Geotechnical Investigation and Seismic Hazard Study

Turner Energy Center

Turner, Oregon

Prepared for:

Calpine Corporation
Portland, Oregon

February 21, 2002

Professional
Geotechnical
Services

Foundation Engineering, Inc.

Foundation Engineering, Inc.
Professional Geotechnical Services

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February 21, 2002

Project 2011116

Turner Energy Center
Geotechnical Investigation and
Seismic Hazard Study
Turner, Oregon

Dear Mr. Tetzloff:

We have completed the requested geotechnical investigation and seismic hazard study for the above-referenced project. The findings of the seismic study were summarized in a report dated February 8, 2002. For completeness, portions of the seismic study are included in the body of this report and the complete seismic report is appended. Our scope of work is in general accordance with Calpine's Technical Specification 02100 for subsurface soils investigations.

This report includes a description of our work, a discussion of site conditions, a summary of field and laboratory testing, and a discussion of engineering analyses and relative seismic hazards. Recommendations for site preparation, foundation and pavement design, and construction are enclosed.

It has been a pleasure assisting you with this phase of your project. Please do not hesitate to contact us if you have any questions or if you require further assistance.

Sincerely,

FOUNDATION ENGINEERING, INC.

William L. Nickels Jr., P.E.
Project Engineer

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WILLIAM L. NICKELS JR.
EXPIRES: 12/31/02

James K. Maitland, P.E.
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enclosures

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EXECUTIVE SUMMARY

A geotechnical investigation and seismic hazard study have been completed for the proposed Turner Energy Center in Turner, Oregon. Findings from the seismic site hazard investigation are summarized in a report dated February 8, 2002. The seismic study fulfills requirements presented in the guidelines for site-specific seismic hazard reports for essential and hazardous facilities, and major and special-occupancy structures in Oregon. Our study concluded there are no relative seismic hazards that would preclude development of the proposed facility or that would require site mitigation or special foundation considerations.

The proposed facility will be located on a ±44-acre site, east of Wipper Road. The borings and field resistivity locations were sited to evaluate the subsurface conditions beneath the major energy plant components and support facilities. Shallow test pits were also dug to complement information obtained in the borings. Coordinates for the boring and resistivity locations were provided by Burns & McDonnell and the project surveyor staked the locations in the field. One downhole seismic velocity survey was completed in the deepest boring to obtain a shear wave velocity profile.

The explorations show the site is typically underlain by gravelly silt (topsoil) to a depth of ±1 to ±3 feet, followed by very dense alluvial deposits of sandy gravel and cobbles to a depth of at least ±75 feet, the limits of our explorations. Occasional lenses or interbeds of medium grain sand were observed in several of the borings and test pits. Some very weak cementation was also observed in some of the gravel strata in the test pits.

We evaluated spread footings and mat foundations for bearing capacity and settlement assuming the base of all footings and mats would be supported on very dense alluvium. An allowable bearing capacity of 6,000 psf and 3,000 psf is recommended for the mat foundations and spread footings, respectively, assuming the footings are designed and constructed as recommended herein. Immediate settlement for the mats and individual footings bearing in the dense alluvium is estimated to be less than 1 inch.

Piezometers were installed in all borings to monitor ground water levels across the site. Even though Oregon experienced a record dry year, October ground water levels were as shallow as ±2½ to 3 feet below the ground surface. Water levels rose closer to the ground surface during subsequent winter readings. Consequently, foundation and trench excavations extending below ±2 to 3 feet will likely encounter moderate ground water infiltration and require dewatering.

Approximately 2 to ±5 feet of fill (imported and generated from the site) will be required to raise the power block area to ±El. 290, and ±2 to ±3 feet of fill will be required to raise the switchyard area to ±El. 286. Raising the site will reduce the dewatering effort in the deeper excavations for the mat foundations and will also help protect the area from seasonal flooding. The earthwork recommendations presented herein assume site grading and filling will be completed during dry weather.

