Appendix F

Geotechnical Investigation

FOUNDATION TYPE SELECTION REPORT
SMUD SOLAND WIND FARM, PHASE 4
MONTEZUMA HILLS ROAD
SOLANO COUNTY, CALIFORNIA

PROJECT NO.: 20193088.001A

FEBRUARY 1, 2019

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Project No. 20193088.001A

Sacramento Municipal Utilities District
4401 Bradshaw Road, Mailstop EA301
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Attention: Mr. Kevin Hudson, e-mail: Kevin.Hudson@smud.org
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SUBJECT: Wind Turbine Foundation Type Selection Study
Solano Wind Project Phase 4
Montezuma Hills Road, Solano County, California
Task No. 10058745, Rev. 0, Contract No. 4500112309

Dear Mr. Hudson, et al:

The attached report presents the results of Kleinfelder's foundation type selection evaluation for the proposed Vestas V150 wind turbine structures to be constructed as part of the subject project. This report describes the study, findings, conclusions, and recommendations for use in the planning and preliminary design of the proposed wind turbine structures.

Kleinfelder appreciates the opportunity to provide geotechnical engineering services to SMUD during the preliminary design phase of this project. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully submitted,

KLEINFELDER, INC.

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APPENDIX

1 INTRODUCTION

This report presents the results of a foundation type selection study conducted for the proposed Solano Wind Farm, Phase 4 project to be constructed in the Montezuma Hills area of Solano County, California. The purpose of this study was to review existing geologic and geotechnical information for the site area, review wind turbine foundation demands from Vestas, evaluate suitable foundation types for the proposed wind turbine structures, evaluate approximate costs of appropriate foundation systems and provide and scope of work and cost estimate for a preliminary geotechnical investigation that can be used by the wind farm developer in their preparation of a design-level scope of work and cost estimate.

Conclusions and recommendations presented in this report are based on the subsurface conditions indicated in the reviewed geotechnical reports, our review of site geologic data, and our understanding of the project details. Conclusions and recommendations contained in this report are preliminary in nature and are suitable for project planning and preliminary design only.

1.1 PROPOSED CONSTRUCTION

Kleinfelder's understanding of the project is based on information received from SMUD for previous wind farm projects in the Montezuma Hills area, our previous work at the site, and a site meeting with representatives of SMUD on December 6, 2018. The proposed improvement areas are shown on Figure 1, Site Location Map.

It is understood the project will involve the design and construction of infrastructure required to install 19 new Vestas V150 4.2MW wind turbine generators (WTG's) on two separate parcels. These WTG's will have a rotor diameter of 150 meters (about 492 feet), a hub height of 105 meters (about 345 feet), and a total height of about 180 meters (about 591 feet). The tower base flange outer diameter will be about 4.55 meters (about 15 feet). According to Vestas, the limiting ground line rotation for the tower bases is 0.17 degrees. This will require a foundation system that can resist very large overturning moments with minimal settlement or tilt.

1.2 PURPOSE AND SCOPE OF SERVICES

As requested by SMUD, we have prepared this report to provide a preliminary evaluation of suitable foundation types for the proposed V150 wind turbine structures and to provide preliminary geotechnical engineering recommendations in support of final design efforts to be performed by a wind farm developer. Our scope of work was outlined in our proposal dated December 21, 2018 and included the following:

- Review of available geologic, geotechnical and wind turbine structure information in the area of the proposed wind turbines
- Research publicly available information and reach out to wind energy providers to obtain information of types of foundations in use for 100-meter hub height or similar wind turbines
- Evaluate appropriate foundation types for the new V150 wind turbines including preliminary engineering analyses
- Evaluate approximate costs of viable wind turbine foundation systems
- Evaluate disturbed areas and volume of soil displaced during fountain construction
- Prepare a Wind Turbine Foundation Type Selection report. The report will include the following:
    - A discussion of the project and its objectives
    - Discussion of the anticipated geologic and geotechnical conditions at the wind turbine sites
    - Discussion of wind turbine foundation types evaluated
    - Results of engineering analyses regarding suitable foundation types
    - Discussion of constructability issues associated with each appropriate foundation type
    - Results of the cost analysis of foundation types considered
    - Evaluation of disturbed area and volume of soil displaced during foundation construction

- Based on the results of the Wind Turbine Foundation Type Selection study, provide a scope of work and cost estimate for the requested Subtask 2 geotechnical study based on the foundation type (s) recommended in the Foundation Type Selection study.

2 DOCUMENT REVIEW

The following documents were reviewed in the preparation of this report.

1. Civil Works Specification, Vestas Document No. 0006-5131 V09, Type T09 – General Specification, dated December 19, 2017
2. Vestas PQ data sheet V105-V136 50Hz
3. Vestas Document No.: 0067-7060 V01 2017-12-01 General Description 4MW Platform
4. Vestas Document No.: 0067-7067 V08 2017-12-21 Performance Specification V150-4.0/4.2 MW 50/60 Hz
5. Vestas Document Number 0069-2467 Ver 00. Tower Foundation Design Basis V150-4.0/4.2MW, 105-Meter Hub Height IEC IIIB, dated 09/11/2017
6. Vestas Document Number 0068-5792 Ver 00. Preliminary Foundation Loads, V150 4.0 MW IEC3B & 4.2 MW IEC S, Mk3E, 105. Dated 08/07/2017.
7. Vestas Document Number 0068-6714. T966900 Tower Drawings. Version 1 Created 08/18/2017 and Reviewed 02/27/2018.
8. "Geotechnical Investigation, Proposed Communications Facility, SMUD Met Tower, 7162 Montezuma Hills Road, Solano County, California," prepared by BMI dated May 15, 2009.
9. "SMUD Solano Wind Project - Phase 1, Montezuma Hills Road, Solano County, California, Geotechnical Engineering Study," prepared by Youngdahl Consulting Group, Inc., dated October 28, 2002.
10. "SMUD Solano Wind Project - Phase 2B, Turbine Site Re-evaluation, Toland Road, Solano County, California," prepared by Youngdahl Consulting Group, Inc. dated October 15, 2007.
11. "SMUD Solano Wind Project Phase 3, Montezuma Hills, Solano County, California, Final Geotechnical Engineering Study," prepared by Youngdahl Consulting Group, Inc. dated November 26, 2007.

12. "SMUD Solano Wind Project - Phase II, Montezuma Hills Road, Solano County, California, Geotechnical Engineering Study," prepared by Youngdahl Consulting Group, Inc. dated May 27, 2004.

13. "SMUD Solano Wind Project - Turbines V47-16 Through V47-22, Montezuma Hills Road, Solano County, California, Geotechnical Engineering Study," prepared by Youngdahl Consulting Group, Inc. dated August 23, 2006.

3 SITE CONDITIONS

3.1 SITE DESCRIPTION

The Phase 4 of the Solano wind project includes 2 areas (west and east) within the present wind farm development as shown on Figure 1, Site Location Map. The locations of the proposed V150 wind turbines within these areas are shown on Figures 2 and 3, Site Map - East Area and Site Map - West Area. The site areas consist of rolling hills with the wind turbines located on the ridge tops. The hills and ridgetops are covered with volunteer grasses and/or grain crops. Trees and brush do not exist within the proposed the development areas.

In the Phase 4 east area, existing Vestas V90 wind turbines are located on the ridge tops along the northwestern portion of the parcel. The proposed V150 wind turbine sites are at elevations ranging from approximately 110 to 195 feet above mean sea level (according to Google Earth). Access roads are in place to many of the ridge tops except at the southeastern portion, located to the east of Montezuma Hills Road, that is presently undeveloped.

In the Phase 4 west area, access roads are generally lacking, and the area is largely undeveloped. The proposed V150 wind turbine sites are at elevations ranging from approximately 96 to 237 feet above mean sea level (according to Google Earth).

3.2 SUBSURFACE CONDITIONS

The following description provides a general summary of the subsurface conditions encountered during explorations in the area for previous projects. Specifically, boring logs from the Phase 1 and Phase 3 reports are either on site or adjacent to the subject sites. It should be noted that even though many of the boring logs call out silts, the laboratory Atterberg Limits tests on many of those soil units reflect lean clay properties, not silt.

Boring logs on and near the Phase 4 east area typically show interbedded lean clays, silts (likely lean clays), and silty, clayey and poorly-graded sands. The sandy soils are typically medium dense to very dense and the silts and clays are medium stiff to hard. Occasional zones of weak to moderate cementation were observed on some of the soil layers. Boring Logs just east of the

Phase 4 west area generally indicate thick silt (likely lean clay) layers, with interbeds of dense sands. These conditions are the basis of our preliminary engineering analyses for this report.

