November 15, 2018 (Revised November 28, 2018)  
Project No. 20190449.001A

Black & Veatch  
5 Peters Canyon Road, Suite 300  
Irvine, California 92606

Attn: Jeff Neemann

Subject: Revised Letter of Supplemental Recommendations  
Proposed Communication Tower  
Proposed Perris II Desalter Facility  
Riverside County, California


Dear Mr. Neemam:

This letter has been prepared to provide supplemental recommendations for the proposed communication tower located at the Proposed Perris II Desalter Facility located in Riverside County, California. This letter has been revised to provide additional clarity for foundation design and construction.

The following letter provides a reference to our previous geotechnical report, summary of our background with the project, summary of our current field investigation and laboratory testing, and supplemental recommendations for design and construction of the proposed project. The conclusions and recommendations provided in the listed geotechnical report (referenced above) remain applicable to the proposed development unless otherwise presented herein.
PROJECT DESCRIPTION

The project site is located on the west side of Murrieta Road in the City of Menifee, California. The project site is bounded by Murrieta Road on the east, the Salt Creek Channel on the south, an existing Eastern Municipal Water District (EMWD) facility on the southwest, and residential homes on the west and north. The project consists of a new EMWD desalter facility with numerous facilities includes tanks, pump stations, chemical storage facilities, buildings, and underground utilities. Recommendations for the new facility were provided in our two referenced reports.

Subsequent to our 2015 and 2018 reports prepared for the project, we were notified by Mr. Brad Sours with Black and Veatch that a new communication tower will be located adjacent to the proposed forebay. Figure 1 shows the approximate location of the new communication tower in relation to other portions of the project as well as our previously performed borings. Based on our discussion with Black and Veatch, we understand that the proposed communication tower is proposed to be approximately 80 feet tall and supported on cast-in-drilled hole (CIDH) pile foundations.

SUBSURFACE CONDITIONS

As noted in our referenced geotechnical reports, subsurface conditions at the site generally consist of artificial fill underlain by alluvial deposits. In the vicinity of the proposed communication tower (Borings KB-01 and KB-02), the artificial fill consists of silty sand to sandy silt and varies in thickness between 1 ½ feet to 4 feet thick. Alluvial deposits underlie the artificial fill and generally consist of varying layers of poorly graded to well-graded sands with varying amounts of silt, clay, and gravel. At a depth of approximately 45 to 50 feet below existing grades (bgs), a weakly cemented clayey sand was encountered to the maximum depth explored of approximately 76 ½ feet bgs.

Groundwater was encountered during our previous investigations. During our recent explorations in May 2018, groundwater was encountered at depths of 23 and 31 ½ feet bgs. During our previous investigations at the site, groundwater was encountered at depths ranging between 22 and 46 feet bgs. However, it should be noted that the groundwater level can fluctuate depending on factors such as seasonal rainfall, groundwater withdrawal, construction activities on this or adjacent properties.
RECOMMENDATIONS

The recommendations presented below are supplemental to the recommendations in our referenced reports. The recommendations in the referenced report remain valid except where updated in the sections presented below.

We have based our conclusions and recommendations on our understanding of the current project, the subsurface investigation described above, our laboratory testing, and our geotechnical analysis.

Liquefaction Analysis

To assess the potential for liquefaction of subsurface soils near the proposed communication tower, we used the liquefaction analysis procedures outlined in Youd et.al. (2001), Seed et.al (2003), and Idriss and Boulanger (2004 and 2008). For estimating the resulting ground settlements, we used the methods proposed by Tokimatsu and Seed (1987), Cetin et.al (2009), and Idriss and Boulanger (2008), respectively. These methods utilize corrected standard penetration test (SPT) blow counts to estimate the amount of volumetric compaction or settlement during an earthquake.

As discussed previously, groundwater was encountered between 22 to 46 feet bgs during our previous investigations. A historic high groundwater of 16 feet was used for our analyses to account for fluctuation in our highest encountered groundwater depth of 22 feet bgs.

According to Section 1803.5 of the 2016 CBC, the PGA used in the liquefaction analysis should be consistent with the maximum considered earthquake (MCE). Based on our analysis, it is our opinion that layers of medium dense poorly to well-graded sands at approximately 20 to 30 feet bgs will be subject to liquefaction in the event of a major earthquake occurring on a nearby fault. We estimate that the liquefaction-induced seismic settlement to be on the order of ½ to 1 ½ inches.

Because of variations in distribution, density, and confining conditions of the soils, seismic settlement is generally non-uniform and serious structural damage can occur due to differential settlement. The amount of differential settlement will depend on the uniformity of the subsurface profile. For uniform subsurface conditions, differential settlement on the order of 50 percent of the total seismic settlement could be expected. For highly heterogeneous sites,
differential settlements on the order of 75 to 100 percent of the total seismic settlement could be expected. For the historic high groundwater level condition, differential settlement at this site may be as much as \(\frac{3}{4}\) inches over a horizontal distance of 50 feet.

