APPENDIX 10G

Geologic and Foundation Design Criteria

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10G1 Introduction

This appendix contains a description of the site conditions, and preliminary foundation-related subsurface conditions for the Humboldt Bay Repowering Project (HBRP) to support the Application for Certification (AFC). Geologic hazards discussed are surface faulting potential, seismic ground motions, and tsunami potential. Soil-related hazards addressed include soil liquefaction, hydrocompaction (or collapsible soils), and expansive soils. Preliminary foundation and earthwork considerations are based on general published information available for the project area including past and recent geotechnical investigations at and adjacent to the site on the PG&E Humboldt Bay Power Plant property, established geotechnical engineering standard of practices, and pertinent codes and regulatory requirements. A detailed geotechnical investigation is being conducted to address the subsurface soil conditions in order to develop site-specific and detailed design conditions.

Information contained in this appendix reflects the codes, standards, criteria and practices generally used in the design and construction of site and foundation engineering systems for the facility. More specific project information will be developed during execution of the project to support detailed design, engineering, material procurement, and construction specifications.

10G2 Site Conditions

The HBRP project site is within the existing approximately 143-acre PG&E parcel that is used for the Humboldt Bay Power Plant. The site is at an elevation of 12 feet above mean lower low water (MLLW) on the margin of Humboldt Bay adjacent to the town of King Salmon and some 3 miles south of Eureka in unincorporated Humboldt County. The proposed power plant is sited on a flat surface underlain by fill that was placed on the tidal flat at the foot of the northeast side of a small hill known as Buhne Hill. Buhne Hill protects the site from storm waves that enter Humboldt Bay through the channel and the tidal marsh to the north separates it from the tides of Humboldt Bay. Elk Creek and its marshes lie to the east and northeast of the site.

A site-specific preliminary geotechnical investigation was performed in July and August 2006 at the project site. A copy of the preliminary geotechnical report is included as an attachment to this appendix.

10G3 Site Subsurface Conditions

10G3.1 Stratigraphy

The site is underlain by Holocene estuarine deposits consisting of layers and lenses of silt, organic silt, and fine sand. Beneath the estuarine sediments lie deposits of the late

Quaternary Hookton Formation. These consist of alluvial, estuarine, and shallow marine beds of sand, silt and clay with local gravel lenses and beds. A more detailed discussion of the stratigraphy is presented in Section 8.4, Geologic Hazards and Resources.

10G3.2 Seismicity/Ground Shaking

The project region is a zone of high seismic activity and strong earthquakes causing seismic significant shaking are expected to occur in this area in the future. The site is located in Seismic Zone 4, according to the California Building Code. Hence, the project has elected to use the California Building Code (CBC) for seismic design loads. The CBC refers to the Structural Engineering Design Criteria (Appendix 10B, paragraph 3.1.3). Assuming a type D soil, this results in a 10 percent probability of exceedance in 50 years (10/50). A description of the local geology and the relative location of major geologic faults in the area is presented in Section 8.4, Geologic Hazards and Resources.

The site lies within the flood zone from a tsunami. The level of probable maximum tsunami flooding is estimated to be 21 to 36 feet above MLLW (or 9 to 24 feet at the Humboldt Bay Power Plant site). Currently, there are no officially recognized probabilistic flood hazard maps for the region; however, previous studies estimate a 10 percent probability of exceedance in 50 year (10/50) of elevations at the site of between 5 and 9 feet. Tsunami flow velocities are estimated to be between 6.5 and 16.5 feet /second.

10G3.3 Ground Rupture

The Little Salmon fault, a reverse fault, lies beneath PG&E's Humboldt Bay Power Plant property and is a Holocene fault that is a landward expression of the Cascadia subduction zone. Two strands of this fault bound Buhne Hill. The Buhne Point fault projects along the south side of Buhne Hill about 800 feet from the HBRP. The Discharge Canal fault projects 300 feet to the north of the HBRP. Extensive trenching for the ISFSI has shown that no faults occur between these faults. Hence there is no potential for surface fault rupture from these faults at the HBRP. A more detailed discussion of the faults is presented in Section 8.4, Geologic Hazards and Resources.

