Failure Investigation of a Helical Anchor Tie-Down System Supporting an Olympic Size Swimming Pool

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ABSTRACT

Adoption of new technologies and a push for money-saving value engineering designs may produce unpredictable and unwanted results. Particularly with shrinking budgets, proposals that reduce initial costs become more appealing. However, without careful consideration and implementation, cost-reducing measures can become more expensive in the end.

This paper presents a case study of geostructural forensic analysis related to the failure of a helical anchor tie-down system selected to support an Olympic size swimming pool against hydrostatic uplift forces. The selection of helical anchors over a more expensive traditional anchorage system appeared to be a smart value engineering decision for the project's design-build construction team. However, structural failure occurred soon after construction. A review of design and construction documents revealed a myriad of mistakes leading to the failure and very costly repair of the pool's bottom slab. The demolition and consequent restoration of the slab triggered the forensic study.

The geostructural forensic analysis initially focused on the tension capacity of the anchorage system. However, review of design data indicated several critical mistakes at the anchor-to-concrete slab connections. Moreover, issues with final installation elevation, which were overlooked in the original design and construction, necessitated the need for field modification of the connection. A step-by-step summary of the forensic analysis of the tie-down support system failure is presented herein.

INTRODUCTION

Any structure constructed below the water table must be able to successfully resist buoyant forces in order to remain in place. Swimming pool structures constructed wholly or partially below the water table are subjected to hydrostatic uplift when the weight of the pool water plus the dead load of the pool structure is less than the weight of the volume of groundwater displaced by the pool. Moreover, the design should also consider the case when the swimming pool is empty, which can be the governing case in situations involving uplift forces. An anchorage system is required to keep these structures in place.

There has been growing interest in helical pile and helical anchor applications in the United States since 1980's. The increasing popularity is dictated to a certain degree by a better familiarity and confidence in this relatively new technology within the construction community. It is also driven by economics. The cost for installing helical anchors is substantially lower than traditional driven or drilled piles. As a result, the number of complex structures supported by helical piles is constantly rising. The majority of these projects employing helical piles provide big savings. Nevertheless, failures do occur. This case study shows an example of a helical support system failure that was caused by both design and construction mistakes.

BACKGROUND

The swimming pool that is the subject of this paper was constructed in an area having a high groundwater table. According to the geotechnical report, the groundwater has a static level of about 1.5 m (5 ft) below existing grades. The report also indicates that the water level may fluctuate seasonally by up to 1.2 m (4 ft). Consequently, a dewatering system was needed during construction to temporarily drawdown the groundwater level so that the work could be performed "in the dry". Moreover, normal groundwater conditions necessitated that the pool design include provisions for an appropriate anchorage system to enable it to resist hydrostatic uplift (buoyant) forces during its service life. The anchorage system selected for this purpose utilized helical steel piles in a rectangular grid pattern, whereby, piles were connected to a steel anchorage assembly that was cast-in to the bottom slab of the pool. Helical anchors were oriented in an orthogonal grid spaced at roughly 2.74 m (9 ft) on-center over an area approximately 50 m (164 ft) by 25 m (82 ft) in plan. A total of 208 anchors were installed to resist the uplift force, see Fig. 1 for pile layouts.

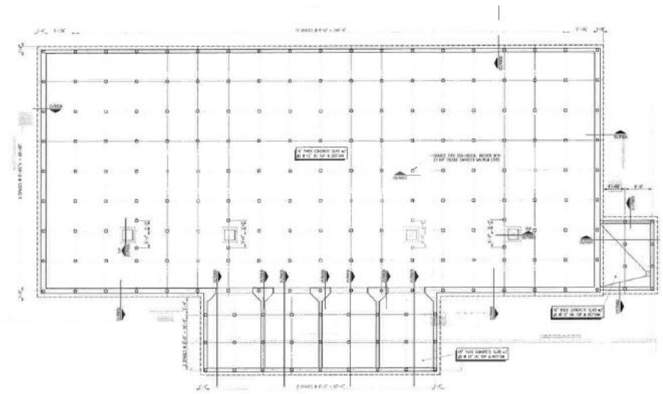

Fig. 1. Swimming Pool's Anchorage Layout.

