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Forensic Analysis of an Elevated Pool Vault

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Abstract

Distress observed in the plaster lining and gunite/shotcrete of a pool structure located within a podium slab on the third floor of an eight-story student housing building located in central Texas was determined to be causally related to the gunite/shotcrete mix. The gunite/shotcrete mix combined high alkali Portland cement with siliceous aggregates sufficient to generate alkali-silica reaction (ASR). Additional construction deficiencies associated with the thickness of the pool shell and the clear cover over the steel reinforcement were determined to have exacerbated the distress in the structure. Upon demolition of the pool structure, design deficiencies were subsequently identified in the recessed concrete vault that supported the pool structure. The identified design deficiencies included inaccurate structural design and analysis with finite element modeling software, inconsistencies in the thickness of the floor slab, omission of a shear key at the abutment/connection of the floor slab and the vault walls, and an inadequate amount of bonded, non-prestressed reinforcement in the floor slab. These deficiencies culminated in the demolition and reconstruction of the vault. This paper will explore the different parties involved in the design and construction of the project, the errors that resulted in deficient conditions, and the positions maintained by the different forensic engineering consultants representing the various parties.

Keywords

Analysis, alkali-silica, ASR, building code, construction, case study, craze cracks, defect, deficiency, demolition, forensic engineering, evaluation, finite element, ground-penetrating radar, GPR, gunite, Portland cement, investigation, map cracks, methodology, non-prestressed reinforcement, performance, petrography, plaster, podium slab, pool, reaction, shear key, shotcrete, siliceous aggregates, slab, specification, variances, vault

Introduction and Background

A building located in central Texas was comprised of a five-story wood-framed superstructure (i.e., the framed portion of the building above the foundation) intended for multi-family residential/student housing. The wood-framed superstructure was constructed on a concrete podium slab above a three-story parking garage (one story below-grade) with retail and leasing space at the ground level.

A pool structure (pool vault and pool shell) was located in the plaza deck portion of the podium slab on the third floor of the subject building. According to the structural engineering plans for the subject building, the plaza deck portion of the third-floor podium slab was designed with a rectangular, recessed concrete vault, herein referred to as the "pool vault." The pool vault extended downward into the second-story area below to accommodate the pool shell, which was designed by another engineering firm. The pool shell was designed as a gunite/shotcrete shell that was constructed over the underlying pool vault. The general terminology and cross section of the subject pool structure is illustrated in **Figure 1**.

The authors served as a consultant for the general contractor and were tasked with determining the probable cause(s) of distress in the pool structure and providing recommendations for remedial measures, if applicable.

Pool Shell Evaluation

In order to provide an opinion regarding the probable cause(s) of distress in the pool shell, the authors reviewed the architectural/engineering documents for the subject building and pool structure, performed visual observations of the pool and adjacent deck surfaces, performed a relative elevation survey of the pool coping, observed ground-penetrating radar (GPR) surveys, obtained concrete cores for laboratory testing, and analyzed the

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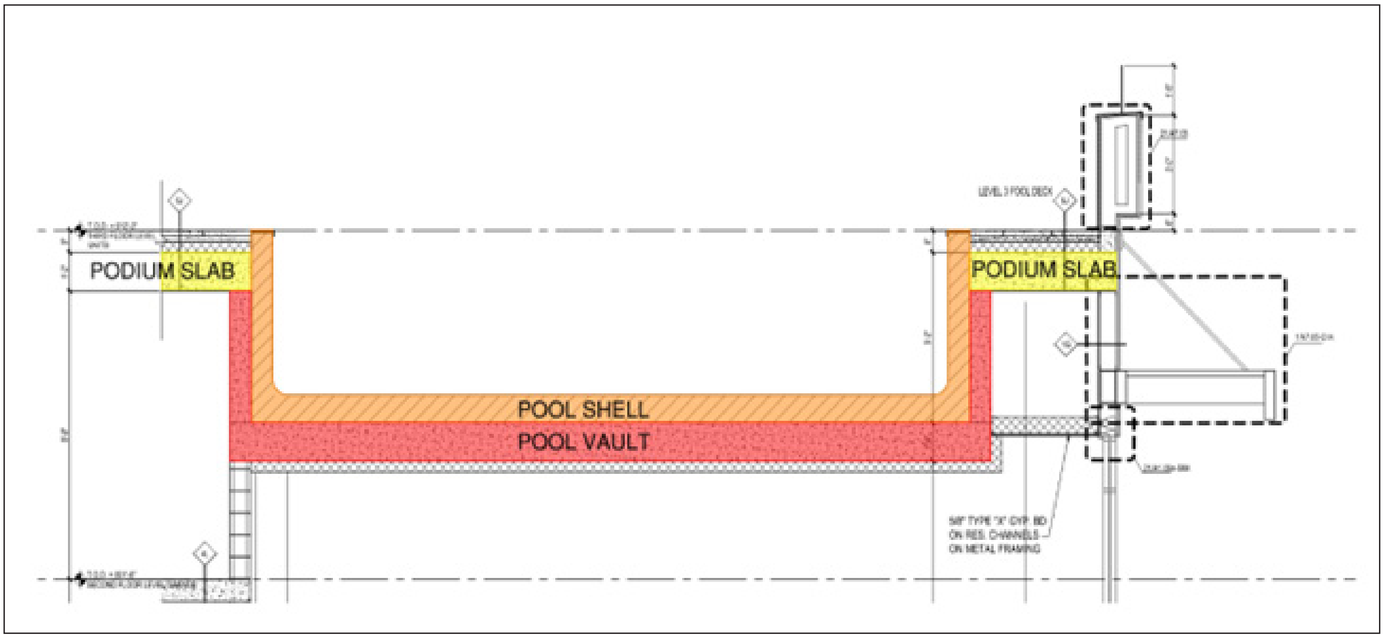