GEOTECHNICAL INVESTIGATION AND SEISMIC HAZARD STUDY
TURNER ENERGY CENTER
TURNER, OREGON

1.0. INTRODUCTION

1.1. *Project Description*

Calpine Corporation plans to construct a gas-fired energy plant on a ±43-acre site located east of Wipper Road in Turner, Oregon. The site is made up of several smaller parcels that are generally located within T8S, R2W, NE quarter of Section 32 – Willamette Meridian, Marion County. The project location is shown on the Vicinity Map, Figure 1A, Appendix A.

an air cooled condenser

Preliminary plans provided by Calpine show the major components of the plant will include: two combustion turbines (CT's) and associated auxiliary equipment, two heat recovery steam generators (HRSG's) including stacks, one steam turbine generator (STG) and condenser, and a mechanical draft cooling tower. Above grade storage vessels will include three water storage tanks, and an ammonia storage and unloading facility. Buildings will include a large enclosure for the STG and CT's, and several smaller, independent structures for administration personnel and auxiliary equipment.

A 230 kV and 115 kV switchyard will be located on the west side of the facility adjacent to Wipper Road. Foundation requirements for the switchyard components include small mat foundations and drilled shafts. However, foundation loads were not available at the time this report was prepared. Therefore, analysis, design and construction recommendations for the drilled shafts will be provided in a report addendum.

1.2. *Purpose and Scope of Work*

The purpose of the investigation was to evaluate the soil and ground water conditions across the proposed site. Once the borings and test pits were completed, the suitability of the materials for foundation support and potential construction challenges were evaluated. In addition, a site-specific seismic hazard study was completed to evaluate the relative seismic hazards at the site.

Our proposed scope of work for a two-phase investigation was outlined in a proposal dated June 28, 2001. Subsequent to submitting our original work scope, additional work was added to meet developing project requirements. The work included additional drilling (completed simultaneously with the original drilling scope), laboratory testing and field monitoring as outlined in a memorandum dated October 16, 2001. Additional tasks related to the seismic hazard investigation were also added following a review of Exhibit H by the Oregon Department of Geology and Mineral Industries (DOGAMI). Among other items, DOGAMI suggested a dynamic (SHAKE) analysis be performed. The additional, seismic tasks were outlined in a memorandum dated January 18, 2002.

Our completed scope of work included the following tasks and subtasks:

Task 1. Field Exploration, Sampling and Testing:

- Drilling and sampling of ten borings to a maximum depth of ±70 feet. Standpipe piezometers were installed in all borings (except BH-2) to monitor ground water levels. A slope indicator casing was installed in BH-2 for the downhole seismic velocity survey.
- Digging and sampling of 24 test pits to a maximum depth of ±13 feet.
- Completion of eight field resistivity surveys using the four-pin method.
- One downhole seismic velocity survey to obtain shear wave velocity data for the site.

Task 2. Laboratory Testing:

- Testing required to classify the foundation soils and estimate their overall engineering properties.
- Testing required to evaluate the corrosion potential and chemical attack to concrete.

Task 3. Evaluation of the sites geologic setting, relative geologic and seismic hazards, geotechnical information, engineering analyses and report preparation which contains the following:

- A discussion of the findings from Task 1 and Task 2.
- Regional and local geology, subsurface conditions and ground water.
- Local faulting, seismicity, evaluation of relative seismic hazards and preparation of site response spectra for three earthquake scenarios.
- Recommended modulus of subgrade reaction for the design of mats and slabs-on-grade.
- Allowable bearing capacity, estimated settlements and sliding coefficients.
- Lateral earth pressures for retaining walls.
- Flexible and rigid pavement design.
- Drilled shaft design and construction recommendations for switchyard transmission towers (to be addressed in a report addendum).
- Discussion of site-specific conditions that may impact the proposed construction.
- Recommendations for site preparation and grading.

Calpine Corporation (Calpine) is the project owner. Foundation Engineering, Inc. (FEI) was retained by Calpine to complete a geotechnical investigation and seismic hazard study for the site.