Cast-in-place reinforced concrete construction was used for the walls and floor of the pool. The pool was designed as a liquid retaining structure with an 18 inch thick cast-in-place concrete bottom slab, doubly reinforced with #5 deformed reinforcing bars spaced at 12 inches on-center in both directions. Pool walls were designed as cast-in-place cantilevered retaining walls.

After placement of concrete walls and the bottom slab, temporary dewatering wells were turned off while construction continued with the pool being empty. The bottom slab of the pool was noticed to rise approximately three months after the dewatering system had been turned off and decommissioned. The bulge was measured to be about 15 inches along the central portion of the pool. This failure was sudden, without any apparent prior signs of distress. The decision was made to partially fill the pool with water to counteract the buoyant force, which allowed the slab to drop more than half the distance toward its original position.

DESIGN AND INSTALLATION CONSIDERATIONS

Individual anchors in the interior region of the pool slab must resist large uplift forces. As a minimum, the interior anchors used for the pool could be designed for the hydrostatic uplift forces acting on the tributary area of a single anchor. In general, the net hydrostatic uplift force is equal to the buoyant force minus the weight of the pool slab.

The uplift load acting on the bottom of the pool slab follows a simple load path. When functioning correctly, uplift forces are transferred from the concrete slab through a structural connection to anchors and then safely into the supporting ground. Even if anchors have sufficient capacity to resist uplift forces, the slab can still ultimately fail if at least one of the elements making up the force transferring connection system between the slab and anchors fails or if the slab is not adequately designed to bridge between anchor supports.

Subsurface Conditions

A subsurface investigation including soil borings, cone penetrometer testing (CPT), and laboratory analyses were conducted by a local geotechnical firm for the design of the pool facility. Soil borings indicated relatively uniform soil conditions at the site. The pool area is underlain predominantly by marine deposits consisting of fine and medium poorly graded sand (SP), interlayered with silty sand (SM) and occasional lenses of silt (ML) and clay (CL). The subgrade soils are generally loose to medium density with standard penetration test (SPT) values ranging between single digits to low teens at the upper 15 to 20 feet below the bottom of the pool. There is a distinct increase in blow counts, with N values over 40, directly below the loose and soft upper layer. The CPT sounding results, presented in Fig. 2, correlate well with soil boring data.

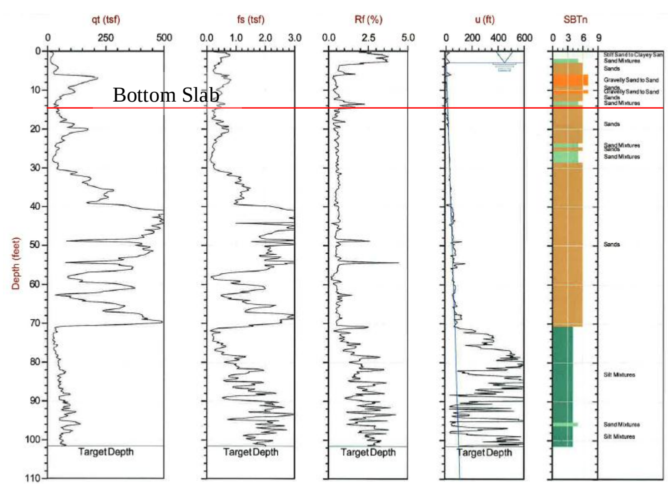

Fig. 2. CPT Sounding Results – Horizontal Red Line Indicates Bottom of Pool's Slab.