Figure 1
Terminology and cross section of the subject pool structure.

collected information. A general photograph of the pool and adjacent deck surfaces is shown in **Figure 2**.

At the time of the initial site visit, concrete cracks were observed outside portions of the north, east, and south perimeters of the pool vault walls that ranged from hairline (0.003 inches) to approximately 0.020 inches in width. Cracks and/or mortar separations were also observed in the cast stone coping around the perimeter of the pool that ranged from hairline to approximately $^{3}/_{16}$ (0.1875) of an inch in width. Further, cracks in the plaster lining of the pool were observed that typically ranged from hairline to approximately 0.060 inches in width, with the largest crack measured at $^{1}/_{8}$ (0.125) of an inch in width. The cracks observed in the plaster lining of the pool were oriented horizontal, vertical, and diagonal, and most of the cracks appeared to exhibit a pattern consistent with craze cracks or map cracks, as defined in ACI 201.1R-08 and illustrated in **Figure 3**[1].

Utilizing a Zip Level Pro-2000, a relative elevation survey was performed on the cast stone coping around the perimeter of the pool in order to investigate the possibility of post-construction differential movement. The relative elevation survey indicated that the coping around the perimeter of the pool exhibited an overall levelness variance on the order of 1.2 inches. The relative elevation survey

Figure 2
General view of the subject pool and adjacent deck surfaces.

Figure 3
Cracks observed in the plaster lining of the pool shell.

Figure 4
GPR survey performed on the floor of the pool shell.

did not exhibit any salient pattern or trend associated with the levelness of the coping nor any anomalous elevations indicative of structural movement.

GPR surveys were performed within the pool structure to evaluate the placement of steel reinforcement within the floor and walls of the pool shell, as illustrated in **Figure 4**. The GPR surveys performed on the floor of the pool shell did not detect any salient signs of steel reinforcement; however, it should be noted that the presence of cracks and/or moisture may have limited the effectiveness of the surveys[2]. A GPR survey performed on one wall of the pool shell detected the presence of steel reinforcement spaced at approximately 12 inches on center horizontally and vertically. The same GPR survey also indicated an average gunite/shotcrete cover of approximately 3½ inches (horizontal bars) to 3¾ inches (vertical bars). A GPR survey performed on another wall of the pool shell detected the presence of steel reinforcement spaced at approximately 9 to 14 inches on center vertically and approximately 12 inches on center horizontally. The same GPR survey also indicated an average gunite/shotcrete cover of approximately 5¼ inches (horizontal bars) to 5¾ inches (vertical bars).

Selective demolition was performed within the pool shell, as illustrated in **Figure 5**, in order to determine whether the cracks observed in the plaster lining continued into the gunite/shotcrete and to verify the locations of the steel reinforcement indicated in the previous GPR surveys. An exploratory opening located near the central portion of the pool shell extended into the gunite/shotcrete, and it was intended to locate the steel reinforcement in the floor of the pool shell; however, no steel reinforcement was encountered at this location.

Gunite/shotcrete core samples were obtained from various locations within the floor and the walls of the pool shell in order to evaluate the compressive strength of the gunite/shotcrete. The structural engineering plans for the pool shell did not specify a compressive strength for the gunite/shotcrete; however, according to the American Shotcrete Association, in conjunction with the authors' experience in the design, construction, and forensic investigation of pool shells, the average compressive strength of the tested cores (5,515 pounds per square inch - psi) exceeded typical industry strength specifications of around 4,000 psi for most pool structures of similar construction[3].