The site has the potential for surface cracking and horizontal displacement from lateral spreading from liquefaction during and earthquake. Liquefaction potential at the site is discussed in section 10G4.1

10G3.4 Groundwater

The groundwater at the HPRP site is controlled by the sea level and the tides. Depending on the permeability of the Holocene estuarine strata beneath the site the elevation of groundwater lies between mean lower low high water and mean higher high water (MHHW). Analysis for the ISFSI indicates that perched water in the Holocene estuarine deposits lies at elevations 8 to 10 feet above MLLW and is brackish. The piezometric surface of the confined Hookton aquifer is at elevation 5 feet above MLLW.

10G4 Assessment of Soil-Related Hazards

10G4.1 Liquefaction

During strong earthquakes, loose, saturated, cohesionless soils can experience a temporary liquefaction where the soils have lost their shear strength and act as a fluid. Liquefaction is dependent on depth to water, grain size distribution, relative density of the soils, degree of saturation, and intensity and duration of the earthquake. The potential hazard associated with liquefaction is seismically induced settlement and possible lateral spreading. Soil liquefaction and lateral spreading can lead to foundation bearing failures with excessive settlements when:

- Potential ground accelerations from earthquakes are high
- The depth to saturated strata is shallow
- The strata have significant loose sandy layers that have low standard penetration test (SPT blow counts) that show low soil density.

The depth to groundwater at the HBRP in the Holocene estuarine sediments is shallow, 2 to 4 feet. The soil types in the Holocene estuarine deposits generally consist of loose, poorly consolidated sands, silts and organic silts. These deposits have a high potential for liquefaction but are limited to less than about 12 feet below the site (the depth will be proven during detailed drilling planned for the geotechnical investigations at the site). The underlying Hookton Formation deposits consist of generally dense sands and stiff clays that are not considered to be susceptible to liquefaction.

10G4.2 Expansive Soils

Expansive soils are clayey soils that shrink and swell with wetting and drying. The shrink-swell capacity of expansive soils can result in differential movement beneath foundations. Expansive soils shrink and swell with wetting and drying. Expansive soils are not expected in the Holocene estuarine deposits nor in the Hookton formation because these predominately consist of low plasticity stiff clays. Expansion potential index tests will be conducted on suspect clay deposits in the foundation area as part of the geotechnical studies that will be done for the HBRP. The potential for expansive soils at the site is low. However, if such soils are found, they will be mitigated in the foundation design. Expansive soils are further discussed in Section 8.11, Soils and Agriculture.

10G4.3 Collapsible Soils

Soil collapse (hydrocompaction) is a phenomenon that results in relatively rapid settlement of soil deposits due to addition of water in arid to semiarid climates. This generally occurs in soils having a loose particle structure cemented together with soluble minerals or with small quantities of clay. Water infiltration into such soils can break down the interparticle cementation, resulting in collapse of the soil structure. Based on the fact that the site is in a wet climate and is only 2 to 4 feet above the saturated sediments, the potential for significant soil collapse is virtually nil.

10G5 Preliminary Foundation Considerations

10G5.1 General Foundation Design Criteria

For satisfactory performance, the foundation of any structure must satisfy two independent design criteria. First, it must have an acceptable factor of safety against bearing failure in the foundation soils under maximum design load. Second, settlements during the life of the structure must not be of a magnitude that will cause structural damage, endanger piping connections or impair the operational efficiency of the facility. Selection of the foundation type to satisfy these criteria depends on the nature and magnitude of dead and live loads, the base area of the structure, and the settlement tolerances. Where more than one foundation type satisfies these criteria, then cost, scheduling, material availability and local practice will probably influence or determine the final selection of the type of foundation.

An evaluation of the information collected for the AFC indicates that no adverse foundation-related subsurface and groundwater conditions would be encountered that would preclude the construction and operation of the proposed structures. The site can be considered suitable for development of the proposed structures in consideration of the geotechnical investigation to support of the engineering design, and using the information to address the preliminary foundation and earthwork considerations discussed in this appendix.

10G5.2 Foundation Types

The various components of the proposed power plant facility shall be supported on the types of foundations recommended in the detailed geotechnical investigations that will be performed for this project. The foundation types may include conventional spread foundations, either individual spread footings, or continuous wall footings, and pile foundations. The site preparation work and foundation selection shall be engineered to mitigate any effects of soil shrinkage, expansion, and settlement and lateral spreading from liquefaction. Site preparation work shall include, but not be limited to, the removal or mixing of expansive soils and liquefiable layers.