Based on the field and laboratory test results, the geotechnical engineer recommended 12-inch square pre-cast concrete piles for the pool's wall and slab support. Pile embedment was anticipated at about 35 to 40 feet below existing grades in order to develop required pile capacity. The predicted allowable capacity in compression and in tension was 50 tons and 8 tons, respectively.

Helical Anchors Design

The helical anchors used on the project were manufactured by Hubbell Power Systems/Chance Civil Construction. The type of helical anchor used was model SS5, consisting of a 1-½ inches square solid steel rod with three attached helix plates 8, 10, and 12 inches in diameter. Three different pile lengths (28, 32, and 36 ft) and helix configurations were considered for the pool anchorage system based on the location and soil boring data. The selected system was designed for an allowable tensile capacity of 27 kips. Catalog information for this model of helical anchor indicates maximum ultimate tensile capacity of 55 kips. Design compressive and tensile resistance for this device is based on theoretical and empirical methods and checked in the field by installation criteria and limited pull-out tests on selected piles.

The anchorage system used also depended on the connection between the helical anchor and the pool bottom slab to transfer uplift forces from the slab to the helical anchors. The anchor cap assembly consists of a pipe sleeve and steel cap plate that are fitted loosely over and connected to the square shaft of the helical anchor. After being assembled and attached to the helical anchor, it is embedded (cast) into the pool slab. Pipe sleeves used on this project were originally designed to be connected to the helical anchor shaft using a bolted pin-connection, as shown in Fig. 3. This pile cap connection has been rated in the catalog for a maximum tensile capacity of 20 kips.

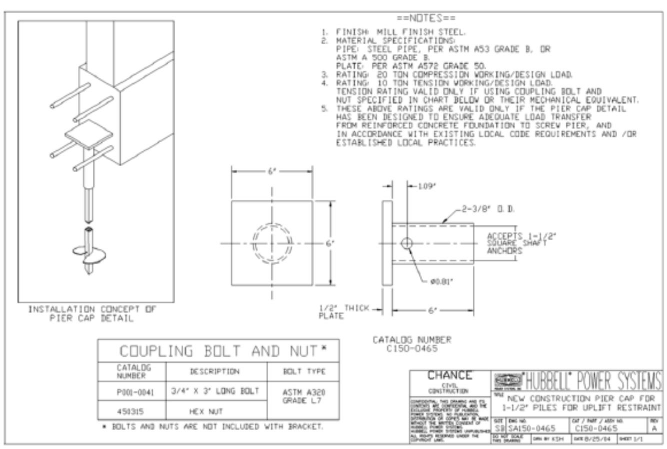

Fig. 3. Anchor Cap Detail – Hubbell Power Systems.

However, this connection was field modified so that the as-built shaft-to-cap connection was welded using inconsistent weld types and procedures, Fig. 4.

Installation

Construction installation logs indicate that the actual anchor length varied between 22 to 32 feet. All of the installed anchors met the driving criteria defined as 5,500 ft-lb of torque, or the manufacturer defined maximum twist of the steel rod. A verification load test was performed on five anchors. One of the tested anchors failed the 200% working load test acceptance criteria.

Fig. 4. Field Modified Anchor Cap Where Helical Shaft Has Been Inserted Through a Hole in the Cap Plate and Welded.

FIELD INVESTIGATION

Damage to the bottom slab of the pool necessitated its replacement, which allowed the opportunity for a closer look at the anchor and slab condition during the demolition phase. Field measurements confirmed that helical anchor shafts were 1-½-inch square steel bars and that the pipe elements making up the pier caps were 2 inch nominal diameter standard weight steel pipes, as indicated on the Hubbell/Chance reference drawing, Fig. 3. Cap plates, however, were found to be connected to the pipe sleeves by welding rather than pin-connection.

