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Coronado Cays Homeowners Association
Attn: Sergio Gonzalez, Facilities Manager
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Coronado, CA. 92118
Ph: (619) 423-4353, E-mail: maintenance@cchoa.org

Reference: Coronado Cays - Tennis Topped Parking Garage Recommendations

Dear Mr. Gonzalez:

The purpose of this letter is to provide my updated professional structural engineering recommendations for the referenced project. The project consists of an existing elevated post-tensioned concrete slab that serves as a parking garage cover and a tennis court and community deck. The slab is two separate slabs with a construction separation joint between them to help control overall deformation due to concrete shrinkage (due to concrete hydration as it hardens) and concrete shortening due to the post-tensioning process. The slab has cracking in several areas and the slab is sagging (defecting downward) that is uncommon for post-tension slab.

My October 5, 2020 report was based on my initial site walk with the addressee on September 17, 2020 without the benefit of concrete breakout at the problem areas on top of the slab. On October 22, 2020, I performed a second site walk with the addressee, and Alan Sawyer of ALS Reinforcing Steel, Inc., who has been repairing/retrofitting post tensioned slabs for over 30 years. By this time, the tennis court surface had been removed, revealing the absence of protection against water intrusion as well as areas of distress with post tension cables near or at the top surface of the concrete. At this time, Alan was confident that the tendons could be repaired by locating the breaks, installing couplers, pulling new smaller wire through the cable sheathing and re-tensioning once broken out concrete was replaced. The HOA entered contract with ALS to start the concrete break out at the known problem areas and start the process of locating all breaks, removing concrete to make room for the couplers in anticipation of replacing the corroded and broken wires. As the process continued, the vibration due to the concrete chip out process resulted in two separate tendons snapping, one of which hit Alan in the chest. Shortly thereafter Alan let me know for the first time that he was starting to get concerned about more tendon breaks and that tendons were breaking at multiple locations along a single line. After a long discussion, we agreed that it was best to stop the breakout and inform the HOA of our concerns.

It was then decided to take 6 core samples at each of slabs, half for concrete strength and the other half for chloride content, an indicator of how corrosive the concrete is (due to water intrusion).

The specified design concrete strength on the original construction documents was 4000psi. The average tested strength of the six cores works out to 2653.33 psi which is only two-thirds the required strength, not good.
Chloride content in existing concrete structures, measured in parts per million (Ppm), is corrosive starting at 299 Ppm, is considered treatable up to 1496 Ppm, and beyond that should be replaced. The chloride content was measured for each core at the upper 1inch, from 1 to 2 inches below the surface, and 2 to 3 inches below the surface. The values at the upper most portion were higher based on proximity to entry point of moisture, and averaged 1005 Ppm at top, 540 Ppm at 1-2 and 288 at 2-3. This means the upper two inches are considered corrosive, with lower section being almost at the threshold of being corrosive. A corrosive environment in concrete leads to rusting of reinforcing bars

(rebar), which then expands resulting in concrete cracks and delamination, and ultimate spalling (falling portions). The concrete strength can also be diminished, which the concrete compression tests reveal was the case. The concrete could be treated with a liquid product called Ferrogard 903 (applied like pain) which seeps into the concrete and coats the rebar to retard the corrosion process. However, it does not restore the strength of the concrete itself.

The results of the core tests are two-fold: they imply that the concrete can be mitigated for corrosion, but the low concrete strength means that more testing would be required to determine if any areas of the slab have adequate strength to be reused; but it appears that much of the concrete would need to be replaced to facilitate a successful tensioning of new replacement cables.

The ACI (American Concrete Institute) provides the design and construction rules for concrete construction and is adopted by the California Building Code. It provides span to thickness ratio guidelines for sizing slabs. The maximum ratios for post-tension slabs are 42 for floors and 48 for roofs. This slab is considered a floor because it supports people. The existing span to thickness ratio is 42.5 (8-inch-thick slab and 28.33 ft span), so it is a substandard assembly.
For a conventionally reinforced concrete slab, the limit is 33 which would equate to a 10.4-inch-thick slab being required.

Another issue is that almost every column has a square crack pattern at the top of slab which is likely due to inadequate shear strength at the slab to column interface and extra tension on the concrete due to the failing cables resulting in sagging slabs. We do have a concern about the design at the top of the columns. It is also known that the post-tension slab industry was still in the early stages of development and I am concerned that the original design is not as robust as the current code would require.

The reasons that I believe that the current retrofit methodology should be abandoned are:

1. The concrete does not have adequate strength.
2. The process to determine how much concrete could be reused would be time consuming and expensive (concrete cores at 8ft on center in each direction).
3. Knowing the slab is corrosive in most areas is concerning despite the ability to retard the corrosive properties with Ferrogard 903.
4. The danger to the workers and possibly residents to high tensioned cable breaks, which can result in concrete and cable projectiles that could harm property or people.
5. The unknown number of cable breaks... during the breakout process the number of discovered and newly broke cables continued to mount.
6. The unpredictable performance of tensioning replacement cables with mixed strengths and condition of concrete.
7. The unknown cost of the retrofit due to more breakout being required each time a break is discovered, which may in turn cause more breaks.
8. The original design may have just met the standard of its time, but it is clearly substandard to the current understanding of post-tension slab design and performance.

The are other retrofitting options available, but not practical for the following reason:

1. External post-tensioning (EPT) consists of adding post-tension cables to the underside of the structure and providing steel brackets with epoxy bolted connections to hold the cable down 4 to 6" below the slab at mid-span in order to create the upward force to lift the slab. This can be ruled out due to the current ceiling height of the slab: the plans say height varies from 7'-6" to 7'-101/2", but it seems lower than that. EPT would require about a 12inch depth to be effective. It is understood that tall vehicles have already been known to scrap the ceiling.
2. Applying carbon epoxy fiber reinforcement is another external method of reinforcing a slab. This could only be done in addition to repairing the damaged cables, because the slab does not qualify for non-post tension due to the lack of slab thickness.

3. The issues with existing concrete strength also shed doubt on the viability of options 1 or 2.

It is not impossible to retrofit the existing slab for continued use, but what is not known is how long it would take, and what the expense would be. I predict that if the attempted process was to continue, it would ultimately result in the need to replace most of the concrete and end up with many cables with multiple breaks along a given run, which makes it even harder to return the structure to its original capacity.

IF the HOA still wants to pursue a retrofit, then it would be prudent to temporarily protect the areas that have concrete broken out. The only area that seems to warrant shoring at this time would be the column cap at south edge of the slab, at the column on the east side of the construction joint, as the concrete to the exterior of the existing crack will fall away eventually. A follow up inspection of condition may be warranted of the structure in 4 to 6 months to determine if further deterioration has occurred that may require shoring if other action has not occurred yet.

If the HOA elects to abandon the structure and remove it, I do not recommend replacing it in kind. It would be an expensive parking cover and tennis court, particularly since nice public tennis courts are available at the park about a ½ mile south. I would recommend pursuing a lighter weight, less expensive roof structure that is wood framed or possibly steel. The roof covering also provide an opportunity for a large solar array, which would make the expense more affordable over time due to energy savings. A common trend locally and in the southwest is steel canopies with solar panels creating the roof structure. This would allow for using a four-column canopy design, with columns spaced to be friendly to the parking layout and repeating it to provide the full coverage. The HOA should also consider hiring an Architect or Design Build Firm to help determine the best fit for the association.

If you have any questions, please do not hesitate to contact our office. The opportunity to provide this service is appreciated.

Sincerely,
DUNN SAVOIE INC.

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