Additional gunite/shotcrete core samples were obtained for petrographic examination. Concrete/gunite/shotcrete petrography is the examination of prepared samples under microscopes that use reflected light, transmitted light, and/or electron beams to identify basic components of the sampled material, study cracks/microcracks, and identify secondary deposits that form when the material deteriorates. Petrography can be utilized to evaluate proportioning of the concrete/gunite/shotcrete mix (i.e., percentages of aggregates, cementitious materials, water, voids, etc.) as well as evaluate mixing/consolidation, finishing operations, curing, cracking, and causes of low strength. In addition, petrography can be utilized to visually identify mechanisms affecting durability, such as freeze-thaw damage, alkali-aggregate reactions, chemical attack, and more[4].

Through petrographic examination, it was determined that all of the applicable gunite/shotcrete core samples contained macro-cracks, microcracking, and abundant evidence of alkali-silica reaction (ASR) that radiated from chert (hard, fine-grained sedimentary rock) aggregates, cracked chert aggregates, partially-consumed chert aggregates, and/or desiccated alkali-silica gel within voids as illustrated in **Figure 6**.

Figure 5
Selective demolition performed within the horizontal floor surface of the pool shell.

Alkali-Silica Reaction (ASR)

ASR is a chemical reaction that can occur in cementitious mixtures (e.g., concrete, gunite, shotcrete, etc.) between chemical compounds found in Portland cement (alkalis) and silica found in many common aggregates. The alkalis of concern (primarily sodium and potassium hydroxides) form alkali hydroxides as they dissolve in water, which increases the pH of the concrete mixture. Siliceous components of aggregates, such as quartz, cristobalite, tridymite, chalcedony, chert, opal, and acidic volcanic glass, dissolve at higher pH levels (typically above 13) to form a hydroscopic gel that swells and increases in volume as it absorbs moisture. As the gel expands, it exerts internal pressures that can lead to cracking when the internal pressures exceed the tensile strength of the hardened/cured cementitious mixture. Cracking may appear more-prominent near the surface as a result of alkalis migrating upward/outward with the bleed water. ASR cracks typically appear as craze cracks or map cracks near the surface of the concrete/gunite/shotcrete[5].

As previously stated, most of the cracks observed in the plaster lining of the pool appeared to exhibit a pattern consistent with craze cracks or map cracks, which is consistent with distress patterns causally related to ASR. As a result, the authors determined that the distress observed in the plaster lining and gunite/shotcrete of the pool shell was causally related to ASR. In addition, the distress related to ASR was exacerbated by construction deficiencies associated with the thickness of the pool shell and the depth of gunite/shotcrete cover over the reinforcement steel, which likely increased the volume of reactive materials available for ASR to occur and located the tensile reinforcement further away from the surface of the structure, reducing its effectiveness to resist surface cracking.

Figure 6
Desiccated alkali-silica gel adjacent to
a chert aggregate viewed under microscope.

Curative treatments for structures affected by ASR are not easily performed, if at all possible. Isolated repairs of damaged sections are possible; however, ASR can be expected to continue in non-repaired portions of the affected structure. Due to the presence of desiccated alkali-silica gel within voids, as identified in the gunite/shotcrete core samples, it was expected that the distress would likely continue to manifest as the desiccated alkali-silica gel experienced additional hydration and expansion. In some cases, the rate of ASR can be retarded through the application of a waterproofing membrane, which can reduce the volume of water available to fuel the expansion of the alkali-silica gel; however, this treatment was not practical for a pool structure due to the likelihood of contact between the gunite/shotcrete and moisture from the pool water[6].

While curative treatments for structures affected by ASR are not easily performed, if at all possible, preventive measures can be taken prior to construction in order to mitigate the likelihood of ASR occurring. Such preventive measures can include project-specific specifications that limit the amount of Portland cement in an attempt to reduce the alkalinity of the concrete mix, specifications for a low-alkali proprietary cement product, and/or specifications for performing aggregate testing to identify and limit the amount of silica within the concrete mixture.

Pool Shell Demolition

Based upon the magnitude of distress causally related to ASR within the pool shell — in conjunction with the likelihood for such distress to continue — it was recommended by the authors, as well as various other consulting firms (including the owner's consultant), that the pool shell be removed and replaced.

During demolition of the pool shell, cracks were observed at various locations in the concrete support structure underlying the pool shell. Of particular concern was a horizontal crack that exhibited lateral displacement located in the outside face of the vault wall near the abutment of the vault wall and the vault floor as shown in **Figure 7**.

Investigations of the observed cracks in the pool vault were subsequently performed by a consultant for the building owner, which ultimately led to allegations of deficiencies related to the design and construction of the pool vault.

Figure 7
Horizontal crack exhibiting lateral displacement near the abutment of the vault wall and the vault floor.