2.0. GEOLOGY

Local and regional geology and an assessment of geologic hazards are presented in this section. A discussion of tectonic setting, local faulting and seismicity is presented in the Seismic Site Hazard Investigation report, Appendix D.

2.1. *Literature Review*

Geologic and seismic publications and maps were reviewed to evaluate relative seismic hazards at the site. Water well logs, available from the Oregon Department of Water Resources website, were also reviewed to establish an estimate of the subsurface conditions prior to our site investigation.

2.2. *Regional Geology*

On a regional scale, the project site lies near the middle of the Willamette Valley, a broad, gently deformed, north-south-trending basin separating the Coast Range to the west from the Cascade Range to the east. In the early Eocene (approximately 55 million years ago), the Willamette Valley province was part of a broad continental shelf extending from the Cascades westward beyond the present coastline. Basement rock underlying the site area includes the Siletz River Volcanics, which erupted as part of a submarine oceanic island archipelago. The archipelago collided with the converging North American plate and was accreted to the western margin of North America near the end of the early Eocene. The volcanics subsequently subsided and the basin that formed over them became a repository for marine sediments throughout the late Eocene and Oligocene. After emerging from a gradually shallowing ocean, the marine sediments were covered during the middle Miocene by the Columbia River basalt, which poured through the Columbia Gorge from eastern Oregon, spreading as far south as the Salem area. Uplift and tilting of the Coast Range block and the Western Cascades during the late Miocene formed the trough-like configuration of the Willamette Valley. Thick layers of Late Pleistocene and Holocene Alluvium and glacial outwash deposits blanket the Columbia River Basalt and older Tertiary units in the site area (Orr and Orr, 1999).

2.3. *Local Geology and Geologic Hazards*

The project area is underlain by Pleistocene-age, glacial outwash gravel deposited in a large alluvial fan by the North Santiam River (Orr and Orr, 1999). The alluvial deposits extend along Mill Creek through Turner Gap (believed to be an ancestral channel of the Willamette River) to the Salem vicinity (Schlicker, 1977). Local geologic cross-sections indicate ±300 feet of gravel overlying Columbia River Basalt (Hampton, 1972). A water well located in T9S, R1W, Section 4 (near Stayton), penetrated basalt at ±335 feet.

Geologic hazards of the Turner area (excluding seismic) include a high potential for flooding and shallow ground water (Schlicker, 1977). The risk of landslides and surface erosion is low for this site due to the flat terrain and distance from the surrounding hills.

3.0. DISCUSSION OF SITE AND SUBSURFACE CONDITIONS

3.1. *Physiography, Site Topography and Vegetation*

The project area lies ±7 miles southeast of Salem and ±1 mile southwest of Turner in the Stayton Basin. The basin is bounded by the Waldo Hills to the north, the Salem Hills to the west and the Western Cascades to the east. The Stayton Basin slopes gently to the west and is drained by the North Santiam River and Mill Creek.

A topographic survey was completed by Northstar Surveying, Inc. The project site is relatively flat. The high point lies in the southeast corner of the site at ±El. 290 and slopes gently to the northwest to ±El. 282. Average slope inclination is less than 0.3 degrees (±0.5 percent).

Most of the vegetation consists of short grass. Scattered, small diameter trees extend along property lines of the smaller, interior parcels. The entire site appears to be fenced. At the time of our explorations, a portion of the property was used to graze sheep and horses. The Perrin Lateral canal is located immediately north and east of the project area and flows west and north into Mill Creek.

3.2. *Field Exploration*

The field exploration program consisted of ten borings and 24 test pits. The borings and Test Pit 1 (TP-1) through TP-8 were completed October 8 through October 15, 2001. Sixteen additional test pits were dug on November 27, 2001, to complement the first exploration phase. The objective of the exploration program was to observe material variability, density and/or stiffness of the strata beneath the proposed structures and pavements (i.e., site characterization). All explorations were logged by a staff geologist or project engineer. The boring and test pit locations, and the proposed layout of the facility are shown in Figure 2A, Appendix A. The boring and test pit locations (based on NAD 83 Northings and Eastings) and the completed depths are summarized in Table 1B and Table 2B, Appendix B.