3.3 GROUNDWATER

Free groundwater was not encountered in the borings drilled at the site. This is likely due to the fact that they were located along ridgetops that are well above nearby bodies of water or potential groundwater recharge areas. Perched groundwater may exist at the interfaces between the surficial soils and the denser/stiffer soils at depth. This condition is evident in some of the ridge areas where small landslide scarps are present on the mid to lower flanks of the hillsides.

It is possible that groundwater conditions at the site could change due to variations in rainfall, groundwater withdrawal or recharge, construction activities, well pumping, or other factors not apparent at the time the explorations were performed.

3.4 VARIATIONS IN SUBSURFACE CONDITIONS

Our interpretations of soil and groundwater conditions at the site are based on the conditions encountered in the borings drilled for previous projects in the site area. The conclusions and recommendations that follow are based on those interpretations.

4 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical standpoint, the proposed Vestas V150 wind turbine construction is considered feasible. Additional geotechnical engineering evaluation will be needed to develop geotechnical parameters for final design of the foundations, and the soil types and densities at each tower location will likely vary. However, we have evaluated suitable foundation types for these structures that limit the depths of excavation needed, allow for improvement of the bearing soils in areas where needed, and provide a consistent approach to construction regardless of the variations in foundation soil conditions. A critical aspect of the foundation design is the tight limit on differential settlement which may not result in a foundation rotation in excess of 0.17 degree. The foundation also must meet various other design requirements such as limit states related to bearing capacity, stiffness, strength and fatigue of various materials.

As the scope of this work is limited to preliminary foundation system selection and sizing, detailed verifications of strength and fatigue limit states were not carried out. Steel reinforcement quantity estimates and concrete and grout strengths were selected based on basic strength checks and our experience with similarly-sized wind turbine structures.

The following sections discuss typical wind turbine generator (WTG) foundation design considerations and our evaluations along with our conclusions and recommendations with respect to suitable foundation types, estimated construction costs and estimates of the volume of soil disturbed during foundation construction operations.

4.1 DESIGN CONSIDERATIONS

Per accepted industry standards and guidelines, WTG foundations are designed to meet three types of limit states, which include serviceability limit states such as those related to settlement, tilt and stiffness, ultimate limit states such as those related to bearing, sliding and material strength, and fatigue limit states. Design load cases are prepared per International Electrotechnical Commission (IEC) 61400-1 by the WTG manufacturer, in this case, Vestas. These load cases include operational load cases to be considered for serviceability limit states, extreme load cases (normal and abnormal) to be considered for ultimate limit states and fatigue load cases typically provided as mean fatigue loads or load spectra in the form of Markov matrices. The load factors associated with these different types of load cases are specified in IEC

61400-1. The material resistance factors are governed by national material standards such as ACI for concrete, AISC for steel, etc. In the US, a recommended practice document was prepared by ASCE and AWEA in 2011 to guide practitioners in the selection of material resistance factors and load combinations appropriate to wind turbine generators and compatible with US standards (ASCE/AWEA, 2011).

4.1.1 Load Cases Provided by WTG Manufacturer

For this project, two load documents (Ref. [5] dated 9/11/2017 and Ref. [6] dated 8/7/2017) were provided for the Vestas V150 4.0/4.2MW WTG at 105-meter hub height. Both load documents are preliminary, non-project specific, not for construction, and were prepared per Edition 3 of IEC 61400-1. Extreme and operational load magnitudes were comparable in both documents and the discrepancies between the documents is negligible at this stage of the project. For fatigue loads, the difference in load documents is not evident and cannot be verified without running fatigue calculations. Furthermore, it should be noted that the earlier version of the load document (Ref. [6] dated 08/07/2017) calls for inclusion of an uncertainty in loads of ±10%. The later version (Ref. [5] dated 09/11/2017), however, does not call for this uncertainty provision. Thus, the loads from the later version have been adopted for this study.

Typically, load cases provided in WTG load documents are the top several cases from load simulations and are categorized into extreme cases (normal extremes and abnormal extremes), select operational level loads and fatigue load cases. In this project, the load documents provide the top load case for each of the load components (moment, axial, horizontal and torsion). Such information is provided for the normal extreme load cases in Table 2.1 of the load document. A partial safety factor on normal extreme load cases is 1.35. Abnormal extreme load cases are given in Table 2.3 of the load document and partial safety factor of 1.1 applies to these load cases. Table 2.2 gives the top load cases sorted without consideration of the partial safety factor; i.e., this table shows the top characteristic (unfactored) load case for each load component from both normal and abnormal extreme cases.

Normal production load cases are given in Section 3 of the load documents provided by Vestas. Per industry practice and recent updates to standards (e.g. IEC 61400-1 Edition 4 PRV), three operational load levels are provided:

- S1: A characteristic normal operation extreme, in this case coinciding with DLC 1.4 given in Table 2.1 of the load document,

- S2: “Frequent” load level: this load level corresponds to an annual probability of exceedance of 10^{-4}; i.e., a load level that will be exceeded 0.01% of the time. This is equivalent to a period of discontinuous exposure to higher loads of 0.87 hour per year (17.5 hours over a 20-year design life),

- S3: “Quasi-permanent” load level: this load level corresponds to an annual probability of exceedance of 10^{-2}; i.e., a load level that will be exceeded 1% of the time. This is equivalent to a period of discontinuous exposure to higher loads of about 87 hours per year (or a total of 1,752 hours, or 2.5 months, over a design life of 20 years).

The above operational load levels are used for verifying various serviceability limit states per accepted industry practice, as outlined in subsequent sections.

4.1.2 Consideration of Seismic Loading

The project site is located in an area of high historic seismic activity. Per industry practice, the load cases provided in the load document prepared by the WTG manufacturer do not include a seismic loading that is appropriate for the site and structure. This is typical since the seismic loading is dependent on the location-specific seismic input, site geotechnical characteristics and the structural response that is dependent on the structure and soil-structure interaction. The evaluation of the seismic loading is typically left to design stages when the specific location of the turbine is identified, and the geotechnical information is also available. Industry practice, as standardized in IEC61400-1, involves superposing the seismic loading estimated for the site and structure with the larger of several operational load situations that should be provided in the load document by the WTG manufacturer. Such superposition, as specified in IEC 61400-1, should be accomplished using unity for all load and material safety factors. Per IEC61400-1 Edition 4 PRV, these load situations include:

- Mean loads during normal power production determined at rated wind speed, V_r (defined as the “minimum wind speed at hub height at which a wind turbine's rated power is achieved in the case of steady wind without turbulence.”)

- Loads during emergency stop at rated wind speed, V_r

- Loads during idling or parked condition at no wind and cut-out wind speed, V_{out} (defined as the “highest 10 min average wind speed at hub height at which the wind turbine is designed to produce power in the case of steady wind without turbulence.”)

The preliminary load documents provided by Vestas did not include these specific load cases that need to be combined with the seismic loading. For the purpose of our preliminary assessment of the seismic load combination, we considered DLC 1.4 as the S1 load case and we combined it with the seismic load estimate per the recommendation found in ASCE/AWEA RP 2011. For these assumptions, we found that the seismic load combination did not govern but was very close to governing. Site specific seismic parameters were estimated per ASCE 7-10 using available geotechnical data from the earlier project phases. Two sets of parameters were estimated for the east and west areas, both producing a Seismic Site Class D per ASCE 7-10, but with the west area parameters producing the higher seismic loading. Seismic loading estimate was also facilitated by using the tower dimensional and weight information provided by Vestas (Ref. [7]). It should be noted that the full geotechnical exploration planned for this project should include a focus on accurate assessment of the Seismic Site Class at each WTG location. A Seismic Site Class worse than D would have resulted in the seismic load case governing the design of the foundation.

Finally, it should be noted that it is common to assign ASCE 7-10 Occupancy Category II to wind project installations, instead of Occupancy Category III that is common for classic energy generating stations and power plants. This assumption results in an importance factor of 1. The rationale behind this assumption, explained in ASCE/AWEA RP2011 Section 4.4, is that wind farms do not provide continuous power at consistent levels and are not typically relied upon for essential or emergency response facilities. If SMUD is intending to rely upon this wind project as an Occupancy Category III facility, the seismic load estimate would need to be updated and it is the more likely to govern the design.

4.1.3 Serviceability Limit States

Serviceability limit states that must be verified for any type of foundation system under consideration. These include settlement (total & differential) and foundation stiffness. Differential settlement or tilt is limited to 0.17 degree (3 mm/m) per the WTG load documents. Per accepted industry practice and draft international standard IEC 61400-6, the settlement/tilt limit state is verified using the S3 operational load level.