Pool Vault Evaluation

In order to provide an opinion regarding the probable cause(s) of distress in the pool vault, the authors reviewed the architectural/engineering documents for the subject building and pool structure, performed visual observations of the pool vault, observed GPR surveys, observed selective demolition of the pool structure, reviewed investigation reports issued by other consultants involved in this forensic investigation, and analyzed the collected information considered to be relevant to the evaluation.

The evaluation of the pool vault began with an attempt to compare the measurable aspects of the as-built pool vault with the original design specifications for the pool vault. It was found that the structural engineering plans provided by the structural engineer of record (SEOR) lacked sufficient instruction for the construction of the pool vault as well as the integration of the pool vault with the podium slab.

The structural engineering plans provided by the SEOR did not include any specifications, sections, and/or details associated with the abutment/connection of the third-floor podium slab to the walls of the pool vault. In addition, the structural engineering plans did not include any specifications, sections, and/or details associated with the abutment/connection of the walls of the pool vault to the floor of the pool vault. Further, the structural engineering plans did not include any specifications regarding the thickness and/or steel reinforcement for the walls of the pool vault. The omission of such information was recognized by the general contractor and/or its subcontractors during the original construction, and a request for information (RFI) was subsequently submitted for clarification.

The SEOR responded to the RFI with specifications for mats of vertical and horizontal reinforcement bars to be installed in the walls and floor of the pool vault. The response was void of information regarding specific sections or details that were missing from the structural engineering plans. Ultimately, the pool vault was inspected by a third-party representative prior to concrete placement during the original construction, and, according to inspection reports, the pool vault was apparently found to be compliant with the presumed intent of the SEOR.

The authors and other consultants agreed to perform independent structural analyses of the as-built structure. The structural analyses would evaluate the expected capacity of the structure as-designed, as well as the actual capacity of the structure as-built, with respect to the loads required by the applicable building code. The presence/absence of steel reinforcement, as well as the length of corner bars/hooks (if present), could not be reliably determined through construction-phase photographs, construction-era documentation, and/or non-destructive evaluation. Due to the aforementioned unknowns, structural analyses could not be performed without acquiring additional information about the as-built structure.

As a result, all consultants collectively agreed that selective demolition was warranted in isolated areas of interest, and the demolition would progress as necessary until the as-built construction of the pool vault could be determined with a reasonable degree of certainty to allow for structural analyses of the as-built structure. Ultimately, the selective demolition progressed until it was determined that corner bars/hooks were omitted from the as-built structure. Independent structural analyses performed by each consultant, considering the omission of corner bars/hooks, indicated that the concrete pool vault was not structurally adequate to support the required design loads of the superstructure, and the concrete pool vault was subsequently demolished in its entirety, as illustrated in **Figure 8**.

The owner's consultant concluded the concrete pool vault was not structurally adequate to support the required

Figure 8
Selective demolition of the pool vault.

design loads of the superstructure, and the observed cracks in the pool vault were due to a combination of "poor construction and non-code-compliant structural engineering design." The owner's consultant issued a Certificate of Merit against the SEOR, and legal counsel for the SEOR subsequently retained a consultant of its own to evaluate the claims made against the SEOR by the owner's consultant.

Alleged Design Deficiencies

The owner's consultant performed a structural analysis of the as-designed pool vault utilizing RAM Concept by Bentley Systems, Inc. (RAM)[7]. The consultant acknowledged that the concrete pool vault could have been designed to act as a composite structure (i.e., the walls and slab of the vault work in tandem in the transfer of loads). Alternatively, it could have been designed for the floor slab of the pool vault to transfer loads to supporting columns without additional support from the walls of the concrete pool vault.

An illustration depicting the difference between composite action and non-composite action is provided in **Figure 9**. Regardless of the intent for the design by the SEOR (composite structure vs. non-composite structure), a general contractor and/or its subcontractor would not ordinarily possess the knowledge, education, and/or training necessary to identify and comprehend the intent of the SEOR with respect to the potential need for composite action between the walls of the concrete pool vault and the adjacent floor slabs.

The owner's consultant acknowledged that the as-designed pool vault would not achieve composite action due to the fact that the structural engineering plans by the SEOR did not include the necessary specifications/details to tie the walls and floor slabs together. As a result, the owner's consultant performed a structural analysis utilizing finite element modeling software with a model that did not consider composite action of the concrete pool vault. Based upon the analysis of non-composite action, it was concluded by the owner's consultant that the floor slab of the concrete pool vault experienced stresses that exceeded the allowable stress limits of the applicable building code.