3.2.1. *Borings*. The boreholes were drilled with a CME 75, truck-mounted drill rig and mud-rotary drilling techniques. The borings extended to depths of ±25 to ±75 feet. Samples were obtained at 2½-foot intervals to a depth of ±5 feet (upon encountering very dense gravel) and at 5-foot intervals thereafter. Disturbed samples were obtained with a split-spoon. The Standard Penetration Test (SPT), which is run when the split-spoon is driven, provides an indication of the relative stiffness or density of the foundation soils.

One-inch (I.D.) standpipe piezometers were installed in all borings, except for Borehole 2 (BH-2), to monitor ground water levels beneath the site. In BH-2, a 2.75-inch (O.D.) slotted, PVC inclinometer casing was installed and grouted in-place to a depth of ±75 feet. The slotted casing provides a track for the geophone used during the downhole survey. All installations were capped at the ground surface with a locking Morris monument set in concrete.

3.2.2. *Test Pits.* The test pits were dug with a rubber-tired, Case 580 extend-a-hoe. The deeper test pits extended to a maximum depth ranging from ±9 to ±13 feet. Shallower test pits (TP-5, TP-6, TP-9, TP-12, TP-15 and TP-18), dug to establish subgrade conditions within future roadways and parking areas, were generally ±4 feet deep or less. In some instances, the depth of the test pit was limited by caving sidewalls or ground water infiltration. No undisturbed soil samples were obtained due to the consistency and coarseness of the materials encountered. The soil profiles were logged and levels of ground water infiltration, where it occurred, were noted. All test pits were backfilled with the excavated material.

3.3. ***Subsurface Conditions***

Most of the borings and test pits encountered a relatively thin soil mantle (topsoil) followed by dense to very dense mixtures of sand, gravel and cobbles. The exception was in TP-4 and TP-17 where a thin zone of high plasticity clay was encountered below the topsoil. Detailed soil descriptions encountered in each exploration are provided in the boring and test pit logs (Appendix B). A general description of the three primary soil strata is provided in the following subsections.

3.3.1. *Layer 1: Topsoil.* The topsoil consists of dark brown, low to medium plasticity, loosely structured silt with variable amounts of gravel. The topsoil unit is referred to as gravelly silt in the appended logs. The topsoil thickness ranges from ±1 to 3 feet, but typically averages ±1½ feet. The mixing of the silt and gravel is likely the result of farming.

No SPT's were taken in the topsoil. However, based on observations made in the test pits, the coarse fraction of the material appears to be dense and supported the drill rig and backhoe at the time of our October explorations. It should also be noted that the topsoil was relatively dry in October (prior to the rainy season).

3.3.2. *Layer 1A: Clay.* The topsoil is underlain by grey, medium stiff to stiff, high plasticity clay in TP-4 and TP-17. The clay extends below the topsoil to a depth of ±2 to 3 feet and is underlain by dense to very dense alluvium (Layer 2). The clay will be removed where encountered beneath structures.

3.3.3. *Layer 2: Alluvium.* The topsoil and clay are underlain by dense to very dense mixtures of silt, sand, gravel, cobbles and scattered boulders (alluvium). Zones (or interbeds) of medium to fine-grain sand were also encountered in the explorations at various depths. The alluvium extended to the limits of our exploration (±75 feet), and may be as deep as ±300 feet according to geologic publications and local water well logs.

Uncorrected SPT values recorded in the coarse alluvium range from 26 blows per foot (bpf) (medium dense) to practical refusal (i.e., greater than 50 blows for any one of the 6-inch increments of drive). SPT values obtained in gravelly soils should be viewed with caution and require engineering judgment to characterize the density of the strata. However, we believe the SPT values obtained in the ten borings are representative of a dense to very dense profile. Our assessment is supported by the challenges encountered during drilling, the quantity of practical refusal SPT drives and the relatively high shear wave velocities obtained in BH-2.

