Structural Retrofit of Special Moment-Resisting Frames of Concrete

A case study

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In the mid-1990s, a cast-in-place post-tensioned concrete parking structure was designed to serve the growing demands at California Polytechnic State University at San Luis Obispo (Cal Poly). Construction began in November 1998 and was completed in September 2000. The 936-stall parking structure has three elevated decks with a footprint of 280 x 300 ft (85 x 91 m) (Fig. 1).

The design of this structure commenced in the wake of the 1994 Northridge earthquake. Given the poor performance of some parking structures during that earthquake, special attention was given to the seismic design and detailing of the proposed structure. The seismic design requirements were enhanced to exceed those of the 1994 Uniform Building Code (UBC),[1] which was in effect at the time. These requirements were developed jointly by the engineer of record—Watry Design Group—and the California State University Seismic Review Board. To isolate the effects of the ramps from the moment frame system, the structure was subdivided into three independent smaller portions separated by light wells. Each of these smaller portions was stabilized by concrete special moment-resisting frames (SMRF). To ensure ductile performance of the structure, all of the columns in the structure (including gravity columns),

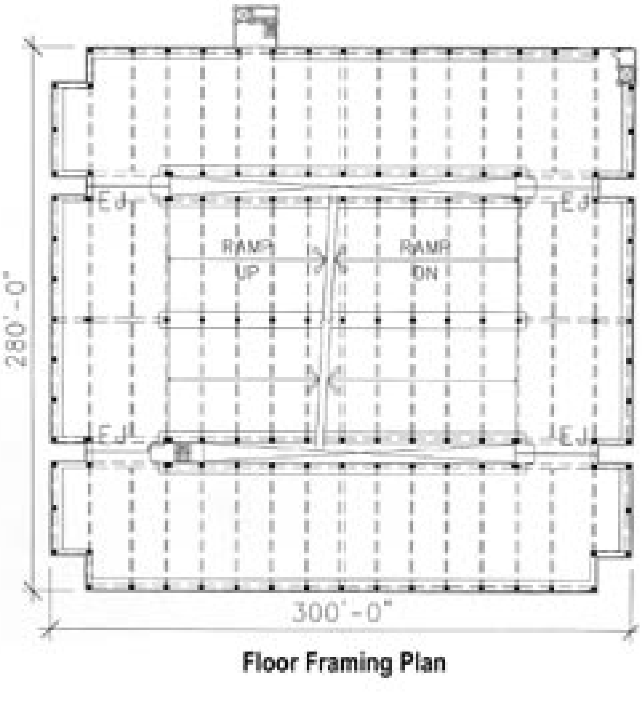

Fig. 1: Floor framing plan showing the configuration of the structure

Fig. 2: View of the completed parking structure with reinforced joints at the lower two levels

were detailed with full confinement ties over their entire height. Figure 2 shows the structure in context with the site.

When a deficiency in the joints of the seismic-force resisting moment frames was discovered during construction, the architect, engineer, and contractor worked collaboratively to develop an innovative repair solution that would restore the project to its intended level of performance. The team was challenged to create an economic and aesthetically pleasing structural repair solution for 240 beam-column joints in the lower two levels. Because most of these deficient joints occurred on the exterior face of the structure, repairs would significantly affect the architectural look of the structure. Alternate framing methods were considered, and a concrete solution was chosen based on seismic performance, economy, aesthetics, and maintenance. The contractor and engineer worked closely to develop innovative construction techniques to complete the difficult repairs. The completed joint repairs blend in with the original framework and are effective in preserving the original architectural expression of the structure.

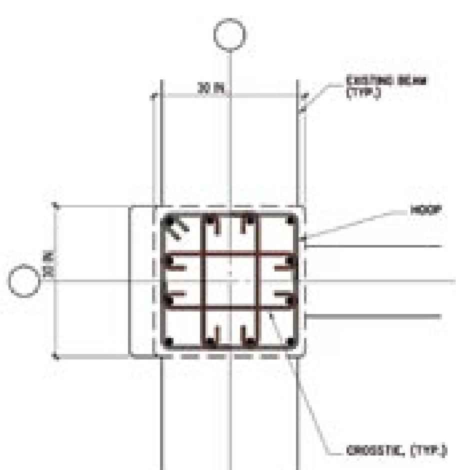

Fig. 3: As-designed joint reinforcing

MISSING CROSSTIES

Moment frame joints must have transverse confinement reinforcement to strengthen the concrete within the joints and to provide lateral support for the vertical column reinforcement. This confinement steel is normally comprised of hoops and crossties (Fig. 3). Both are considered critical for adequate performance of joints as well as overall structural stability and are therefore required by code.[2]

Each moment frame joint was designed to include 36 crossties. During the course of construction, it was discovered that, although column hoops had been installed in the joints, the required crossties had not. The installed joint reinforcement was therefore half of that required by code.