The WTG load document also specifies the minimum foundation stiffness required to maintain the validity of the quoted loads. The minimum stiffness is specified as acceptable pairs of rotational and lateral stiffness values. One of the acceptable pairs is a rotational stiffness of 90 GN*m/rad and a lateral stiffness of 10 MN/m. This pair has been adopted in this preliminary foundation system selection. Furthermore, although not clearly mentioned in the load document, this pair is understood and is treated as a pair of dynamic stiffnesses evaluated at the S3 load level where positive contact pressure is maintained over the entire foundation/subgrade interface. It is increasingly common for WTG manufacturers to also specify a static stiffness requirement evaluated at higher load (and strain) levels where part of the foundation can be allowed to be at zero contact pressure. For this check, we adopted static stiffness values that are half the dynamic stiffness values.

4.1.4 Zero-Pressure and Pile Tension Allowances

Accepted industry practice and certification agency standards require positive (compression) contact pressure at the foundation/subgrade interface at the S3 load level and allow for up to half the foundation width to be at zero contact pressure for the S1 load level. These criteria have been observed in this preliminary foundation sizing exercise. For the piled raft option where drilled shafts are placed around the perimeter of a circular raft or cap, it is common to avoid tension in these shafts at operation load levels S3 or S2. The objective of this practice is to avoid cyclic degradation of shaft uplift capacity. For this purpose, a cyclic stress ratio is computed based on very low mobilization of drilled shaft capacities (in download and uplift). The cyclic stress ratio must be less than one. The choice between S2 and S3 would depend on, among other things, the type of soil and its susceptibility to cyclic degradation.

4.2 FOUNDATION TYPES CONSIDERED

Considering the sizes of the V150 towers (15-foot-diameter tower bases), their large overturning moments (nearly 90,000 foot-kips), the dynamic nature of the loading and the anticipated ground conditions, we have considered the following foundation systems:

1. Patrick and Henderson foundations 18 feet diameter by 70 feet deep

2. Gravity base option: Round reinforced concrete mat foundations bearing on native soils at a depth of about 10 feet. This option includes consideration of ground improvement at softer locations where bearing capacity needs to be increased or settlement needs to be reduced. The ground improvement methods could include compacted aggregate piers, rigid inclusions (unreinforced concrete columns) or over-excavation and re-compaction or replacement with structural fill.

3. Piled raft option: A round pile cap supported by a ring of reinforced concrete drilled piers

No options involving rock have been considered since bedrock is not anticipated to be within the depths of consideration. In addition, driving piles in the stiff soils at the site is not considered feasible. Therefore, driven pile foundations were not considered. Our comments regarding the design, construction and cost issues associated with these options are discussed below.

4.2.1 Patrick and Henderson Tensionless Pier Option

The Patrick and Henderson tensionless pier foundation (a cylindrical, hollow, tensionless pier constructed within two corrugated steel casings) of the size needed for these structures would require a very large diameter drilling rig to drill a shaft of approximate diameter of 18 feet to a depth of about 70 feet. The shaft would still need to be filled with soil in the center and flowable fill or grout material will be needed for the space around the outer casing. There are poorly-graded sand layers at many of the tower areas that may be subject to collapse and cave-ins in near vertical excavations. Therefore, such drilling and foundation construction operations are expected to be fraught with difficulties and very challenging, particularly in poorly graded sand layers that may contain perched groundwater. Alternatively, an excavation of that depth would likely need to be over 150 feet wide at the top and backfilling would be a major operation. The excavation depth is also larger than the reach of large excavating equipment which would require intermediate staging depths and even larger excavation volumes. This would require a specialized operation to construct the foundation and qualified bidders would be limited. Neither of these approaches appear to be practical for this project. Therefore, while this option was popular in the area for smaller wind turbines, we find that it is not practical for this size turbine and site geotechnical conditions.

4.2.2 Gravity Base Option

For a reinforced concrete gravity base (or mat) foundation bearing on undisturbed native soil, the mat would have to be about 72 feet wide and about 10 feet deep, given the estimated soil bearing capacity derived from existing reports. The mat would be thicker at the center, and it should taper as needed to accommodate the applied moments in the foundation and to optimize on the concrete quantity. A drawing of this foundation system is shown on Figure 4.

Since the tower sites are on ridge tops, it is possible that at the bearing elevation, there could be differential soil conditions (i.e., weak surface residual soils, hard clay or claystone, dense sand or weak sandstone). If the mat is within one consistent and competent soil unit, it may not need any ground improvement beneath it. However, if differential soil conditions or soft materials are found upon excavation to footing subgrade, reinforcing elements such as compacted aggregate piers or unreinforced concrete inclusions could be used to provide more uniform bearing conditions for the mats. These elements are typically constructed by a specialty contractor but are generally much cheaper than reinforced concrete drilled piers. The design of such, often proprietary, elements would likely be provided by specialty contractors. Based on our discussions with a ground improvement contractor, the depths of compacted aggregate piers are anticipated to be on the order of 6 feet below the bottom of the mat, as presently conceived, but will depend on the actual soil conditions present. They indicated that concrete inclusions would be at higher costs and, thus, were not evaluated. Another option to improve the bearing capacity beneath the footings would be to over-excavate the soils to some depth and replace them as compacted fill. Mass concrete construction would not require specialty contractors and many general contractors would be qualified to bid the foundation work.

The gravity base option is the most popular foundation system used in the onshore wind energy industry, especially for large wind turbines. Ground improvement technology extends the range of applicability of this foundation system to softer sites. In our experience, unless the site is extremely soft or extremely competent (good quality rock), the gravity base foundation is a viable option by default. Since most contractors are familiar with this system, pricing it is expected to be relatively accurate, and building the foundation is expected to be common practice.

4.2.3 Piled Raft Option

Given the high seismic loading anticipated for the site and potential foundation deflection issues associated with differential ground support, a potentially stiffer foundation system that should be considered is the piled raft system. This system consists of a group of drilled shafts connected by a cap or mat at their heads. The cap is circular but smaller in diameter than the gravity base system and bears at a similar depth (10 feet in our sizing exercise). The cap is supported around its perimeter by a circle of drilled shafts. The cap tapers towards the edge to optimize concrete quantity but its edge cannot be as thin as the gravity base because of the concentrated pile loads. A drawing of this foundation system is shown on Figure 5.

4.3 PREFERRED FOUNDATION TYPES

Based on the lab testing results from this and the previous studies, the use of reinforced concrete gravity base foundations and those underlain by some form of ground improvement (such as compacted aggregate piers), or a reinforced concrete cap connecting a ring of drilled shafts appear to be the most cost-effective foundations for the V150 structures. Rigid concrete inclusions normally require placement of aggregate base materials above them that can be several feet thick. Although we would not rule out that type of foundation system, the need for additional excavation and fill placement makes that option not as cost effective as compacted aggregate piers, and possibly the piled raft option as well.

The advantage of these foundation types is that most of the construction is typical mass excavation, open hole drilling, and reinforced concrete construction. If ground improvement elements under a gravity base foundation are used, those elements would likely need to be constructed by a specialty contractor and designed using a design-build approach. There are several contractors in the area that provide compacted aggregate pier design and construction. All of the equipment needed for construction can be delivered on typical heavy trucks over the local highways.

4.4 FOUNDATION COST COMPARISON

In evaluating the preferred foundation types for this project, we have performed preliminary cost estimates of the options considered viable. The foundation types and relative costs are shown in the table below. These are costs for constructing 19 foundations consecutively.

Table 4.1
Approximate Foundation Construction Costs

Option	Foundation Type	Element	Subtotal	Rough Cost
A	72-foot dia. gravity base foundation bearing at 10' below existing grade	Earthwork	$81,000/WTG or $1,539,000 for 19	$384,000/WTG or $7,296,000
		Reinforced Concrete	$303,000/WTG or $5,757,000 for 19	
B	72-foot dia. gravity base foundation with 7-foot over-excavation and replacement	Earthwork	$81,000/WTG or $1,539,000 for 19	$430,000/WTG or $8,170,000
		Reinforced Concrete	$303,000/WTG or $5,757,000 for 19	
		Over-ex	$46,000/WTG or $570,000 for 19	
C	72-foot dia. gravity base foundation with compacted aggregate piers	Earthwork	$81,000/WTG or $$1,539,000 for 19	$437,000/WTG or $8,303,000
		Reinforced Concrete	$303,000/WTG or $5,757,000 for 19	
		Aggregate piers	$53,000/WTG or $1,007,000 for 19	
D	Piled raft foundation: 58-foot dia. cap on (18) 3.5-foot dia. x 30-foot-deep drilled piers	Earthwork	$60,000/WTG or $1,140,000 for 19	$452,000/WTG or $8,588,000
		Reinforced Concrete	$257,000/WTG or $4,883,000 for 19	
		Drilled Piers	$135,000/WTG or $2,565,000 for 19	

It should be noted that the costs of gravity base foundation alternatives shown above are relatively close to the piled raft foundation. It is our opinion that the piled raft system should work well in the various soil conditions present at the site without requiring much, if any, modification based on field conditions at each site. If the on-site soils are not found to be stiff enough to make the gravity base alternative cost effective, the piled raft option could be the final design of choice, or at least be a good option for some of the sites.