The owner's consultant also performed an alternative structural analysis utilizing finite element modeling software with a model that considered composite action

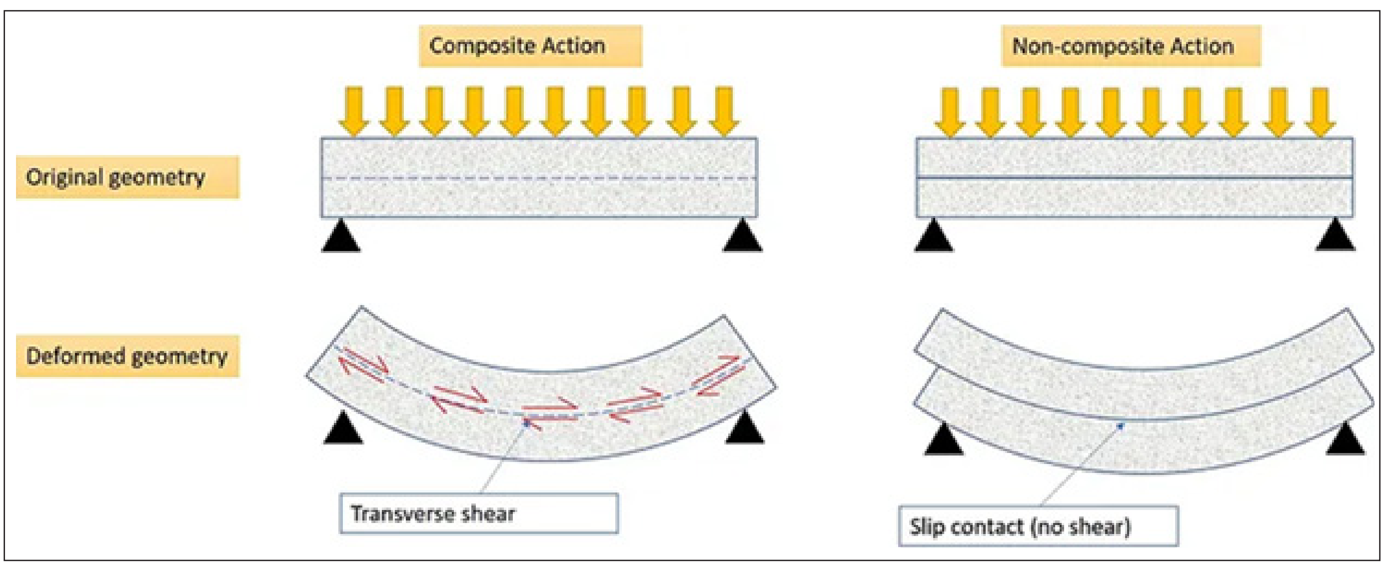

Figure 9
Composite vs. non-composite action.

of the concrete pool vault in case it was the intent of the SEOR for the walls and floor slabs to achieve composite action. Based upon the analysis of composite action, it was concluded by the owner's consultant that the floor slab of the concrete pool vault still experienced stresses that exceeded the allowable stress limits of the applicable building code.

The authors also performed their own structural analyses of the concrete pool vault utilizing RAM with a model that did not consider composite action of the concrete pool vault as well as a model that considered composite action[7]. They also determined that the design of the concrete pool vault by the SEOR was not adequate to support the required design loads of the superstructure regardless of whether or not the vault was intended to achieve composite action.

Based upon the structural analysis performed by the authors for the scenario of non-composite action, it was found that the as-designed structure stress was 236 percent of the allowable stress in bending. For the scenario of potential composite action, it was found that the as-designed internal shear stress in the floor slab was 244 percent of the allowable stress in punching shear at the central columns of the pool vault. The inability of the as-designed pool vault to support the required design loads of the superstructure (within the allowable stress limits) was determined by both the authors and the owner's consultant to be a design deficiency.

It should be noted that a load-bearing wall that supported the wood-framed superstructure from the fourth-floor to the roof was supported on the third-floor podium slab approximately 11 feet away from the concrete pool vault. Based upon the aforementioned structural analyses, it was determined by both the authors and the owner's consultant that the SEOR failed to accurately consider the portion of the design load applied from the aforementioned load-bearing wall that would be transferred through the third-floor podium slab into the concrete pool vault, which contributed to the structural inadequacy of the engineered design, as illustrated in **Figure 10**.