The discovery that the crossties were not in the joints of the second and third levels was made during the construction of the fourth level. Once detected, the problem was quickly corrected at the fourth level by installing crossties in the joints prior to placing concrete. External strengthening was required at the 240 joints that had already been cast at the second and third levels. The typical column size was 30 in. (760 mm) square with a 12-bar configuration. Most of the frame joints had beams on three sides (Fig. 4). The intersecting beams obstructed opportunities to wrap the joints cleanly and added to the complexity of the repair. Also, post-tensioning cables passed through some joints, adding risk to any drilling during the retrofit operations.

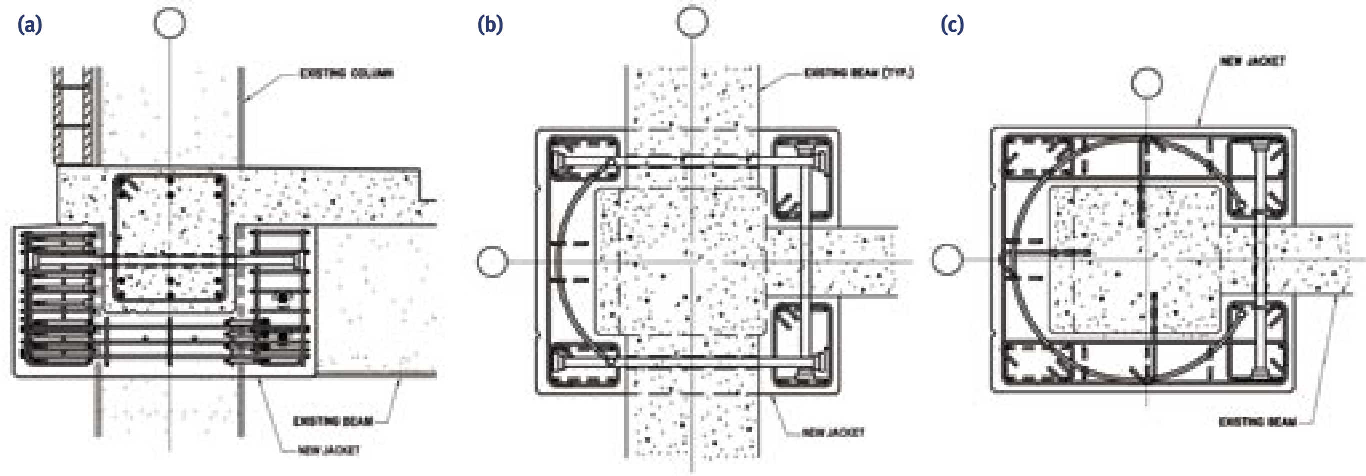

Fig. 4: beam soffit

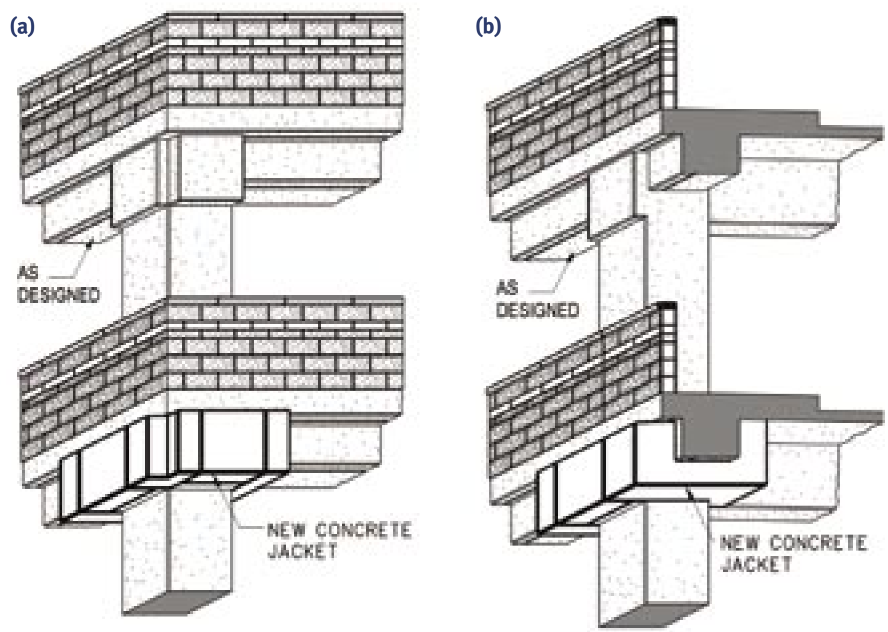

Fig. 5: Isometric studies of as-designed and repaired joints: (a) corner; and (b) interior