4.5 EARTHWORK DISTURBANCE

Earthwork quantities were estimated for each foundation type being considered. The table below shows soil quantities based on a shrinkage factor of 0.90 from in situ density to compacted fill density. The approximate earthwork volumes for the various foundation elements are provided in Table 4.2 below in cubic yards (cy).

Table 4.2
Approximate Earthwork Quantities

Earthwork Element	Gravity Base Option	Piled Raft Option
Volume of topsoil	175 cy	135 cy
Volume of excavated material	2365 cy	1750 cy
Volume to be backfilled	2040 cy	1500 cy
Volume of taper at grade	45 cy	20 cy
Volume of Piles	--	200 cy
Backfill imbalance	+275 cy	+230 cy
If improvement is needed under gravity base mat, use one of the following methods:		
In place volume of over-excavation to 7 feet	1275 cy	--
Backfill imbalance considering over-ex re-compaction	+150 cy	--
Volume of ground improvement elements	45 cy	--
Backfill imbalance considering ground improvement	+235 cy	--

The surface area disturbed for excavation is the outer diameter of the sloped excavation, plus about 15 feet to allow for access around the excavation for weatherization, etc. For the 72-foot-diameter gravity base foundation founded 10 feet deep with 7 feet of work space around it gives a disturbed area diameter of about 106 feet. Add the 15 feet around it and that's a diameter of 136 feet. For the 58-foot-diameter piled raft foundation founded 10 feet deep with 7 feet of work space around it gives a disturbed area diameter of about 92 feet. Add the 15 feet around it and that's a diameter of 122 feet. There will need to be ramps to get heavy equipment and truck access the excavated subgrades that would be about 15 feet wide by 200 feet long at a 5 percent grade.

The contractors will likely have laydown areas for WTG components, soil stockpiles and equipment that could be on the order of 400 by 400 feet at each WTG tower site. An additional area adjacent to the foundation that is about 60 by 120 feet would be needed for a crane pad. The estimated area of ground disturbance due to construction in square feet (sf) is provided in Table 4.3 below.

Table 4.3
Approximate Disturbed Areas for Construction

Element	Gravity Base Option	Piled Raft Option
Foundation Excavation	14,527 sf	11,690 sf
Access Ramp	3,000 sf	3,000 sf
Laydown Area	160,000 sf	160,000 sf
Crane Pad	7,200 sf	7,200 sf
Totals	**184727 sf**	**181890 sf**

5 ADDITIONAL SERVICES

5.1 PRELIMINARY GEOTECHNICAL EVALUATION

5.1.1 General Task Approach

As stated in the RFTP, SMUD desires to obtain geotechnical information that is useful for the wind farm developer to plan for a design-level geotechnical study. Existing information is not sufficient for WTG foundation design and a design-level geotechnical study is recommended for each tower site.

Considering that the geology of the site area includes uplifted and folded sedimentary deposits and soft rock, we propose to evaluate the geotechnical conditions at each ridge where the towers are planned. The geotechnical conditions across the tops of each ridge are anticipated to be similar and may vary from sandstone to claystone overlain at variable depths of residual clay soils/decomposed rock. We do not propose to drill each tower site. However, the borings drilled will be located at proposed tower locations that we feel best represent the anticipated conditions along the various ridge tops.

Considering the preferred foundation type(s) developed from this foundation type selection report, we have estimated the depths of exploratory borings that would provide the required subsurface data. At this time, based on our preliminary engineering analyses, we anticipate the borings will need to be on the order of 75 feet deep. In addition, the site soils are relatively stiff/dense. Considering what is needed for a gravity base (mat) foundation, proper soil sampling and laboratory testing is important to evaluate the strength and deformation characteristics of the site soils for foundation design. This will require air or mud rotary drilling methods and the use of a specialized sampler (i.e., Pitcher Barrel Sampler) to obtain relatively undisturbed samples for laboratory consolidation testing. The Pitcher Barrel sampler is designed specifically to recover relatively undisturbed samples from formations that are too hard for thin-wall Shelby direct push samplers or too weak or water-sensitive to permit satisfactory recovery by conventional rock coring methods.

To evaluate foundation stiffness for validation of WTG manufacturer requirements, shear wave velocity of the subsurface soils is an important input parameter. We are recommending performing limited geophysical surveys at 5 of the 10 proposed boring sites using the Multi-channel Analysis of Surface Waves (MASW) approach. This method uses a geophone array about 230 feet long placed on the ground surface, with a shotpoint located in-line with the array. The equipment can be carried by hand if needed and, other than the insertion of the geophone probes, will not disturb the ground surface. The seismic energy is generated through multiple impacts with a 12-lb sledge hammer against a metal plate placed on the ground surface at the shotpoint location. From that data, a seismic model of the subsurface is prepared through refraction and dispersion analysis of the recorded geophone signals. The analysis yields inferred layering of geotechnical strata and the associated shear wave velocity for each layer.

A detailed description of the proposed geotechnical work associated with this project is presented below.

5.1.2 Scope of Work

5.1.2.1 Document Review

Kleinfelder will review pertinent geologic and geotechnical information available in the area of the proposed wind turbines. It is anticipated that the site contains clay soils underlain by interbedded lean clays and sands and poorly-indurated claystone or sandstone bedrock.

5.1.2.2 Field Exploration

The subsurface conditions at the site will be explored by drilling 10 borings with a track-mounted drill rig. The track-mounted rig was selected to minimize damage to the fields when accessing the proposed boring sites and to access the locations during the wet winter and spring months. Boring depths will be at least 75 feet below the ground surface. A professional from Kleinfelder will maintain logs of the subsurface conditions encountered and obtain samples for visual examination, classification, and laboratory testing. In order to obtain relatively undisturbed soil samples for laboratory testing, and due to the site access issues, we propose to use air rotary drilling techniques to drill the borings. We anticipate each boring will take about 1 day to complete.

Prior to initiating our subsurface explorations, all site utilities and utility easements must be accurately located in the field, on a scaled map, or both. Kleinfelder will work with the drilling subcontractor to provide at least 72-hours' notice to Underground Service Alert (USA), as required by law. Since the USA service will not locate utilities on private property or locate existing irrigation lines, we have provided for a private utility locator service to clear the proposed boring sites prior to drilling. We also request that SMUD provide information to Kleinfelder regarding known underground utilities in the proposed tower areas at least one week before beginning our field investigation. Kleinfelder will accept no responsibility for damage caused to existing utilities not accurately located in the manner described above.

Kleinfelder will obtain a Solano County drilling permit prior to drilling the borings. Upon completion of drilling, borings will be grouted closed and capped with a few feet of compacted soil. Excess drill cuttings will be spread out on site and covered with straw in vegetated areas.

5.1.2.3 Sampling Procedures

Soil samples should be collected from the borings at depth intervals of approximately 2½ feet in the upper 10 feet and 5 feet thereafter, or where significant changes in soil stratigraphy are noted. Since the foundation bearing depth is on the order of 10 feet below existing grades, soil samples below that depth will be used to evaluate bearing capacity and deformation characteristics of the site soils.

It is typical for soil samples to be collected from borings using driven samplers advanced with a drop hammer. For samples where disturbance is not a significant issue, the samples will be collected at selected depths by driving either a 2.5-inch inside diameter (I.D.) California sampler, or a 1.4-inch I.D. Standard Penetration Test (SPT) sampler 18 inches (unless otherwise noted) into undisturbed soil. The samplers will be driven using a 140-pound automatic hammer free-falling a distance of 30 inches. Blow counts will be recorded at 6-inch depth intervals for each sample attempt and will be reported on the boring logs. It is important that samples in the upper bearing area of foundations be recovered using methods that create minimal sample disturbance, such as a Pitcher barrel sampler. Those samples are best for triaxial shear strength testing and consolidation testing in an oedometer. We propose to obtain 2 Pitcher barrel samples from each boring in the intervals between about 15 feet and 30 feet.

