directions of the parking structure.

A site-specific response spectrum was developed based on the geotechnical survey of the site and geologic data. Recorded accelerations from the 1989 Loma Prieta event were modified to develop spectrum-matched acceleration records. Both the normal and parallel components of each of the motions were thus synthesized.

Analysis showed that the member sizes were adequate. The member transverse reinforcement was closely spaced near the joints at the location of plastic hinges to ensure ductile behavior. Columns were typically reinforced with twelve #11 bars. ACI 318 requirements for joint design and column-to-beam moment capacity were also satisfied. The 2001 CBC limits the story drift ratios of structures that have a period greater that 0.7 second to 2% of story height. The largest computed drift ratio for this structure was approximately 1.2%.

The 2001 California *Building Code* recommends the square root of the sum of the squares (SRSS) method in calculating the displacement demand on adjacent structures. The computed SRSS displacement was 5.2 inches at the roof. Because this displacement was less than the provided seismic gap of 6 inches, the design is adequate. Figure 9 shows the response history for the differential movement at the roof. Note that the maximum computed displacement is approximately 4 inches.

Nonlinear static analysis of the fixed-base model was conducted to determine the expected post-yield performance of the structure. At the performance point, the roof displacement is approximately 2.3 inches. This corresponds to the anticipated roof displacement during the code-equivalent (475-year) seismic event. At this level, the members are essentially elastic. Figure 10 shows the deformed shape of a perimeter north-south frame, at a roof displacement of approximately twice the performance point, obtained from analysis. Note that the level of nonlinear deformation is still very limited. Because the design of this building was primarily governed by the drift and seismic gap limits for the spring-supported model, there is significant reserve capacity.

Construction

Construction of the first unit began in 2003. This unit was opened to the public in 2005, and the second unit was completed in late 2006.

As is expected for a structure of this size, a few minor difficulties were encountered during construction. These can serve as lessons for future design and construction. Following is a summary of a few of the more notable issues encountered.

One issue was congestion of prestressing tendons and reinforcement. One such location is at beam-to-column joints. This is typical for construction in seismic zones where the joint is heavily confined to prevent failure. Another location is at the midspan, where both tendons and mild reinforcement occupy the bottom portion of the beam. Because of congestion, it can be difficult to vibrate the concrete adequately, which can result in the formation of isolated rock pockets.

One such rock pocket or void was observed at the bottom midspan of one of the north-south beams after the forms were stripped. The repair consisted of chipping out any loose concrete and filling the void with a cement-based patching compound. Similar locations were also inspected for voids. Figure 11 presents the repaired portion of one beam. The lesson learned here is that it is imperative that details for similar conditions include reinforcement and tendons drawn to scale in order to identify and mitigate constructability issues.

Another issue was that concrete deck was exposed to rain immediately after one of the pours, resulting in an unfinished slab with pockmarks (see Figure 12). This affected more than 7,000 square feet of slab on the second-level deck and required remediation. The rain-damaged concrete deck was a major concern for three reasons. First, the deck had a greater potential for cracking from shrinkage. Second, the durability, or wearability, of the slab was diminished. And third, the concrete strength of the deck, particularly the top 1 inch, might have been compromised. Slab evaluation and remediation consisted of the following process:

1. Representative core samples from the affected area were tested. Test results indicated that the concrete compressive capacity was near or above the specified nominal value. Nondestructive testing was performed prior to coring to identify the location of existing rebar and tendons. Cores were also examined for concrete homogeneity. The cored slab was repaired with a non-shrink grout.

2. The affected slab surface was treated with a resurfacing compound, which restored the slab's structural integrity, durability, and serviceability. Repair consisted of placing a cement-based product over the prepared damaged slab. Before application of the product, the slab area was cleaned with a bead blaster, followed by pressurized water spray to remove all latents, and to moisten the slab. All cored holes were roughened and filled with a non-shrink grout immediately after the slab surface preparation and before placing the topping material. The surface swirl texture was applied immediately after the topping. The topping had an average thickness of approximately 0.18 inch.

Nominal cracks in several columns supporting the ramp to the roof level were observed. Cracking appeared to be nonstructural and likely resulted from the additional restraint of the subject columns by the ramps, along with direct exposure to sunlight and extreme temperatures (in excess of 100 degrees Fahrenheit). Observed cracks were not large enough to warrant remediation.

There were also issues related to the post-tensioning tendons and their anchorage:

1. Figure 13 depicts a void near the anchorage of a slab tendon after the form was stripped. The prestressing special zone reinforcement was unaffected. The repair consisted of chipping out loose concrete, cleaning the reinforcement, and patching the affected area with a cement-based grout.
2. Figure 14 depicts the top of a slab, showing exposed prestressing conduit. The prestressing profile was placed slightly higher than specified in the design, and the concrete cover was less than required. The repair consisted of removing the tendon, routing down the concrete as required, replacing the tendon, and patching the concrete.
3. It was noted that tendon sleeves were cut in several locations. These sleeves were repaired and a watertight seal was restored before the concrete cover was replaced.

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