ALTERNATIVES

Several alternatives to repairing the joints were developed and evaluated based on cost, performance, aesthetics, and anticipated maintenance. The intent was to restore the lost confinement by creating an external jacket with sufficient strength and rigidity to confine the joint. Carbon fiber wraps were deemed insufficiently rigid to replace the omitted ties. Bolting through the joints was deemed impractical because it would have required drilling 36 holes at each joint. The presence of intersecting beams that prevented full-height access to the joint made these solutions even more impractical. The most practical repair solution was quickly restricted to an external jacket. Although jackets would still require through-bolting, the number of bolts would be limited.

Two design options for the external jackets—structural steel plates or reinforced cast-in-place concrete—were developed and studied. A cost consultant priced both options and found that costs for the two options were about equal at $5000 per joint. The contractor reviewed the proposed retrofit for feasibility and economy. The concrete option was selected as the best value for this project because of superior aesthetics, long-term maintenance, and greater stiffness.

Once the concrete option was selected, the project architects were engaged to make the concrete jackets look as pleasing as possible. The jackets were sized and shaped to match the existing architecture of the structure, and reveals were suggested to add visual interest. Figure 5 shows the proposed repair as designed by the architect, and Figure 6 shows the finished repair.

ENGINEERING DESIGN

Special consultant

The problem facing the design and construction team required a solution that had no known precedent. S.K. Ghosh Associates Inc. joined the team to help research and develop a suitable repair. Once the repair solution was developed, it was reviewed by the California State University Seismic Review Board.

Fig. 6: Ground-level view of completed structure

Fig. 7: Reinforcing and soffit form at interior column

Additional seismic mass

Verifying the capacity of the overall seismic system to accommodate the additional mass from the accumulated weight of the concrete jackets was the first step. The original seismic calculations were based on a design weight of 164 lb/ft^2 (800 kg/m^2), which included a 5% contingency. Each jacket weighed approximately 4700 lb (2130 kg), which added 8 lb/ft^2 (40 kg/m^2) per floor to the second and third levels. Analyses confirmed that the seismic base shear used in the original design had adequate margin to accommodate the additional mass from the concrete jackets. Therefore, detailed calculations for the retrofit were limited to evaluating the effects that the proposed jackets would have on the performance of the moment frames.

Design of jackets

The performance of the frames relied on adequate confinement of the joints, and the absence of crossties required an engineered solution to reestablish this confinement. The repair used concrete jackets to simulate the confinement that would be provided by framing members on all four sides of the joint. By doing so, the existing joint reinforcement provided by the hoops alone is justified by 1997 UBC Section 1921.5.2.2.[2] This section permits 50% of the column transverse reinforcement required in Section 1921.4.4.1 when members frame into all four sides of the joint and where each member width is a least three-fourths the column width. Section 1921.5.2.2 also allows for an increase in tie spacing from 4 to 6 in. (100 to 150 mm).

The jackets were reinforced with a series of tightly spaced circular ties, through-bars, crossties, and small column cages designed to anchor and confine the circular ties (Fig. 7). All reinforcement was low-alloy, ASTM A 706, Grade 60.[3] The circular ties were provided to confine the sides of the columns. They were proportioned to satisfy UBC Section 1910.9.3 (Eq. 10-6) for spiral reinforcement and UBC Section 1921.4.4.1 (Eq. 21-2) for confinement. Given that the cross-sectional dimensions of the jackets ranged from 48 to 60 in. (1015 to 1525 mm) wide, the circular reinforcement typically comprised No. 6 bars spaced at 3.5 in. (89 mm) on center. Where beams interrupted the arc of the circular reinforcement, headed anchors were provided.

Through-bars were provided to confine the face of the beams framing into the joint. These bars had to replace the area of steel lost by the missing crossties and match the area of steel of five No. 6 circular ties. To achieve both goals, No. 14 bars with threaded terminating anchors were selected. The through-bars were designed to overlap with, and to restore the continuity of, the circular ties where they were interrupted by beams. Small column cages located in the corners of the jackets provided a core of confined concrete to anchor the terminating anchors on the No. 6 and No. 14 bars. They also provided an internal, reinforced spreader beam to distribute the clamping force from the through-bars to the face of the intersecting beams.