10G5.3 Corrosion Potential and Ground Aggressiveness

The ground water in the Holocene estuarine strata is brackish. The ground water in the Hookton Formation is fresh. Corrosivity tests will be conducted to determine whether the site soils to be non-corrosive or corrosive for buried steel based on the chloride content and pH values.

10G6 Preliminary Earthwork Considerations

10G6.1 Site Preparation and Grading

Site grading may include (1) removal of existing deleterious materials and (2) fill to bring the site to a final grade. The site fill work should be performed as detailed in Section 10G6.3 below. All soil surfaces to receive fill should be proof rolled with a heavy vibratory roller or a fully loaded dump truck to detect soft areas.

10G6.2 Temporary Excavations

It is anticipated that confined temporary excavations at the site will be required during construction to remove undocumented fill or loose disturbed soils encountered during construction. All excavations should be sloped in accordance with OSHA requirements. All areas of the site should be subexcavated to a minimum depth of 36 inches below the existing surface to identify any undocumented fill or loose disturbed soils.

10G6.3 Backfill Requirements

All fill material must be free of organic matter, debris or clay balls, with a maximum size not exceeding 6 inches. Structural fill must also be well graded and granular. Granular material with similar specifications can be used for pipe bedding, except that the maximum size should not exceed 0.5 inch.

Structural fill should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557 when used for raising the grade throughout the site, below footings or mats, or for rough grading. Fill placed behind retaining structures may be compacted to 90 percent of the maximum dry density as determined by ASTM D 1557. Initially, structural fill should be placed in lifts not exceeding 8 inches loose thickness. Thicker lifts may be used pursuant to approval based on results of field compaction performance. The moisture content of all compacted fill should fall within 3 percentage points of the optimum moisture content measured by ASTM D 1557, except compact the top 12 inches of subgrade to 95 percent of ASTM D 1557 maximum density.

Pipe bedding can be compacted in 12-inch lifts to 90 percent of the maximum dry density as determined by ASTM D 1557. Common fill to be placed in remote and/or unsurfaced areas may be compacted in 12-inch lifts to 85 percent of the maximum dry density as determined by ASTM D 1557.

10G7 Inspection and Monitoring

A California-registered Geotechnical Engineer or Engineering Geologist will monitor geotechnical aspects of foundation construction and/or installation, and fill placement. At a minimum the Geotechnical Engineer/Engineering Geologist will monitor the following activities:

- All surfaces to receive fill should be inspected prior to fill placement to verify that no pockets of loose/soft or otherwise unsuitable material were left in place and that the subgrade is suitable for structural fill placement.
- All fill placement operations should be monitored by an independent testing agency. Field compaction control testing should be performed regularly and in accordance with the applicable specification to be issued by the Geotechnical Engineer.
- All sources of imported fill must be approved by the Geotechnical Engineer.
- The Geotechnical Engineer must approve the foundation design.

- Settlement monitoring of significant foundations and equipment is recommended on at least a quarterly basis during construction and the first year of operation, and then semi-annually for the next 2 years.

10G8 Site Design Criteria

10G8.1 General

The project will be located near Eureka in unincorporated Humboldt County on an approximately 143-acre parcel north of King Salmon Way. The site would be accessible from King Salmon Way.

10G8.2 Datum

The current site grade is at an elevation of approximately 12 feet above MLLW. Final site grade elevation will be determined after the completion of the geotechnical investigations.

10G9 Foundation Design Criteria

10G9.1 General

Foundations (reinforced concrete spread footings, mats, continuous wall foundations, or piles) will be designed consistent with Appendices 10A and 10B.

Geotechnical design parameters, e.g. allowable soil bearing pressures, lateral bearing, and lateral sliding, for foundation design will be in accordance with the recommendations from the site geotechnical investigation report (see Attachment 1 for the preliminary report).

10G9.2 Groundwater Pressures

Hydrostatic pressures due to groundwater or temporary water loads will be considered.

10G9.3 Factors of Safety

The factor of safety for structures, tanks and equipment supports with respect to overturning, sliding, and uplift due to wind and buoyancy will be as defined in Appendix 10B, Structural Engineering Design Criteria.

10G9.4 Load Factors and Load Combinations

For reinforced concrete structures and equipment supports, using the strength method, the load factors and load combinations will be in accordance with Appendix 10B, Structural Engineering Design Criteria.