SPT values suggesting medium dense soils (i.e., 11 to 30 bpf) were typically obtained within the upper ±5 to 10 feet. Some of the lower values are representative of scattered, medium-grained sand. Very weak cementation was also noted in some of the gravel in the test pits. The gravel and cobbles appear to have a silt and sand matrix between the voids of the larger particles. We do not consider the sand matrix as a type of cementing agent.

A ±1-foot to ±2-foot thick, medium to fine-grain sand interbed was observed in borings BH-2, BH-3, BH-5 and BH-6 beginning at a depth of ±10 feet. Uncorrected SPT values in the sand ranged from 26 to 33 bpf. Corrected $(N_1)_{60}$ values are 40 to 50 bpf, suggesting the sand is dense. The interbed was also observed from ±10 feet to ±11 feet in TP-7 and TP-8. Two shallower ±1-foot to ±1.5-foot thick interbeds were observed in BH-4 and BH-7.

Of the ±100 SPT drives attempted in the gravel and sand, one anomalous value of 10 was obtained in a sand layer encountered in BH-2 from ±39 to ±44 feet. The $(N_1)_{60}$ is 8 bpf, suggesting the material is loose. Sand was not observed at this depth in the surrounding borings that extended below 44 feet, suggesting the interbed is confined horizontally to the vicinity of BH-2.

An interpreted soil profile for the site extrapolated from subsurface conditions observed in BH-1, BH-2, BH-3 and BH-9 is shown in Figure 3A, Appendix A.

3.4. ***Ground Water***

Information obtained during our geologic literature review suggests relatively shallow ground water levels are present in the vicinity of the site throughout the entire year. Data collected from an off-site piezometer installed for an unrelated project also suggests the water level remains within several feet of the ground surface year round. The piezometer is adjacent to the site in the northbound lane of Wipper Road. Water levels measured from June 1999 through November 2001 range from ±2.4 (±El. 285.0) to ±4.5 feet (±El. 282.9) below the paved surface. The shallow and deep water levels were recorded in February 2000 and August 2001, respectively.

One-inch (I.D.) piezometers were installed in nine of the ten borings to observe the seasonal fluctuation in water levels. The depth of water observed in the piezometers following the mid-October installation ranged from 3.0 to 5.1 feet. These depths correspond to ±El. 282.1 (BH-10) to ±El. 283.8 (BH-6). Water depths observed on January 2, 2002, ranged from 0 (at the ground surface) to 2.1 feet. These depths correspond to ±El. 285.7 (BH-4) to ±El. 286.8 (BH-6).

Ground water infiltration was observed in all test pits greater than 4 feet deep. Infiltration generally occurred between ±3 and ±4 feet below the existing ground surface, but was as shallow as ±1½ feet in TP-18 (±El. 285.9) and as deep as ±6 feet in TP-4 (±El. 279.0). The rate of seepage varied between test pits, which is likely due to variations in gradation and interbedded zones of cementation in the alluvium.

During the October explorations, we observed water flowing periodically in the ditch along the south and west side of the site. We understand the water is used for irrigation and fluctuates considerably depending on local use. The water level in Mill Creek and in the Perrin Lateral canal can rise suddenly during periods of heavy rainfall. Because gravels underlie the entire site, we anticipate a relatively good hydraulic connection between the water level in the alluvial soils beneath the site and the water level in Mill Creek. During the November explorations, water perched at the ground surface was also flowing into several of the test pits.

All piezometric data collected to date is summarized in Table 1C (Appendix C).

4.0. FIELD AND LABORATORY TESTING

4.1. *Resistivity Testing*

Eight soil resistivity tests were completed using the four-pin method. The pins were aligned in either a north-south or east-west direction at locations shown on Figure 4A. The pin spacing ranged from 5 to 30 feet and readings were obtained at 5-foot intervals. The pin spacing corresponds to the soil resistivity at that depth. The resistivity tests are summarized in Table 2C, Appendix C.