Although the reference drawing called for pre-drilled holes in the pipe sleeve and shaft, through which a bolt (pin) would be inserted to connect the shaft to the pipe, an alternate connection method was apparently used. It is likely this change was made in the field to adjust for the random variations that were likely encountered in the top-of-shaft elevations. These elevations could be expected to have varied greatly among individual anchor installations due to the differences in helical shaft penetration depths.

Exposed steel rod tops were generally at their design elevations indicating adequate embedment and sufficient uplift capacity. Most of the helical shaft rods exposed during demolition showed inelastic twist deformation of approximately ¼ turn in the upper eight inches of exposed shaft length. The observed permanent twist deformation very likely occurred during installation at the maximum installation torque which was sufficient to produce inelastic torsion in the shaft, Fig. 5.

Longitudinal cracks (i.e., cracks in the long direction of the pool) were observed in the pool floor prior to its demolition. Diagonal cracks radiated out from the corners of the pool slab which intersected the cracks running longitudinally. Both types of cracks were a result of the upward forces on the pool floor as a consequence of the slab hold-down failure.

Fig. 3. Twisted Helical Anchor.

Close study of areas where the concrete slab was removed led to the following observations related to the root cause of the pool slab hold-down failure:

1. Embedded sleeve anchors did not show any sign of pullout from concrete slab.
2. Slab hold-down failure appeared to have originated with the welded connection between the embedded pipe sleeve cap plate and the helical anchor shaft. The weld connection failure was evident for most helical anchor shafts exposed during demolition. It was apparent that the failures occurred and propagated along the weld lines.

STRUCTURAL ANALYSES

A satisfactory structural design requires that every element of a structural system possess sufficient strength to safely resist expected design forces. A structural system will fail when its weakest element cannot adequately resist applied loads. Structural analyses were performed to assess the design forces and the strength of each component making up the anchor cap connection between the helical anchors and the pool slab. Component elements of both the as-designed and the as-built field modified connections were investigated. Knowing the relative strengths of the component elements of the connection, a hypothetical failure hierarchy based on component strengths was determined. Although the structurally weak link in the anchorage system was known in advance from field observations of the failed anchor cap weld, it is of interest to assess the other components of the anchorage system.

Design strength analyses were consistent with relevant sections of ACI 318 (American Concrete Institute, Building Code Requirements for Structural Concrete) and AISC (American Institute of Steel Construction) Manual and Specifications.

Design Load of an Individual Anchor

The governing design load (critical load case for structural design) acting on the helical anchors occurs when the pool is empty and the groundwater table is at its highest. This scenario results in a hydrostatic uplift pressure equal to the groundwater pressure minus the dead load downward pressure of the pool slab.

The groundwater pressure is equal to the density of water times the distance between the bottom of the pool slab and the highest potential groundwater level. As noted earlier, the static groundwater level, as given in the project geotechnical report, was about 1.5 m (5 ft) below the existing grades. Taking into consideration the seasonal fluctuation of up to 4 feet and building elevations, the bottom of the pool slab could be 3.8 m (12.4 ft) below the highest potential groundwater level corresponding to a groundwater pressure of about 775 psf. The dead load pressure of the slab, estimated as the density of reinforced concrete (150 pcf) times the thickness of the slab (1.5 ft), is 225 psf. For design purposes, the groundwater pressure and dead load pressure are multiplied by appropriate load factors, as specified by ASCE 7 (Minimum Design Loads for Buildings and Other Structures), to account for deviations and uncertainties in determining the actual loads. The most unfavorable load combination, using allowable stress design procedure (the design procedure indicated on the design drawings), results from a load factor of 1.0 times the groundwater pressure (acting upward) and a load factor of 0.6 times the dead load pressure (acting downward), resulting in a net uplift pressure of 640 psf (1,037.5 psf using load factors appropriate for strength design procedure). Given that the tributary area of helical anchors used for the swimming pool was 78.56 sq. ft. (9 ft by 8.73 ft), the maximum required uplift resistance based on allowable stress design is based on a service load of 50.3 kips (or 81.5 kips using strength design), significantly larger than the service load of 27 kips specified on the design drawings.