10G9.5 Attachment 1, Kleinfelder, Geotechnical Investigation

10G10 References

California Building Code, 2001.

APPENDIX 10G

Attachment 1: Preliminary Geotechnical Report

KLEINFELDER

FIELD MEMORANDUM

Geotechnical Engineering
Materials Testing & Inspection
Environmental Science & Engineering
Water Resources
Earthquake Engineering
Air Quality

Date: September 7, 2006

To: Wartsila North America, Inc.
16330 Air Center Blvd.
Houston TX 77032

Attn: Dale Love
Jukka Lehtonen

From: Chris Spandau, PE
Terry Craven, PE

Project No.: **68522**

Subject: **Preliminary Report**
Geotechnical Exploration
Humboldt Bay Power Plant
Eureka, California

During the period from July 25, 2006 to August 3, 2006, Kleinfelder explored the subject site by means of three drilled borings and nine cone penetrometer probes. Exploration logs are attached. A site plan showing exploration locations is being prepared by Warsila. Subsequent to the field explorations Kleinfelder performed a series of laboratory tests to develop soil properties for design. Most testing is complete, although a few additional tests are still being performed.

The purpose of the memorandum is to provide the results of our exploration and laboratory testing program for the subject project. We have also included a brief discussion of potential foundation types. Selection of a final foundation design is normally a collaborative process involving the owner, the structural engineer and the geotechnical engineer. The more information that we are provided regarding building loads and tolerable settlement, the better we can assist in this process.

SUBSURFACE CONDITIONS

The following paragraphs summarize the major soil units identified in our field explorations. The attached exploration logs boring logs should be reviewed for a more detailed description of the subsurface conditions at the locations explored.

Fill

Most of the site was covered with asphalt pavement. The existing pavement section consisted of approximately 2 to 6 inches of asphalt concrete over aggregate baserock and/or blended artificial fill material. Below this pavement as well as beneath landscaped areas we encountered fills consisting of silty clay and sandy clayey gravel varying in depth from about 2 to 6 feet thick. The fill soils were

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observed to be soft to very stiff and generally free of organics and oversize rock fragments. The near surface soils are primarily clay-rich, with moderate to high plasticity.

Holocene Bay Deposits (Young Bay Deposits)

Below the fill and extending to depths ranging from 2 to 25 feet, highly variable bay and marsh deposits were encountered consisting of clays, silty clays, clayey silts, and rare clayey sands. Marsh deposits (peat) were observed in numerous boring locations at depths ranging from 2 to 12 feet, and organics were observed in nearly all exploration locations at varying depths up to 25 feet. Soils generally varied from soft/loose near the ground surface to medium dense/medium stiff at depth, with transitions being highly variable.

Pleistocene Hookton Formation (Upper)

Below the Holocene Bay Deposits (and/or beneath onsite artificial fill) we encountered laterally discontinuous beds of clay and silt, and sand and gravel that change laterally with inter-fingering and gradational facies changes. Clay beds have more lateral persistence than interbedded sand and gravel layers. In our exploration this formation was encountered to depths ranging from 80 to 95 feet. The alternating layers of clay, silt, sand, and gravel in the Upper Hookton Formation can be sub-divided according to lithologic characteristics as follows:

1st Bay Clay

Below the Holocene Bay Deposits and/or artificial fill, deposits were encountered consisting of clays, silty clays, clayey silts, and rare lenses of clayey and silty sands. In our explorations this unit was encountered to depths ranging from 35 to 45 feet. Fine-grained soils in this interval generally varied from stiff to hard with depth, although some softer soils were noted at near the top of this unit.

Upper Sand Beds

Below the 1st Bay Clay, we encountered discontinuous beds of clayey and silty sand inter-fingered with rare sandy gravel lenses. During our exploration this unit was encountered to depths ranging from 55 to 75 feet. Coarse-grained soils in this interval generally varied from dense to very dense with depth.

2nd Bay Clay

Below the Upper Sand Beds, deposits were encountered consisting of clays, silty clays, and clayey silts. In our exploration this unit was encountered to depths ranging from 80 to 95 feet. Fine-grained soils in this interval were typically hard.