4.2. *Downhole Seismic Velocity Survey*

Northwest Geophysical Associates, Inc. (NGA) completed a downhole seismic survey in BH-2 on October 24, 2001. The best fit shear wave (interval) velocities are summarized in Table 1. A minimum shear wave velocity of 920 ft/sec occurs in the soils from the ground surface to a depth of ±13 feet. A maximum shear wave velocity of 3,350 ft/sec was recorded for the alluvium between a depth of ±44 and ±54 feet. Velocities that exceed 2,000 ft/sec are high for near surface alluvial soils and suggest the deposit is very dense. The S-Wave Travel Time Plot, Shear Wave Velocities and the P-Wave Travel Time Plot from the study are attached to NGA's report (see Appendix C of the seismic report).

Table 1. Summary of Shear Wave Velocities (BH-2)

Interval (ft)	Velocity (ft/sec)
0 to ±13	920
±13 to ±22	1,710
±22 to ±37	2,330
±37 to ±44	1,710
±44 to ±54	3,350
±54 to ±70	2,180

4.3. *Laboratory Testing*

The subsurface soils consist predominately of dense to very dense, sandy gravel and cobbles with the exception of the plastic clay encountered in two of the test pits. Therefore, the laboratory testing was limited to one Atterberg limits test on a clay sample obtained from TP-17. Shear strength tests and consolidation tests were not required due to the relatively thin topsoil and clay layers. In addition, the large particle size precluded standard sieve analyses, evaluation of moisture-density relationships and CBR testing for pavement design. Additional laboratory tests were completed by others to evaluate the corrosion potential and chemical attack to concrete.

An Atterberg limits test was completed on Sample S-17-1 obtained at a depth of 1 to 2 feet. The tests suggest the sample has a liquid limit (LL) of 80, a plastic limit of 29 and a plasticity index of 51. These limits correspond to a high plasticity clay (CH) according to the Unified Soil Classification System (USCS), and are commonly associated with a moderate to high shrink/swell potential.

Two soil samples were obtained for evaluation of corrosion potential completed by an independent laboratory. Sample S-14-1 was obtained from TP-14 and Sample S-19-1 was obtained from TP-19. Both samples were taken at a depth of ±5 feet. The samples were evaluated for pH, minimum resistivity, redox potential, chloride ion, soluble sulfates and sulfides. A summary of the chemical testing is provided in Table 3C, Appendix C.

5.0. SEISMIC DESIGN

An overview of the seismic design is presented in this section. A detailed discussion of the analyses and development of site-specific response spectra, ground shaking and amplification, and liquefaction hazards are presented in the appended Seismic Site Hazard Investigation report (Appendix D). Our study concluded there are no relative seismic hazards that would preclude development of the proposed facility or that would require site mitigation or special foundation considerations.

Three postulated earthquake scenarios were evaluated using the computer program SHAKE. A site response spectrum was developed for the two possible subduction zone scenarios (interface and intraslab) and a randomly oriented crustal scenario. Our analyses concluded the ground response from the three earthquakes generally lie within the UBC (1997) response envelop for an S_c soil profile located in Zone 3. The spectra are located in Appendix D of the seismic report.

6.0. FOUNDATION ANALYSIS AND DESIGN RECOMMENDATIONS

The analyses and design of the various foundation elements for the main facility are discussed below. Specific recommendations for general site grading, subgrade preparation, foundation preparation and construction are provided in Section 8.0. Granular fill materials recommended for placement beneath mats, slabs and footings are defined in Section 8.1.

6.1. *Preliminary Design Loads*

Burns & McDonnell provided preliminary design loads for the new structures on November 20, 2001. The foundation types include mat foundations, shallow spread footing, and shallow ring foundations. Preliminary design loads and foundation sizes for the various structures are presented in Table 2.