Failure Modes at the Anchor Cap Connection

There are eight primary failure modes associated with the originally designed anchorage system connection. Each would need to be checked in order to ensure adequate strength to resist uplift forces on the pin-connected helical pile anchor caps. The eight failure modes are:

1. Concrete breakout
2. Anchor pullout (from concrete slab)
3. Yielding of the gross section of the steel pipe
4. Rupture of the net section of the steel pipe
5. Shear rupture at the pin-connection
6. Shear rupture of the bolt
7. Bearing at the bolt hole
8. Weld between the cap plate and steel pipe

These eight failure modes were checked for resistance to the specified tension design load of 27 kips as shown on the drawings. In addition to these failure modes associated with the anchor cap, our analysis indicates the anchor/soil pull-out capacity is also insufficient, based on the design load as calculated in previous section.

Concrete breakout and anchor pullout relate to failure within the concrete slab. The strength of the anchor cap based on these two failure modes can be reasonably assessed following the guidelines in Appendix D (Anchoring to Concrete) of the ACI 318 Building Code. The concrete breakout strength could also arguably be assessed following the provisions of Chapter 11.11 (Provisions for Slabs and Footings) of the ACI 318 Building Code. The anchor pullout strength was found to be adequate and therefore not a concern. However, the concrete breakout strength was found to be inadequate compared to the specified service design tensile load of 27 kips, regardless of whether it is assessed using Appendix D or Chapter 11.11 of the ACI 318 Building Code.

The remaining six failure modes relate to failure within the steel pile cap and their strength can be adequately assessed following the guidelines of the AISC Steel Construction Manual. Based on Hubbell/Chance literature for the pile cap fabrication, Fig. 3, the steel pipe could be either ASTM A53 Grade B or ASTM A500 Grade B steel which have slightly different material properties. For analysis purposes, ASTM A53, Grade B steel was assumed.

Design strengths for yielding of the gross section and rupture of the net section of the steel pipe were evaluated following Chapter D, Section D2, of the Thirteenth Edition AISC Specification. Results indicated that the anchor cap pipe was insufficient to resist the specified service design load of 27 kips.

The design strength for shear rupture at the pin-connection was evaluated in a manner consistent with Chapter D, Section D5, of the AISC Specifications and was found to be adequate to resist the specified service level design load of 27 kips, but would not have been sufficient to resist the maximum design load as calculated in the *Design Load of an Individual Anchor Section,* above.

In the original anchor cap connection design by Hubbell/Chance, a ¾-inch diameter ASTM A320, Grade L7 bolt was indicated, Fig. 3. The allowable shear strength of the bolt was evaluated following Chapter J, Section J3.6, of the AISC Specifications and found to be inadequate to resist the specified design service load of 27 kips. It should be noted, however, that because ASTM A320, Grade L7 bolts are not covered in the AISC Specification the properties for an ASTM A325 bolt were used for analysis. The ASTM A325 high strength bolt has nearly the same minimum tensile strength as an ASTM A320, Grade L7 bolt (120 ksi vs. 125 ksi) and similar minimum yield strengths (92 ksi vs. 105 ksi).

The lowest allowable strength was found to be associated with a bearing failure at the bolt hole. This failure mode was evaluated using Chapter J, Section J7, of the AISC Specifications and was found to be significantly less than the specified service design load of 27 kips.

The weld strength connecting the steel pipe and ½-inch cap plate was checked consistent with Chapter J, Section J2.4, of the AISC Specifications. Since the exact details for the weld size and type of electrode used were not specified, it was assumed that a ⅛-inch fillet weld with E70 electrode was used. This is in accordance with what would typically be prescribed based on guidance from the AISC Specifications, considering the pipe wall thickness of ⅛ in. Calculation results indicated that the maximum service load permitted based on allowable weld stresses was about half the specified design service load of 27 kips.