**Foundations**

We understand that CIDH piles are proposed to support the proposed communication tower. As noted previously, liquefiable soils were identified at approximately 20 to 30 feet below the ground surface. In order to mitigate the adverse effects of liquefaction including induced seismic settlement and downdrag on the proposed CIDH piles, we recommend that the depth of the proposed CIDH piles be limited to a maximum depth of 15 feet bgs. Limiting the foundation depth to 15 feet bgs is recommended so that the proposed CIDH pile tip elevations are not founded in liquefiable soils. Provided the proposed communication structure can tolerate the estimated liquefaction induced seismic settlements (e.g. \(\frac{1}{2}\) to 1 \(\frac{1}{2}\) inches), CIHD piles with a maximum embedment depth of 15 feet bgs or less will not experience downdrag loading and may be designed as recommended below.

**Axial Capacity**

The compressive axial capacity of drilled piles may be estimated based on an average allowable skin friction capacity of 300 pounds per square foot. We recommend that the axial capacity of piles use skin friction only, and end bearing (tip resistance) should be ignored. The upper one foot of the skin friction capacity should be ignored. The uplift capacity may be estimated as 70 percent of the allowable compressive axial capacity. A one-third increase in the allowable capacities may be used for transient loading conditions such as wind or seismic loads.

**Lateral Resistance**

The drilled pile foundations lateral resistance can be designed in general accordance with Section 1807.3 of the 2016 CBC. We recommend a passive lateral soil bearing pressure of 250 psf per foot of depth below grade in the upper 15 feet. The total lateral soil bearing pressure should not exceed 2,500 psf per pile. Since drilled piles will act as isolated pole foundations, the allowable lateral soil bearing pressure may be increased by a factor of 2 for short-term lateral loads provided the structure will not be adversely affected by up to \(\frac{1}{2}\) inch of lateral movement at the ground surface.
Settlement

Static settlement of the proposed CIDH piles supporting the proposed communication tower on drilled piles is estimated to be less than ½ inch.

Group Effects

The estimated lateral capacities presented previously for deep foundations are for single piles and do not consider a reduction for group action. Piles in groups may be considered to act individually when the center-to-center spacing is greater than 3 pile widths in the direction normal to loading and 8 piles widths in the direction parallel to loading.

Construction Considerations

The performance and capacities of piles can be influenced significantly by the selected construction methods and procedures used. Construction methods that create large zones of disturbance around the drilled shafts can lead to lower than expected skin friction due to excessive stress relief around the shaft length. Drilling of the pile shafts should be accomplished using conventional heavy-duty excavation equipment maintained in good condition.

The on-site soils are granular and relatively non-cohesive and caving of the pile shafts could occur. Temporary steel casing, or other methods to stabilize the sides of the pile shaft, should be employed by the contractor to mitigate caving of the pile shaft. The reinforcing cage and concrete should be placed as soon as practical after drilling of the hole is complete. The concrete should be pumped to the bottom of the drilled shaft using a down hole tremie. If steel casing is used, the casing should be removed as the concrete is placed but the bottom of the casing should be kept at least 5 feet below the top of the concrete.

Maintenance of the full design cross-section for the entire pile length is a concern when casing is extracted during pile casting. Sometimes the suction created by extracting the casing allows soil intrusion into the shaft resulting in reduced pile cross-section. The contractor should employ methods during construction to reduce or eliminate the potential for soil intrusion during casing extraction. Due to the concern of soil intrusion, post-construction evaluation of the piles using
non-destructive testing should be performed. We recommend that each pile be subjected to Gamma-Gamma testing. During construction, plastic tubing should be installed within the rebar cage of each pile to facilitate pile integrity testing and evaluate if the piles are defective (e.g. have a reduced section due to soil intrusion). If a pile is determined to be defective, the pile section should be evaluated by the structural engineer and replaced as necessary. The structural engineer should detail the number and location of CIDH pile inspection tubes placed.

We recommend that the CIDH pile drilling be observed by a representative of Kleinfelder in order to confirm that proper materials are encountered. This will allow us the opportunity to compare actual subsurface soil condition with those encountered during the field exploration, if warranted due to unanticipated subsurface conditions.

LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder’s profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by the Client and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than one year from the date of the report.

The work performed was based on project information provided by Client. If the Client does not retain Kleinfelder to review any plans and specifications, including any revisions or modifications to the plans and specifications, Kleinfelder assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, Client must obtain written approval from Kleinfelder’s engineer that such changes do not affect our recommendations. Failure to do so will vitiate Kleinfelder’s recommendations.
CLOSING

We appreciate the opportunity to be of services. Please contact the undersigned at (951) 801-3681 if you have questions regarding this submittal.

Respectfully submitted,

KLEINFELDER, INC.

Zachary S. Jarecki, PE  Jeffery D. Waller, PE, GE
Project Engineer  Senior Geotechnical Engineer

Attachments:  Figure 1 – Site Plan
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