Shear in moment frame beams

The width of the concrete jackets shortened the clear spans of the moment frame beams by 2 ft (0.6 m), which significantly increased the seismic beam shear. The center-to-center spacing between the frame columns ranged from 18 to 22.5 ft (5.5 to 7 m), so the seismic beam shear (V_e) increased by 7 to 14% with the jackets. The average yield strengths for the No. 4 and 5 bars were determined to be 65 and 68 ksi (450 to 470 MPa), respectively, based on a study of reinforcing bar mill certificates. These yield values along with an increased strength reduction factor for shear and torsion of 0.90 were used to verify that the existing beams had sufficient shear strength. The increased strength reduction factor was based on the 1997 UBC, Section 1920.2.5, which allows such increases when material properties are determined through measurements and testing. Also, the beams were checked for conformance with the proportioning requirements for moment frame beams that require a minimum span-to-depth ratio of 4 to 1. Based on these calculations, supplemental shear reinforcement in the moment frame beams was considered unnecessary.

Shear in moment frame columns

Because the joints extended below the moment frame beams by 13 in. (330 mm), the seismic shear in each of the columns increased significantly due to reduced clear height. The shear strength of the columns was determined using the 1997 UBC, Section 1911.5.6.2 (Eq. 11-15-1) with reinforcing steel properties based on mill certificates and an increased strength reduction factor for shear and torsion of 0.90. These shear strengths were compared to the seismic design shear force (V_e) from the 1997 UBC, Sections 1921.3.4.1 and 1921.4.5.1. The seismic design shear force was determined by dividing the sum of the maximum probable beam moment strengths by the reduced clear height of the column. Based on these calculations, supplemental shear reinforcement in the moment frame columns was considered unnecessary.

Deformation compatibility

Deformation compatibility calculations were performed in accordance with the 1997 UBC, Section 1921.7 to check the columns that were not part of the lateral force resisting system. In accordance with the 1997 UBC, Section 1921.7.3.2, gravity loads were calculated to ensure that they didn't exceed 10% of the nominal axial strength of the columns. Many of the columns above the second level did not exceed this threshold, so column confinement was not required through the joints. Concrete jackets were provided in all other cases where the loads exceeded 10% of the nominal axial strength of the columns. The shear strength of many of the typical columns was justified by summing the top and bottom probable moment strengths of the column and dividing by the reduced clear height. The shear strength provided by the confinement reinforcement usually surpassed these demands. Where more detailed analysis was necessary to confirm the adequacy of the existing columns, the shear strength was checked for each column based on maximum story displacements computed using an amplification factor of $0.7R_w$ instead of the 1994 UBC requirement of $3/8R_w$ at the suggestion of the California State University Seismic Review Board. For columns with reduced heights caused by the jackets, moments due to displacements were determined using finite element analyses. The shear forces in the columns were determined from these analyses and compared to the shear strengths of the columns. Based on these calculations, supplemental shear reinforcement in the gravity columns was considered unnecessary.

Avoiding damage

Many of the joints that needed repair contained post-tensioning tendons and anchors that needed to be located and avoided prior to drilling into the joints. This issue was of paramount concern for safety and economy. The recoil of a severed tendon can cause injury to those nearby from spalling concrete or tendon hardware that may be expelled from the structure. Also, drilling or coring into a tendon group may necessitate costly tendon repairs estimated at about $5000 per tendon. There was no way to avoid these risks entirely, but maximum effort was needed to minimize the potential damage.

A comprehensive set of drawings was prepared to facilitate individual study of each joint. Similar joints were grouped together. Of the 240 joints needing repair, 40 different types of conditions were evaluated and separately accounted for in design. Among these different types of conditions were joints that occurred at stairs, elevators, expansion joints, and along the soil face of a retaining wall at the exterior of the structure. Each joint was detailed using multiple plan sections and cross sections so that its likely performance could be fully understood. Existing post-tensioning was depicted based on information from shop drawings, and precise drilling measurements were provided for each condition. To account for field placement tolerances, a minimum of 2 in. (50 mm) clearance was provided between the theoretical location of a tendon and the proposed drilling location. The effort proved worthwhile, since the work was completed with only one incident where a tendon was damaged.