Field Modified Weld Connection

Evidently, due to constructability issues related to variable shaft cutoff elevations, the pre-drilled holes in helical anchor shafts were not at the required theoretical design elevation. This required the original pin-connected anchor cap design to be abandoned for a welded connection. In the modified connection, the square anchor shaft was inserted through a hole cut in the ½ inch thick plate of the anchor cap and then welded directly to it.

This modification changed the load path such that the last six failure modes discussed in previous section, are replaced by a single potential failure mode governed by the strength of the weld between anchor shaft and end plate. The weld strength was evaluated following the guidelines of Chapter J, Section J2.4, of the AISC Specification. Based on field observations of this weld and consistent with recommendations from the AISC Specification, the weld was assumed to be a 3/16 inch fillet weld using an E70 electrode.

When the weld is considered to be a fillet weld, the allowable tensile force permitted was found to be approximately 62% of the specified design service load of 27 kips and much less than the maximum design service load calculated in Section *Design Load of an Individual Anchor* above.

A photograph of the actual cap assembly, Fig. 4, shows this weld not to be a true fillet weld, but rather it resembles a partial penetration butt weld. In any case, the photograph shows that the weld quality was not consistent with a quantifiable weld procedure and any strength calculations for this weld are somewhat speculative. Allowable design values calculated using the AISC Specifications are based on quality welds made by certified welders. The welds observed in the field were not consistent with good weld quality and therefore could be expected to have strengths less than that calculated by the AISC Specifications.

Consideration of Anchorage Failure Hierarchy

As discussed in previous sections, even if the anchor cap connection had been constructed as originally designed, it still would have been vulnerable to possible failure because it possessed inadequate design strength for a variety of other failure modes. A hierarchy of failure modes for the original design based on calculations consistent with ACI 318 and Thirteenth Edition AISC Specification procedures, listed in ascending order starting with the mode possessing the least resistance to tensile force is:

1. Bearing at the bolt hole
2. Concrete breakout
3. Weld between the cap plate and steel pipe
4. Yielding of the gross section of the steel pipe
5. Rupture of the net section of the steel pipe
6. Shear rupture of the bolt
7. Shear rupture at the pin-connection
8. Anchor pullout

As a result of the modified as-built anchor cap connection, the calculated hierarchy of failure modes is as follows:

1. Concrete breakout
2. Weld between anchor shaft and ½ inch steel plate
3. Anchor pullout

It should be noted that the concrete breakout strength is based on a 28-day concrete compressive strength of 4,000 psi though it is anticipated that the actual concrete strength achieved was higher. Furthermore, as mentioned in previous section the weld quality between the anchor shaft and cap plate is of poor quality and likely to exhibit less strength than predicted by the AISC Specification calculations. These two factors offer an explanation as to why calculations indicated that concrete breakout could have occurred prior to weld failure in the as-built cap connection, contrary to the observed weld failure mechanism.

SUMMARY

The original design documents for the pool slab anchorage system showed a single pin connection linking helical anchor shafts with the cap assembly needed to transfer uplift forces between the pool's bottom slab and helical anchors. This connection was field modified to a welded connection, presumably to correct a constructability issue resulting from variable helical anchor shaft cutoff lengths, which made the original pin-connection impossible. Pool slab uplift failure was a direct result of the complete fracture and separation of the weld used in the modified connections. However, there were a number of other concerns and a potential failure hierarchy revealed during analysis of other possible failure modes associated with the anchor cap assembly. Structural concerns were prevalent in both the original design as well as in the modified design.

Uplift forces calculated for the original design were found to be non-conservative, particularly when the groundwater table fluctuation criteria reported in the project geotechnical report are considered. Moreover, the service design load of 27 kips indicated on the structural drawings was higher than the 20 kips capacity for the pile cap connection provided in the Hubbell/Chance literature.

ACKNOWLEDGMENTS

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