Table 2. Estimated Structural Loads

Structure	Total Estimated Load (kips)	Foundation Size (ft)	Est. Static Contact Pressure(1) (kips/ft^2)
Combustion Turbine	4,200	111x30x6	2.2
HRSG	14,300	148x46x3	2.6
Steam Turbine/ Generator	9,000	102x43x7	3.1
Water Tank	13,194	Ring Foundation 90 ft diam 2.75 ft wide 3 ft deep	2.1(2)
Cooling Tower and Basin		330x56x1.33	0.6 to 1.3 ksf
Pump Pits			1.8 ksf

Notes: 1. Includes the weight of the mat where $\gamma_{concrete}$ is 0.15 kcf.
2. Assumes the total load is distributed over the entire tank diameter.

6.2. *Mat Foundations*

Geotechnical analyses related to mat foundations for the CT, HRSG and STG include evaluations of allowable bearing capacity, settlement, sliding coefficients, elastic subgrade moduli and passive earth pressure. A discussion of the analyses and recommendations for design are provided in the following subsections.

6.2.1. *Allowable Bearing Capacity*. Maximum contact pressures resulting from dead loads plus normal (live) operating loads, including the weight of the equipment and foundation could be up to 2.1 kips/ft^2 (ksf) (highest at the HRSG structure). The estimated allowable bearing pressure for the dense to very dense alluvium is much higher. Using the assumptions described in Section 6.3, we estimated an allowable bearing pressure of ±3.7 to 5.0 ksf for shallow spread footings. The allowable bearing capacity for larger mats is often taken as twice the value estimated for smaller spread footings. Therefore, for design, a maximum allowable bearing pressure of 6.0 ksf may be assumed, if required to resist transient (i.e., wind and seismic) loads. The allowable value is based on a typical factor of safety of 3.

The allowable bearing pressure value assumes a minimum embedment depth of 3 feet below the finish grade. We have also assumed the material underlying the CT, HRSG and STG mats will consist of a minimum of 12 inches of compacted, granular site fill underlain by dense to very dense native alluvium. The site fill will serve as a drainage blanket for excavation dewatering. The excavation for the mats should extend a minimum of 3 feet beyond the footprint of the foundation to allow for formwork and perimeter drain installation. Backfill around the mats should consist of imported, compacted crushed rock. Temporary dewatering and backfilling are discussed in Section 8.4.

6.2.2. *Settlement*. The native alluvium consists of dense deposits of sand, gravel and cobbles. Compression of the alluvium due to a net change in vertical stress will occur rather quickly and elastically, rather than time dependently. Therefore, compression of the alluvium underlying the mat foundations was estimated using methods based on elastic theory.

We used the shear wave velocity data obtained from BH-2 to estimate the elastic properties of the soil profile with depth. We also used a net change in vertical stress due to mat loading of 3 ksf. Estimates of elastic compression were obtained using several methods. In all cases, our calculations suggest the compression of the underlying alluvium will be less than ¼ inch. However, for design we recommend assuming a maximum total compression of ¾ inch and differential settlement across the mat of ½ inch. This increase will account for zones of less dense material (i.e., sand) that were encountered at various depths across the site.

6.2.3. *Sliding Coefficient*. The friction angle between the bottom of the concrete and imported granular fill or native alluvium is estimated to be 27 degrees. Therefore, a sliding coefficient of 0.5 is recommended for design.

6.2.4. *Passive Resistance*. An ultimate passive pressure coefficient (K_p) of 3.7 is estimated for design of the compacted backfill in front of the mat foundations. However, it is unlikely that the structures will move laterally enough to fully mobilize the passive resistance. Therefore, we have applied a reduction factor of 2, which results in an allowable K_p of 1.85. Therefore, in terms of equivalent fluid pressure, a value of 240 pcf may be used for design.

6.2.5. *Modulus of Subgrade Reaction*. The modulus of subgrade reaction (k_s) was estimated based on predicted elastic compression values discussed in Section 6.2.2. We recommend a k_s of 350 pci for design of the mat foundations. This value assumes the base of the mats will bear on dense to very dense alluvium.