Soil samples obtained from the borings will be packaged and sealed in the field to reduce moisture loss and disturbance. Following drilling, the samples will be returned to our laboratory for further examination and testing.

Subsurface samples obtained during the field exploration program will be retained for 30 days after the completion of the report. At that time, they will be disposed of. If requested, the samples may be stored for a longer time period or delivered to you (sample storage fees and/or delivery fees may apply).

5.1.2.4 Surface Wave Geophysical Surveys

We will evaluate the shear wave velocities of the subsurface soils at 5 selected boring locations using surface-wave geophysical methods to measure profiles of seismic shear wave velocity versus depth. The multichannel analysis of surface waves (MASW) method will be used to interpret the seismic shear wave velocity profile. At the geophysical exploration locations, we will attempt to gather data that will allow interpretation of shear wave velocity to depths of at least 100 feet.

The geophysical work described above will be conducted by a two-man crew plus a Kleinfelder representative. All equipment used to conduct the geophysical measurements is hand portable. Surface waves are generated by striking the ground surface with a range of hand-carried hammers, and the waves are monitored with geophones that are placed on the ground surface. No drilling or other penetration of the ground surface is done for this activity. The geophysical work will be completed in one 10-hour day. The crew will get on-site training from SMUD prior to doing the work.

5.1.2.5 Laboratory Testing

Laboratory tests will be performed on selected soil samples to evaluate the density, gradation characteristics, corrosion characteristics, strength, and compressibility of the materials encountered during the field investigation.

It is anticipated laboratory testing will include moisture content, dry unit weight, sieve analysis, Atterberg limits, unconsolidated-undrained and/or consolidated-drained triaxial compression tests, and one-dimensional consolidation tests. Selected samples will also be tested for corrosivity (pH, electrical resistivity, water-soluble sulfates, and water-soluble chlorides), which may be used by a qualified corrosion engineer in designing an appropriate corrosion control plan for the project. These tests will not be suitable for use in design of electrical grounding systems. The tests selected and the frequency of testing will be based on the subsurface conditions actually encountered.

5.1.2.6 Engineering Analyses

Engineering analyses will be performed for the preferred foundation type(s) identified during preparation of this Foundation Type Selection report. Engineering analyses will include the necessary calculations to confirm or update the preliminary foundation sizes and quantities estimated in this report. For example, for the mat foundation option, bearing capacity analyses will confirm if the assumptions adopted in this report are still valid. The site-specific geotechnical data will allow for a better assessment of bearing capacity and will provide for a better assessment of any potential ground modification requirements. For the piled raft option, the collected geotechnical information will enable recommendations relative to lateral resistance, drilled pier side friction and uplift resistance, and/or pier group axial, lateral and overturning resistance. We would also be able to estimate settlement and deformation under load for the preferred foundation type(s). In addition to the typical analyses of shear strength and lateral resistance parameters, we have provided for finite-element modeling of the gravity base option to validate the soil-structure interactions, gapping/no-gapping response and bearing pressures estimated in this report based on hand calculations."

5.1.2.7 Report Preparation

Kleinfelder will prepare a preliminary geotechnical report describing the results of the field explorations, laboratory testing, engineering analyses, preliminary foundation design recommendations, and recommendations for a design-level study. The report will include the following:

- A description of the proposed project
- A description of the surface and subsurface site conditions encountered during the field investigation, including depth to groundwater
- A description of the site geologic setting
- A brief discussion of the corrosion potential of the near-surface soils encountered during the field exploration based on laboratory corrosivity tests performed. (NOTE: Kleinfelder's scope does not include corrosion engineering. Therefore, detailed analysis of corrosion test results is not included in this proposal)
- A discussion of engineering analyses performed

- Preliminary recommendations related to the geotechnical aspects of:
    - WTG tower foundation design, including recommended foundation type(s), allowable bearing pressure, lateral and uplift resistance
    - Evaluation of foundation stiffness
    - Evaluation of mat foundation subgrade deformation characteristics using finite-element methods
    - Temporary excavations and excavation backfill
    - Seismic design considerations including 2016 CBC design criteria
    - General soil corrosivity discussion
- Recommendations for design-level geotechnical study
- An appendix that will include logs of borings, MASW survey results, and laboratory test results
- An appendix that will include results of finite-element modeling and foundation stiffness evaluations

5.1.2.8 Environmental Assessment

This scope of work does not include an assessment of environmental characteristics involving hazardous or toxic substances. In the event potentially hazardous materials are identified visually or by odor within the exploratory borings, such borings will be immediately terminated, and arrangements will be made to backfill with cement grout. We will notify you as soon as possible of such an occurrence in order to mutually decide whether to continue, modify, or cease the remainder of the field exploration program. All costs incurred as a result of encountering suspected hazardous materials would be charged on a time-and-expense basis over and above the estimated fee for the geotechnical site investigation.

5.1.2.9 Proposed Exploration Locations

The proposed exploration locations based on the proposed WTG sites are provided in Table 5.1 below.

Table 5.1

Proposed Exploration Locations

Area	Soil Boring at Structure No.	MASW Survey at Structure No.
East	P1N2	
	P1N3	P1N3
	P1R2	P1R2
	P1R5	
West	P4N1	P4N1
	P4N2	
	P4N4	P4N4
	P4N7	
	P4N8	P4N8
	P4N10	

5.2 COST ESTIMATE

Our estimate of fees to complete the scope of work described above is shown in Table 5.2 below. The main differences in this cost estimate versus our previous one was the higher cost of drilling and sampling due to the use of track-mounted air rotary drilling methods needed to get quality samples for laboratory testing, additional laboratory consolidation testing, and geophysical surveys. We have also arranged for an ATV to access the sites for utility locates and geophysical surveys. Our drilling subcontractor and utility locator estimates are based on California prevailing wage rates.

Table 5.2

Preliminary Geotechnical Engineering Study Cost Estimate

ITEM NO.	DESCRIPTION	EST. QTY.	UNITS	BILLING RATE	TOTAL AMOUNT
1	Principal Geotechnical Engineer	69	HRS.	$183	$12,627
2	Sr. Geotechnical Engineer	30	HRS.	$142	$4,260
3	Assoc. Geotechnical Engineer	197	HRS.	$120	$23,640
4	Drafting Technician	38	HRS.	$80	$3,040
5	Travel & Other Expenses	LS	EA.	--	$2,301
6	ATV Rental	LS	EA.	--	$1,600
7	Laboratory Testing	LS	EA.	--	$12,750
8	Drilling Subcontractor	LS	EA.	--	$92,300
9	Utility locating Subcontractor	LS	EA.	--	$2,000
10	Geophysical Subconsultant	LS	EA.	--	$3,500
11	**TOTAL ITEMS 1 THROUGH 11**				**$158,018**

Charges in addition to the scope of work described above will be billed on a time-and-expense basis, based on our current rates. We will notify you of events or conditions that arise that may affect the project schedule or budget during the course of the project.

5.2.1 SCHEDULE

At the present time, we anticipate work could begin on the project immediately following receipt of your authorization to proceed. Scheduling of a drill rig, clearing underground utilities through Underground Service Alert (USA), obtaining drilling permits, and arranging site access is anticipated to take about 2 to 3 weeks to arrange. Therefore, drilling should occur in mid-February. We are planning a 10-day field exploration schedule for the borings and 1 day for the geophysical surveys. We will provide results and verbal recommendations upon request and as soon as they become available. Unless something out of our control affects the project schedule, we anticipate submitting our report by March 29, 2019. We will provide results and verbal recommendations upon request and as soon as they become available.

5.2.2 SITE ACCESS

This proposal assumes the site is accessible with our track-mounted drilling equipment and all-terrain vehicles. Necessary removal of fences or gates, permission to enter the site from the current owner or leaseholder, and/or required use permits (other than the drilling permit discussed above) must be secured by the client prior to the initiating field activities. If weather, access, or site conditions restrict the field operations, additional costs may be incurred to modify the exploration program and the project schedule may be affected. Repeated site visits, and/or attempts to drill at the client's request during periods of difficult access or poor weather conditions will be billed on a time-and-expense basis.

5.2.3 UNANTICIPATED SUBSURFACE CONDITIONS

If subsurface conditions differ significantly from those anticipated above, we may need to revise the scope of work and estimated fee in order to complete the project. Should this occur, we would contact you for authorization before proceeding with any additional work.

5.2.4 LIMITATIONS

Our work will be performed and findings obtained in substantial conformance with the geotechnical engineering practice that exists within the area at the time of the investigation. No warranty is expressed or implied. This proposal includes providing information obtained through laboratory testing regarding corrosivity of onsite soils. Kleinfelder is not providing corrosion engineering in this scope of work. Therefore, detailed analysis of corrosion test results is not included in this proposal. We recommend a qualified corrosion engineer be retained to review the test results and design protective systems that may be required.