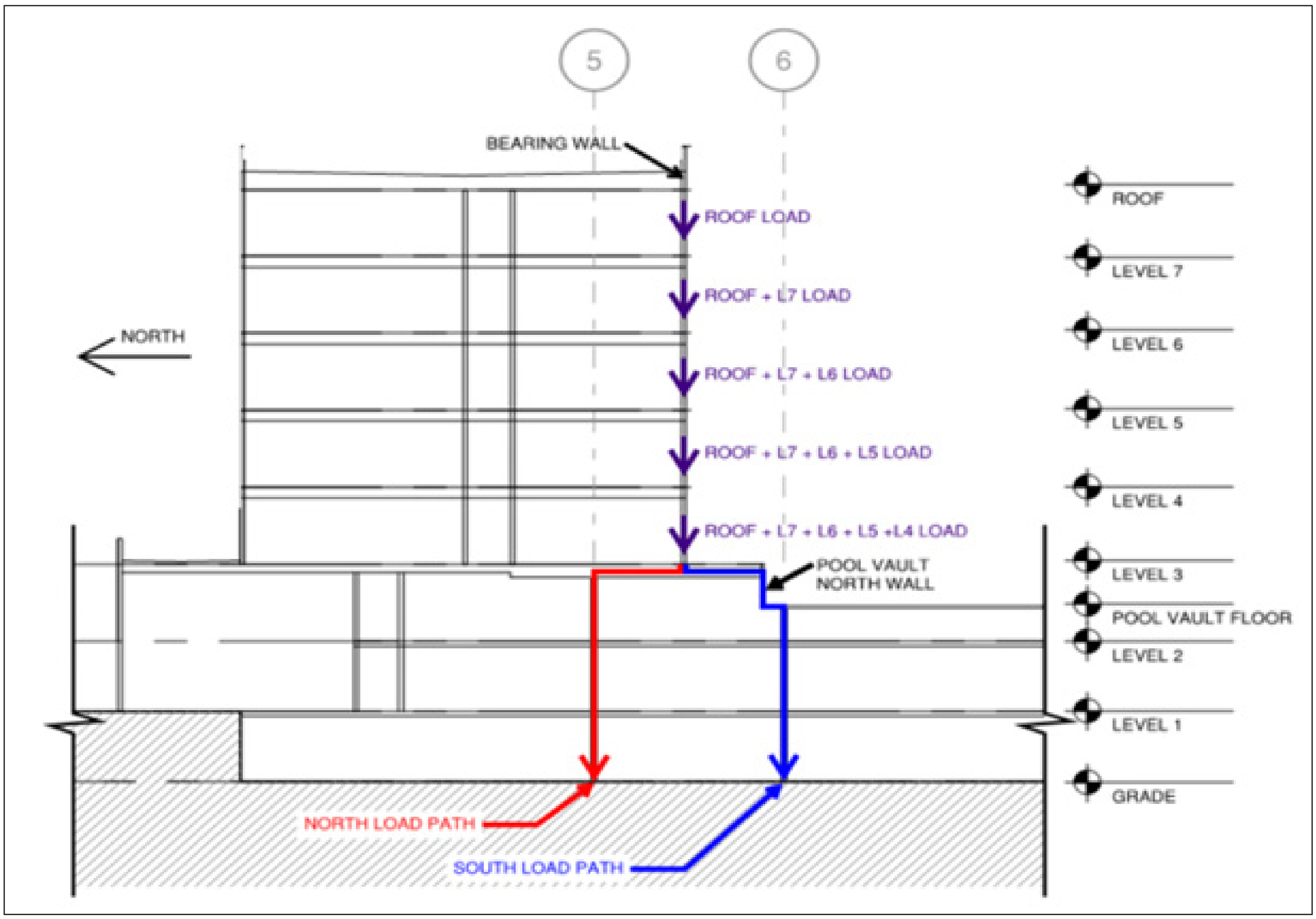

Figure 10
Load-bearing wall supported on the third-floor podium slab.

Based upon a review of the structural engineering plans for the subject building, the authors found that the SEOR did not include any sections or details associated with the abutment/connection of the floor of the concrete pool vault to the walls of the concrete pool vault, including the construction of a shear key at the abutment/connection of the floor and walls, if necessary. The owner's consultant concluded that the SEOR was responsible for a design deficiency associated with the failure to incorporate an abutment/connection of the floor of the concrete pool vault to the walls of the concrete pool vault; however, the SEOR's consultant contested that an adequate load path was provided.

While the general contractor and/or its subcontractors should review the construction drawings related to the constructability of the design, the general contractor and/or its subcontractors are not responsible for (or capable of) performing an independent peer review of the structural engineering plans to verify that the design is structurally adequate or determine whether a shear key should have been incorporated into the design.

The owner's consultant also asserted that the SEOR failed to incorporate adequate bonded, non-prestressed reinforcement (conventional steel reinforcement bars) to meet the minimum requirements of Section 18.9.1 of ACI 318-11 by the American Concrete Institute (ACI) as stipulated by the applicable building code[8]. Due to a lack of required bonded steel reinforcement (i.e., deformed steel reinforcement bars embedded in the concrete), the floor slab of the concrete pool vault was more susceptible to cracking under service loads.