CONSTRUCTION

The first step in the retrofit was drilling the holes. The retrofit design required several holes to be drilled into each joint. Large holes (2 in. [50 mm] diameter) were required for the No. 14 through-bars. These holes passed completely through the beams. Smaller holes of shallow depth were specified on the back side of the columns for No. 4 dowels. To reduce the risk of damaging tendons, repair specifications required the holes to be drilled, not cored. Experienced drillers have the ability to detect interference and back off without damaging the reinforcement. Drilling the large-diameter holes was the most difficult part of the repair. The weight of the large drilling equipment made it difficult to do accurate work. Initial attempts to do the overhead drilling by hand indicated the need for an automated system. Hand-drilling was too slow for production work. Suspending the drilling equipment with slings was considered, but not implemented due to the difficulty of finding suitable access and suspension points for all the various conditions. The method that was finally used for the majority of the joints involved mounting the drilling equipment on the tines of a forklift. A portable ramping system was developed to level the area for the forklift to drive on. This created level holes through the beams.

The next step was sandblasting the concrete surfaces that would be in contact with the retrofit jackets. Heavy sandblasting was required at all contact surfaces to be covered by the jackets to ensure proper bond between the old and new concrete. Plastic sheets enclosing the work area contained dust from the sandblasting operation.

Once the contact surfaces were prepared, workers cleaned the holes, and charged the through-bars through the holes, and installed threaded terminating anchors on each end of the bars. These anchors were chosen because they develop the tensile capacity of the bars in a short length. Also, they are easily installed in the field, and they are small enough to fit inside the narrow reinforcing bar cages located in the corners of the repair jacket. At this point, workers pressure-grouted the through-bars in place with epoxy and installed the small dowels on the back side of the joints.

The internal reinforcement was then installed in the retrofit jackets. The corner cages were preassembled and lifted into place. The circular ties were fitted into the cages so that the terminating anchors overlapped with those of the through-bars. The work was tedious because of the sequencing necessary to lace the various bars into position. Workers installed miscellaneous trim and skin reinforcement as the last step.

The joints were formed with plywood forms. The design required 3/4 in. (19 mm) chamfers on all corners, and reveals were specified to relieve the bulky look of the retrofit. Workers installed the bottom forms first and positioned them to match the soffit of the post-tensioned beams. As in the case of beams, it was important that the clearance below the jackets meet ADA requirements of 8 ft, 2 in. (2.5 m) clear.[4] The reinforcing bars were then adjusted to maintain adequate concrete cover. The side forms were installed last. Check valves, or ports, in some of the side forms allowed pumping concrete into the jacket where access from the top was not available.

Workers placed concrete from the top and vibrated it using conventional methods whenever possible. Unfortunately, most of the joints did not have adequate access from the top because the retrofit jackets were required to abut the soffit of the existing floor slab. For these conditions, concrete was pumped through the ports previously mentioned. A normalweight concrete with a 56-day design compressive strength of 5000 psi (35 MPa) was used for the jackets. External vibrators mounted on the forms consolidated the concrete. The jackets were designed to restore confinement of the joints, so full contact with the slab soffit was not necessary. Therefore, small gaps between the tops of the jackets and the slab soffit due to concrete consolidation were acceptable. Once the forms were stripped, workers repaired surface blemishes and painted the jackets. Continuous special inspection was provided throughout the repair operations.

IMPACT ON SCHEDULE

The retrofit work was performed concurrently with the remaining construction work, but it still had a significant impact on the original schedule. The repair work for the joints extended the completion date of the project by about 5 months. This included the time needed to assess the problem, conceive and prepare the design, and prepare the repair drawings. It also included time for peer review, value engineering, and construction.

The total construction cost for the retrofit was about \$1.2 million (approximately \$5000 per joint for 240 joints), excluding the costs for design and special inspection.

RESTORED PERFORMANCE WITHOUT IMPAIRED AESTHETICS

Few things are as disheartening to a person working in the construction industry as being surprised during the course of an apparently successful project by news that a major flaw in the constructed work exists. The Cal Poly parking structure was approximately 5 months from completion when the bad news was delivered. Naturally, all parties involved with the project were shocked and disturbed by the news, yet through commitment and hard work, the team was able to develop and complete an innovative solution to a unique engineering problem.

The completed retrofit integrates well with the original architectural design, and is not noticeable to an untrained eye. The concrete jackets provide an economical, durable, and low-maintenance solution that fully restores the seismic performance of the structure. Though nobody wins in a situation like this, there is satisfaction gained from knowing that the project was properly repaired, and that the pleasing look of the finished repair satisfied the client.

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Selected for reader interest by the editors.

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