6 LIMITATIONS

This report presents information for planning and preliminary design of the proposed WTG foundations at the SMUD Solano Wind Farm Phase 4 development in the Montezuma Hills area of Solano County, California. Recommendations contained in this report are based on information obtained from previous studies in the site area as well as our review of available geologic information and maps, and our present knowledge of the proposed construction. A final design-level geotechnical report should be prepared for the project. Preliminary recommendations contained in this report may differ from the final design.

It is possible that soil conditions could vary beyond what has been encountered in past studies. If the scope of the proposed construction, including the proposed location, changes from that described in this report, we should be notified immediately to review the changes and provide supplemental recommendations.

We have prepared this report in accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty, express or implied, is made.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

7 REFERENCES

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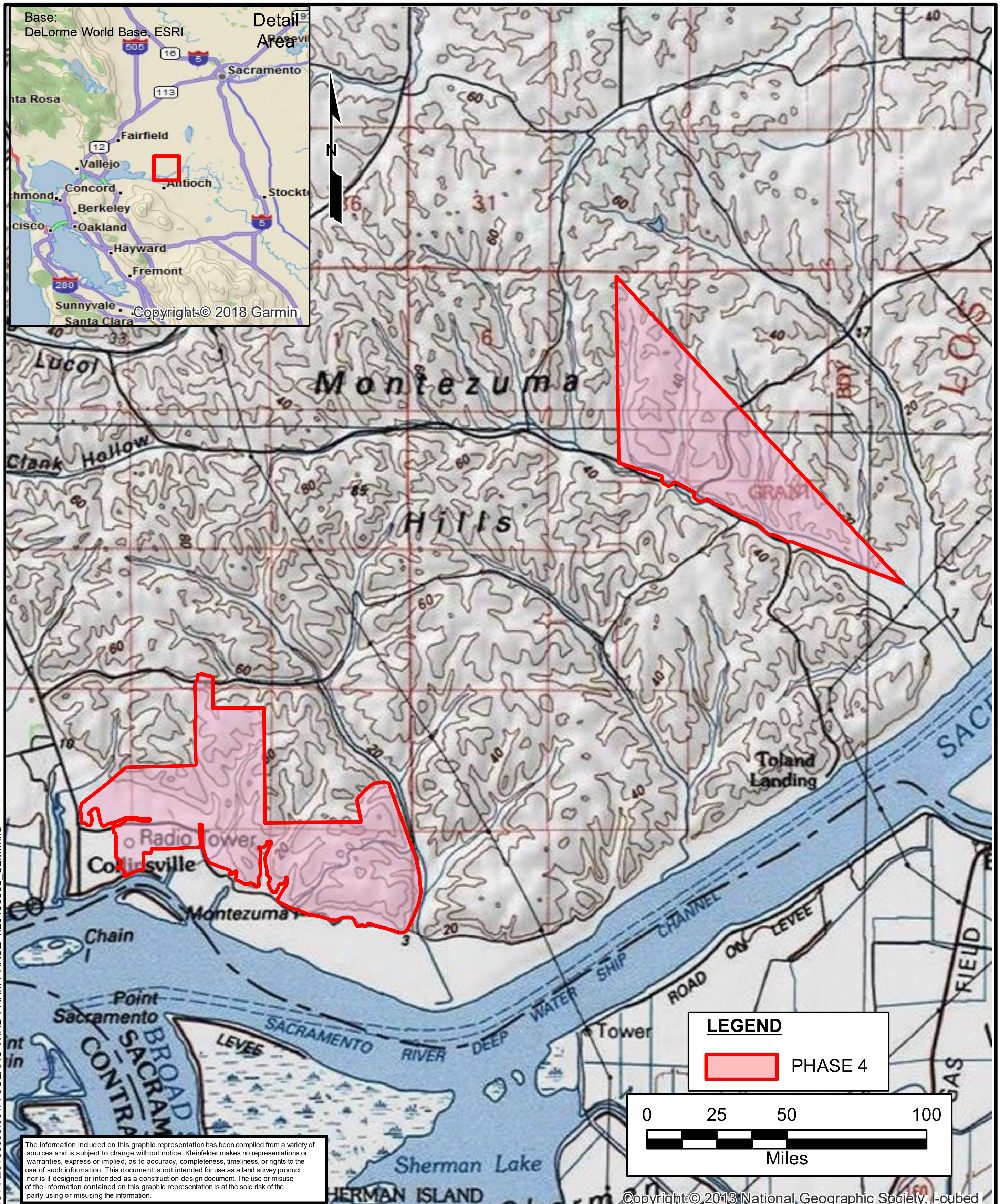
FIGURES



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SITE LOCATION MAP

SMUD-Solano Wind Farm Phase 4
Solano County, California

FIGURE
1

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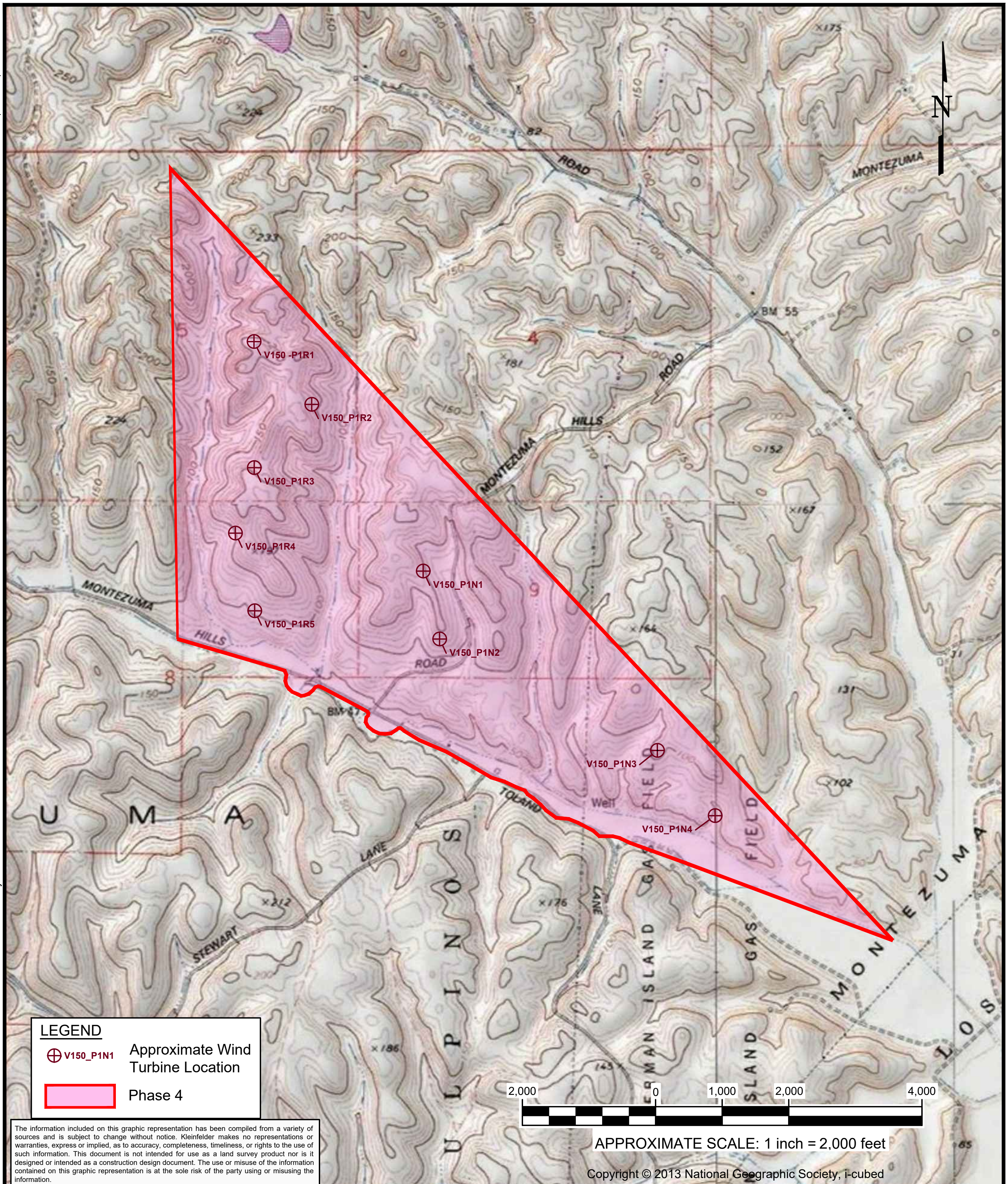