The structural analyses models by the authors corroborated the aforementioned findings of the owner's consultant with respect to a lack of bonded, non-prestressed reinforcement. The inadequate amount of bonded, non-prestressed reinforcement in the floor slab of the concrete pool vault was causally related to a design deficiency by the SEOR. The structural engineering plans for the subject building did not specify an adequate amount of bonded, non-prestressed reinforcement to comply with ACI and the applicable building code.

The SEOR's consultant performed a structural analysis of the as-designed pool vault; however, the consultant did not provide any discussion and/or results of the structural analysis performed. The consultant rather simply asserted that the analyses performed by the owner's consultant incorporated inaccurate assumptions.

Alleged Construction Deficiencies

In addition to the aforementioned design deficiencies, the owner's consultant also asserted claims of various construction deficiencies against the general contractor and/or its subcontractors, including, but not limited to, the thickness of the pool vault floor slab, placement/securing of post-tensioned tendons, premature stressing of post-tensioned tendons, and roughness of concrete at cold joints. While one should endeavor to perform their services in accordance with the building code and/or construction documents, meeting code specifications and/or project specifications after-the-fact is mostly academic. A truly genuine forensic approach to asserted claims of construction deficiencies should not blindly follow codes of practice; rather, it should employ engineering analysis to consider performance aspects of the construction variances before concluding that such variances are construction defects[9].

In the structural engineering plans for the subject building, the SEOR did not include any specifications associated with the thickness of the floor slab for the concrete pool vault. Based upon a review of the design calculations from the SEOR, the owner's consultant stated that the floor slab of the concrete pool vault was intended to be 12 inches thick. The referenced architectural plans for the subject building, however, indicated that the floor slab of the concrete pool vault was to be 14 inches thick. Post-tensioning shop drawings for the third-floor podium slab and concrete pool vault, which indicated that the floor slab of the concrete pool vault was to be 12 inches thick, were reviewed and approved by the SEOR as well as the architect of record (AOR). Following the approval of the aforementioned shop drawings, a concrete forming plan, which indicated that the floor slab of the concrete pool vault was to be 14 inches thick, was subsequently reviewed and approved by the SEOR and AOR. Based upon observations of the as-built floor slab of the concrete pool vault during demolition, the floor slab was constructed approximately 14 inches thick.

The owner's consultant claimed that the design team and the construction team both shared responsibility for the thickness of the pool vault floor slab not being constructed to the thickness intended by the SEOR. The SEOR's consultant claimed that the construction team held responsibility for the thickness of the pool vault floor slab not being constructed to the thickness intended by the SEOR. The owner's consultants claimed that the post-tensioned tendons were placed/profiled in the floor slab of the concrete pool vault in general conformance with the aforementioned post-tensioned shop drawings in consideration of a floor slab that was expected to be 12 inches thick, and the construction of a floor slab 14 inches thick added 2 extra inches of concrete without modifying the tendon profiles. The owner's consultant did not provide any specific discussion regarding how the additional concrete thickness may have adversely affected the structure. The structural analyses models by the authors indicated that the additional thickness of the floor slab was not causally related to the structural inadequacy of the concrete pool vault, and the additional thickness of the floor slab actually decreased the magnitude by which the concrete floor slab was overstressed with respect to shear at column supports.

The owner's consultant also claimed that the elevations of the post-tensioned tendons at the ends of the slab varied from approximately $3^3/_8$ inches for banded/grouped tendons (i.e., tendons spaced closely together) to approximately $7^5/_8$ inches for distributed tendons (i.e., tendons spaced further apart) with an average of $6^3/_8$ inches across the tendons assessed. The owner's consultant referenced Section 7.5.2.1 of ACI 318-11 with respect to tendon placement tolerances[8]. ACI 318-11 states that the allowable performance tolerance for an individual post-tension tendon is +/- ½ of an inch. Although some individual post-tensioned tendons exhibited elevations that exceeded the applicable individual placement tolerance, the average tendon anchor placement reported by the owner's consultant ($6^3/_8$ inches) did not exceed the tolerance.

The SEOR's consultant claimed that the construction team held responsibility for any resultant damage attributed to the variance in elevation of the post-tensioned tendons; however, the consultant did not provide any analysis or discussion regarding the alleged construction deficiency.