Pleistocene Hookton Formation (Lower)

Below the Upper Hookton Formation and extending beyond the depths sampled during this study, we encountered laterally discontinuous beds of silty, clayey, and gravelly sand. Coarse-grained soils in this interval were typically dense to very dense with depth.

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BUILDING FOUNDATIONS

General

As discussed previously, subsurface conditions can generally be divided into the following principal units.

Unit Name	Approximate Depth to Top of Unit (feet)	Approximate Depth to Bottom of Unit (feet)
Fill	0	2 to 6
Young Bay Deposits	2 to 6	2 to 25
1st Bay Clay	2 to 25	35 to 45
Upper Sand	35 to 45	55 to 75
2nd Bay Clay	55 to 75	80 to 95
Lower Hookton	80 to 95	Not encountered

We emphasize that native conditions consist of layered, alluvial soil. As such, transitions between layers are variable and may be very gradual. In some areas the clay and sand layers are "inter-fingered" and the transition between layers could have been delineated at more than one location.

The Fill and Young Bay Deposits are not suitable for the support of foundations in their present condition. The 1sr Bay Clay generally has moderate strength and compressibility and could be suitable for the support of lightly loaded foundations. However this layer contains zones of softer material and also has pockets or thin layers of potentially liquefiable sand. Because of its variability and the potential for differential settlement we do not recommend supporting major structures on this layer.

We recommend that major building and floor slabs loads be supported on a deep foundation system bearing on the Upper Sand unit or deeper soils. It is our opinion that driven piling will probably offer the best type of foundation support for the proposed facility, although augercast piling or drilled piers could also be used. Ground improvement techniques, such as deep soil mixing may also be considered. In this memorandum we have discussed the foundation types that we consider most suitable for the proposed facility.

Driven Piling

Pile types are usually selected based on a combination of considerations including desired capacity, driving conditions and cost of materials. For pile capacities in the range of 50 tons, pre-cast concrete piling would probably be the most economic solution for this site. We expect that 12- or 14-inch square pre-cast piling could develop capacities of at least 50 tons from the upper sand layer, with tip penetrations of about 60 feet below existing grade. However, because of variable conditions piling should be driven to a minimum blowcount in addition to a specified depth, and some variation in pile lengths should be anticipated.

If higher pile capacities are desired, greater variations in pile lengths are probable and some piles may penetrate through the upper sand layer, potentially terminating in the Lower Hookton with tip penetrations of 100 feet or more. For heavily loaded piling we would typically recommend steel pipe piles because of the relatively lower cost to add length (splice), when necessary.

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Because of the high variability of soils across this site, for either pile type we recommend that pile order lengths be based on the results of an indicator pile-driving program. For this program we would typically recommend installing approximately 20 piling distributed across the site. Indicator piling should be of the same type and should be driven with the same equipment as production piling.

The potential for damage to existing facilities from pile driving induced vibrations depends upon a combination of displacement and frequency of the vibration, as well as the sensitivity of the existing facilities. It is generally recommended that vibrations be monitored on all structures that are located within approximately 50 feet of driven piling. Additional monitoring may be warranted for unusually sensitive structures.

Augured Piles or Piers

For moderately loaded piles (less than about 50 tons) drilled piles or piers could also be a suitable foundation type, deriving the majority of their support from the Upper Sand Unit, with tip penetrations of about 25 feet into the Upper Sand unit. These piling have the advantage that lengths can be easily varied without the need to cut off or splice on. However because of the shallow groundwater table, and soft near surface soils, either deep casing or augercast methods would be necessary. Placement of reinforcing steel can be difficult under these conditions, particularly if large uplift or lateral loads need to be resisted (which is probable due to the seismic setting of the site). Also, because blowcount information is not available as an indicator of pile capacity, we would recommend a minimum of three pile load tests to verify adequate capacity.

Deep Soil Mixing or Jet Grouting

These methods create in-situ columns of cement-modified soil to provide suitable foundation support. Typically the columns would be drilled to a minimum embedment of 10 to 20 feet into the Upper Sand unit. They would be specified as requiring 150 to 200 psi soil:cement and would typically be designed for pressures in the range of 50 psi. Thus a 36-inch column could be designed for a structural load of approximately 50 kips, a five-foot diameter column could be designed for a load of approximately 140 kips. Normally this foundation system is best suited for a project where loads are fairly evenly distributed and structures can be supported on mat type foundations that bear directly of the soil/cement columns.

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