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SITE MAP - EAST AREA

SMUD-Solano Wind Farm Phase 4
Solano County, California

FIGURE
2

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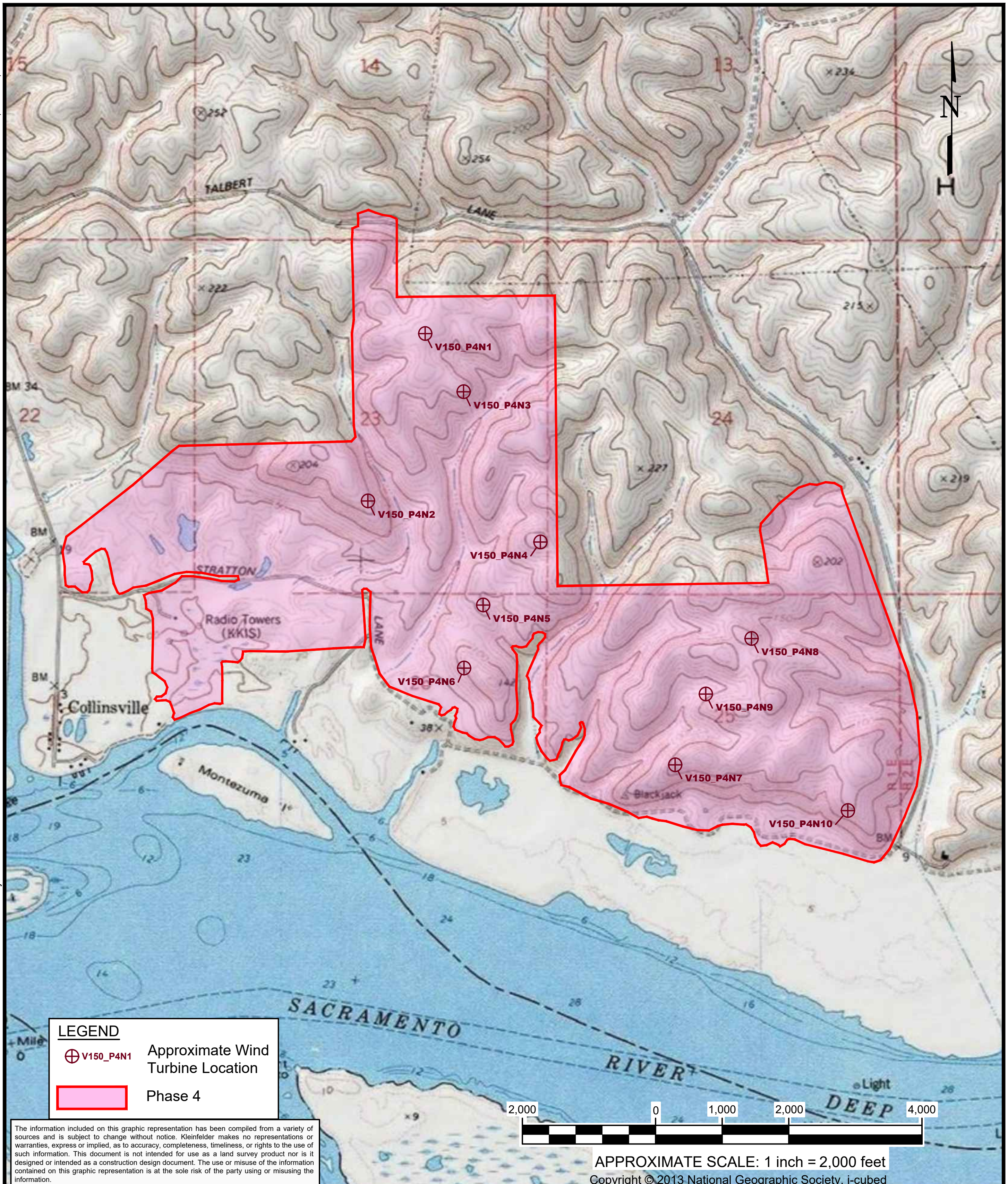

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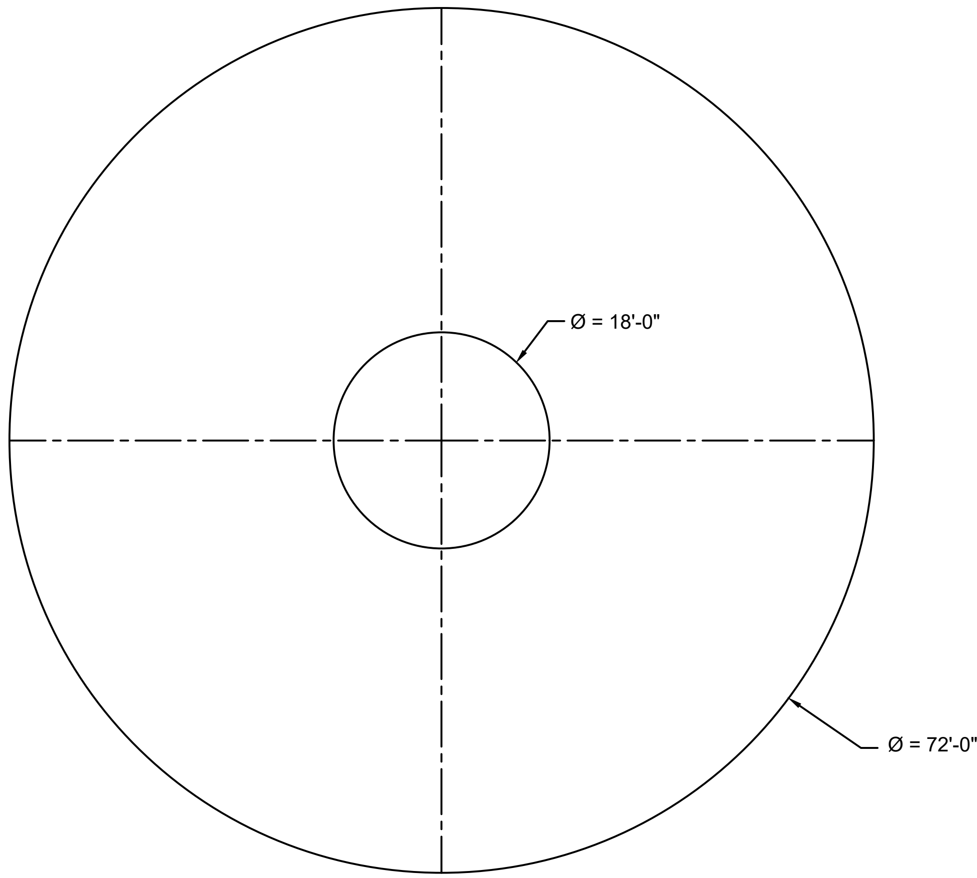
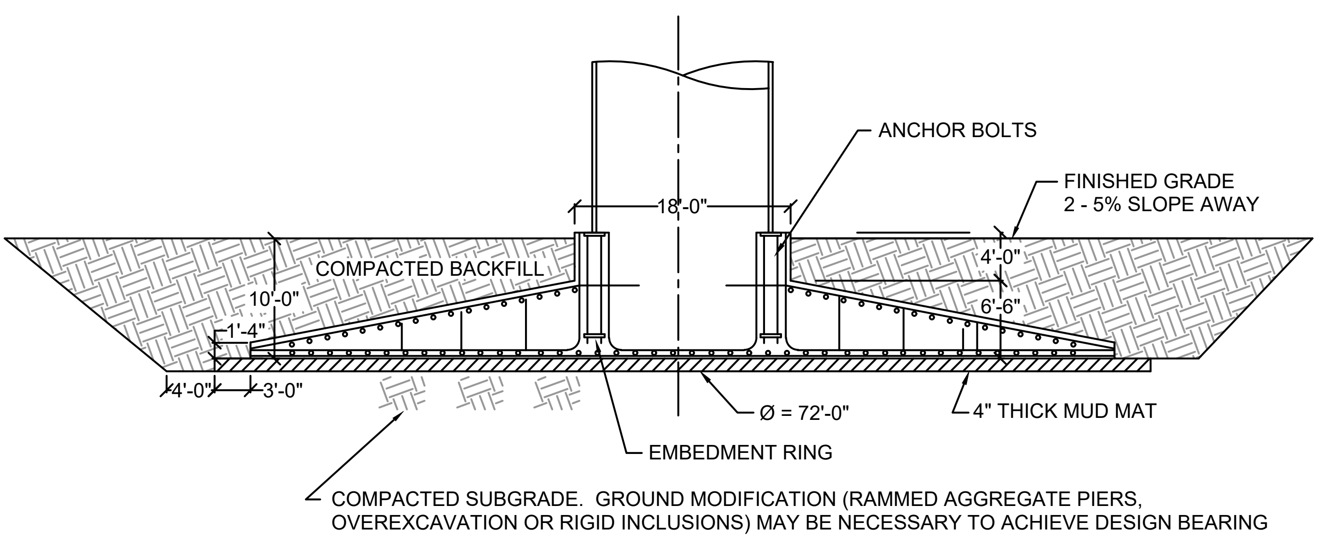
SITE MAP - WEST AREA