The owner's consultant did not perform any structural analysis of the concrete pool vault in consideration of the as-built tendon placement and did not draw any conclusions regarding any potential effects of the as-built tendon placement with respect to the structural integrity of the concrete pool vault. The owner's consultant also suggested that the post-tensioned tendons in the floor slab of the concrete pool vault may have been stressed prior to the concrete attaining the minimum strength specified by the SEOR at the time of stressing. Compressive strength testing of concrete cylinders during construction indicated that the concrete placed at the floor slab of the concrete pool vault exhibited strengths of 3,420 pounds per square inch (psi) at three days, 4,400 psi at seven days, and 5,670 psi at 28 days. The structural engineering plans for the subject building specified that the post-tensioned tendons should not be stressed until the in-place concrete attained a minimum compressive strength of 3,750 psi. The owner's consultant surmised that the required concrete compressive strength for stressing of post-tensioned tendons was likely attained in the concrete test cylinders at an age between three and seven days. Based upon construction-era documentation, the post-tensioned tendons were stressed five days after the concrete had been placed.

The owner's consultant suggested that the actual compressive strength of the in-place concrete could have been lower than the compressive strength of the laboratory-cured concrete cylinders due to the fact that the cylinders were cured at temperatures ranging from 70°F to 74°F while the in-place concrete at the job site likely experienced a temperature range between 75°F to 100°F.

The authors and the SEOR's consultant opined that the owner's consultant failed to prove that the concrete in the floor slab of the concrete pool vault had not attained the minimum required strength prior to stressing of the post-tensioned tendons. In addition, the authors opined that the owner's consultant failed to prove that the cracks observed at the edge of the pool vault floor slab were causally related to premature stressing of the post-tensioned tendons. If the cracks at the edge of the pool vault floor slab were causally related to premature stressing of the post-tensioned tendons, the cracks would have manifested at the time of stressing (or shortly thereafter); however, no documentation was provided that suggested cracks were observed and reported at the time of original construction.

The owner's consultant and the SEOR's consultant also claimed that the construction team failed to roughen the surface of the floor slab of the pool vault prior to placement of the walls. The surface of the cold joint between the floor slab of the pool vault and the walls of the pool vault was reportedly not roughened to an amplitude (measurement of vertical surface deviation between peaks and valleys) of ¼ of an inch as specified by the SEOR.

The authors opined that the original as-built concrete roughness at a location of demolished concrete may be difficult to evaluate during post-construction demolition. The owner's consultant did not perform any structural analysis of the concrete pool vault in consideration of the purported as-built roughness of the pool vault floor slab at the cold joint between the floor slab and the walls, and the consultant did not draw any conclusions regarding any potential effects of the concrete surface roughness with respect to the structural integrity of the concrete pool vault.

The authors maintained the position that none of the construction deficiencies asserted by the owner's consultant were causally related to the inability of the concrete pool vault to support the design loads of the superstructure. Even if the concrete pool vault had been constructed in absolute perfect conformance with the structural engineering plans, the concrete pool vault would still be structurally deficient due to an inadequate design by the SEOR. Remediation/replacement of the concrete pool vault would still be warranted at the subject building.

Summary of Findings

The general contractor and/or its subcontractors should review the construction drawings related to the constructability of the design. The general contractor and/or its subcontractors, however, are not responsible for (or may not be capable of) performing an independent peer-review of the structural engineering plans to verify that the design is structurally adequate, which is what would have been required to avoid the documented structural issues with the concrete pool vault.

The design of the concrete pool vault by the SEOR was not adequate to support the required design loads of the superstructure, regardless of whether the vault was intended to achieve composite action or non-composite action and regardless of whether additional loads were considered from the adjacent load-bearing wall on the third-floor podium slab. The structural inadequacy of the concrete pool vault was causally related to a design deficiency by the SEOR, and it was not causally related to any potential construction deficiencies by the general contractor and/or its subcontractors.

None of the alleged construction deficiencies associated with the as-built thickness of the pool vault floor slab, placement/securing of post-tensioned tendons, premature stressing of post-tensioned tendons, and/or roughness of concrete at cold joints was causally related to the inability of the concrete pool vault to support the applied loads. Even if the concrete pool vault had been constructed in absolute perfect conformance with the structural engineering plans, the concrete pool vault would still be structurally deficient due to an inadequate design by the SEOR, and remediation/replacement of the concrete pool vault would still be warranted.

Conclusion

This case highlights a matter where distress observed in the plaster lining and gunite/shotcrete of an elevated pool structure due to ASR ultimately led to the demolition of the pool structure and subsequent discovery of a more-serious structural issue associated with one or more design deficiencies.

Although the owner's consultant asserted allegations of both design and construction deficiencies, the alleged construction deficiencies were found to be unproven and/or inconsequential to the performance of the structure. Due to inaccurate design and analysis with finite element modeling software, the omission of specifications/details for tying walls and floor slabs together, errors associated with load paths, and inadequate specifications for minimum amounts of bonded reinforcement, it was determined that the as-built construction of the concrete pool vault was found to be inadequate. As a result, the vault had to be demolished and reconstructed.

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