SMUD-Solano Wind Farm Phase 4
Solano County, California

FIGURE
3



PLAN VIEW



SECTION VIEW

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Do not scale sketches

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FOUNDATION DRAWING
GRAVITY BASE

SMUD-Solano Wind Farm Phase 4
Solano County, California

FIGURE
4

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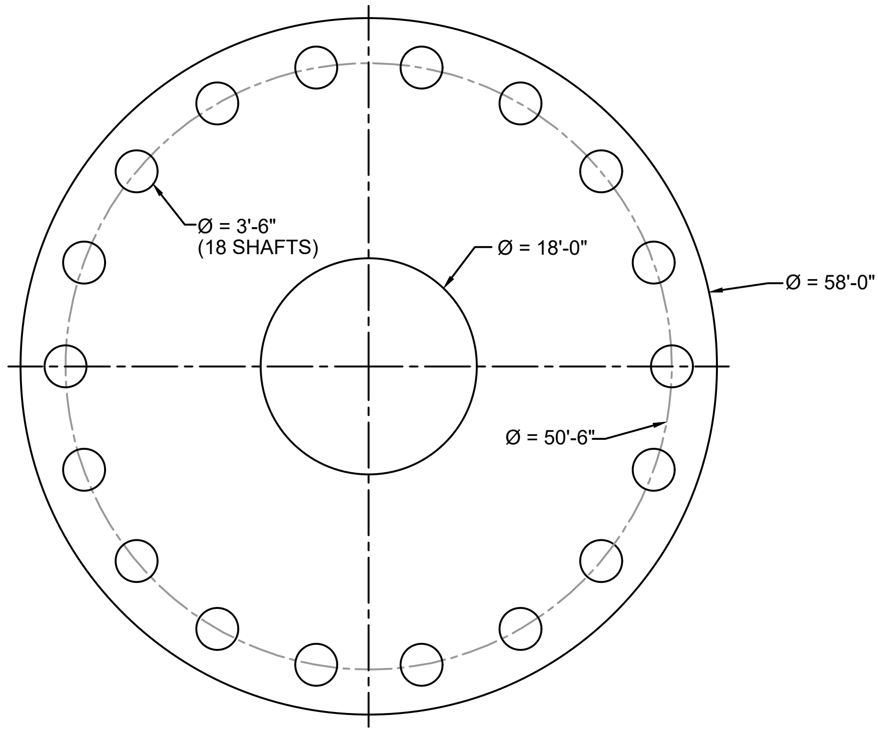

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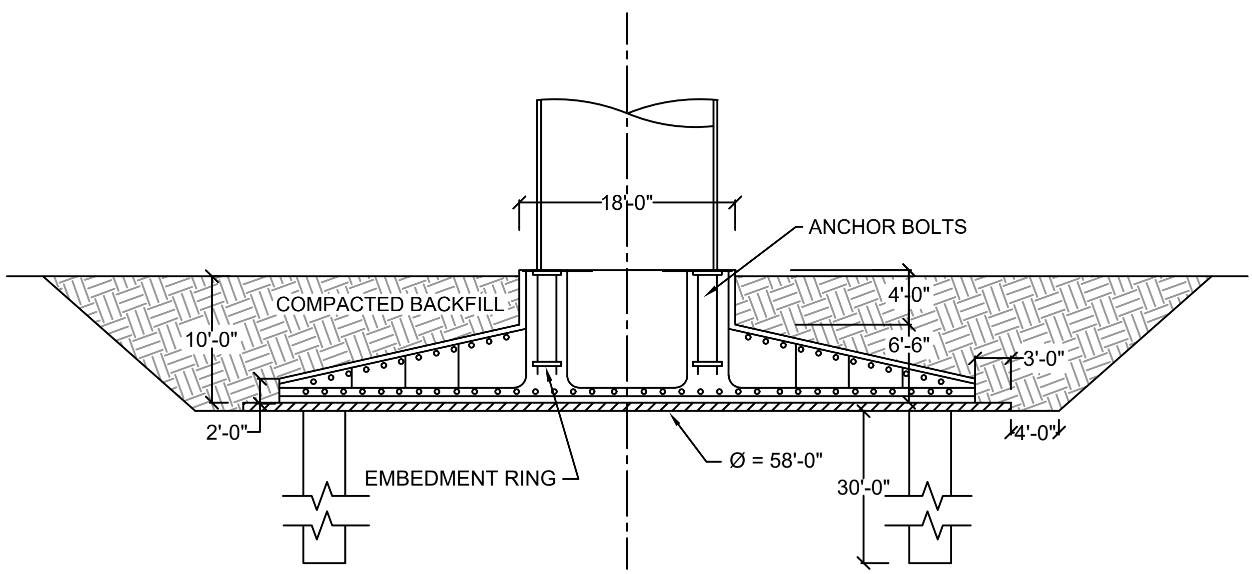

SECTION VIEW

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SCALE: 1 inch = 16 feet

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FOUNDATION DRAWING
PILED RAFT OPTION

SMUD-Solano Wind Farm Phase 4
Solano County, California

FIGURE

5

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APPENDIX A

Preliminary Foundation Quantities

Preliminary Foundation Quantities

Solano Phase 4 Wind Project, California, USA – 19 Vestas V150-4.0/4.2MW on 105m HH IEC 3B

	Parameter		Option 1 – Gravity Base	Option 2 – Piled Raft
Geotechnical Assumptions	Net ultimate bearing (psf)		15000.0	10000.0
	Net allowable bearing (psf)		6667.7	4444.4
	Groundwater depth below FGS (ft)	D_w	Non-buoyant	Non-buoyant
	Shear wave velocity (ft/s)	V_s	>700.0	>700.0
	Backfill moist unit weight (pcf)		110.0	110.0
WTG Loads & Foundation Stiffness	Abnormal Extreme (excl. PLF=1.1) M = Moment (kNm) P = Axial (kN) H = Horizontal (kN) T = Torque (kNm)	M	120800.0	120800.0
		P	5233.0	5233.0
		H	1176.0	1176.0
		T	970.6	970.6
	Normal Extreme (excl. PLF=1.35) M = Moment (kNm) P = Axial (kN) H = Horizontal (kN) T = Torque (kNm)	M	101300.0	101300.0
		P	5246.0	5246.0
		H	963.6	963.6
		T	799.7	799.7
	Stiffness (GNm/rad or MN/m)		Dynamic: 100.0, 9.0	Dynamic: 100.0, 9.0
Foundation Geometry	Shape		Circular	Circular
	Base diameter		72’-0”	58’-0”
	Bearing depth		10’-0” below finished grade	10’-0” below finished grade
	Base middle thickness		6’-6”	6’-6”
	Pedestal height		4’-0”	4’-0”
	Edge thickness		1’-4”	2’-0”
	Pedestal diameter		18’-0”	18’-0”
	Drilled shafts		NA	18 x 3’-6” x 30’-0”
Estimated Quantities	Concrete volume (yd^3)		578.0	630.0
	Cement type		Type II w/ Fly Ash	Type II w/ Fly Ash
	Mud mat concrete (yd^3)		68.0	47.0
	Weight of steel (tons, 75% Grade 75)		49.0	44.0
	No. x Length of anchor rods		176 x 9’-7”	176 x 9’-7”
	Embedment ring: width x thickness		22” x 2.5” Gr. 36	22” x 2.5” Gr. 36
	Anchor rod size x grade		M42 Gr. 10.9	M42 Gr. 10.9
	Tower base grout		15000	15000
	Pedestal concrete strength (psi)		6000	6000
	Base concrete strength (psi)		5500	5500
	Estimated number of foundations		19 out of 19	19 out of 19
Notes	This estimate is based on geotechnical exploration reports from neighboring earlier phases of the project and non-project specific, preliminary, load document by Vestas. The gravity base option must include provision for potential ground modification at a substantial number of WTG locations consisting of rammed aggregate piers (Geopiers) or over-excavation and re-compaction or replacement with structural fill to reach specified bearing capacity.			