6.3. *Shallow Foundations*

6.3.1. *Allowable Bearing Capacity*. Conventional spread footings will be used to support the smaller, single-story support structures and water tanks. We estimated the bearing capacity of isolated spread and continuous footings placed a minimum of 24 inches below the exterior grade on imported granular fill or dense alluvium. We assumed an effective angle of internal friction of 35 degrees to represent the drained strength of both materials. Recommended allowable bearing capacities for several footing sizes and depths are summarized in Table 3. A one-third increase in the allowable value is permitted for transient (i.e., wind and seismic) loading.

The bearing capacity analyses assume the continuous (wall) footings will have a minimum width of 24 inches and will bear on a minimum of 6 inches of compacted select fill. In addition, we assumed the select fill would extend a minimum of 6 inches outside the footprint of the footing. Similar assumptions were made for isolated column footings.

Table 3. Recommended Allowable Bearing Capacities

Footing Type	Minimum Footing Embedment Depth (ft)	Size (ft x ft)	Allowable Bearing Capacity (ksf)
Spread	2.0	2 x 2	3.7
Spread	2.0	3 x 3	4.2
Spread	2.0	4 x 4	4.7
Spread	2.5	2 x 2	4.5
Spread	2.5	3 x 3	4.7
Spread	2.5	4 x 4	5.0
Continuous	2.0	2 x L	2.9
Continuous	2.0	3 x L	3.3
Continuous	2.5	2 x L	3.5
Continuous	2.5	3 x L	3.9

6.3.2. <u>Settlement</u>. We anticipate settlements for the isolated column and continuous footings will be relatively limited and will occur relatively quickly during construction. For design, we recommend assuming a maximum total settlement of ½ inch and a maximum differential settlement between columns of ¼ inch.

6.3.3. <u>Sliding and Passive Resistance</u>. Frictional resistance at the footing base and passive resistance along the footing face or grade beam may be used to resist seismic loads. A sliding coefficient of 0.5 is recommended for concrete footings placed on imported select fill. An allowable passive resistance, calculated using an equivalent fluid density of 240 pcf is recommended for grade beams and footings. The passive resistance assumes a minimum footing or grade beam depth of 2 feet. In addition, the value also assumes that the backfill placed around the foundation elements will consist of compacted granular fill. Passive resistance should be ignored if the backfill consists of native material.

6.4. *Slabs-on-Grade*

Concrete slabs for buildings should be supported on a minimum of 6 inches of compacted, granular site fill or select fill. We have assumed the building pad fill would be underlain by other approved fill materials (fill areas) or undisturbed, dense native alluvium (cut areas). A modulus of subgrade reaction of 350 pci is appropriate for design. Reinforce all floor slabs to reduce cracking and warping. Rebar, instead of wire mesh, is recommended. A vapor barrier is recommended beneath all slabs in moisture-sensitive areas.

6.5. *Retaining Walls*

Active, at-rest and passive resistances recommended for retaining wall design, if required, are discussed below. We have provided the earth pressures in terms of equivalent fluid density, since definitive wall heights and wall footing depths are unknown.

An at-rest earth pressure coefficient (K_o) of 0.43 is recommended for restrained walls. Assuming a total unit weight (γ_t) of 130 pcf for the wall backfill, an equivalent fluid density of 55 pcf is appropriate for design.

An active earth pressure coefficient (K_a) of 0.27 is estimated for the design of unrestrained walls. Assuming a total unit weight (γ_t) of 130 pcf for the wall backfill, an equivalent fluid density of 35 pcf is appropriate for design.

An ultimate passive pressure coefficient (K_p) of 3.7 is estimated for design of the compacted backfill in front of the footing toe or for below-grade portions of mat foundations. However, it is unlikely that the footing will translate laterally enough to mobilize the full passive resistance. Therefore, we have applied a reduction factor of 2, which results in an allowable K_p of 1.85. Therefore, in terms of equivalent fluid pressure, a value of 240 pcf may be used for design.

The lateral earth pressure values provided in this section assume the wall backfill and the backfill lying within the passive wedge in front of the footing toe will consist of compacted crushed rock and/or undisturbed, native alluvium. Retaining wall backfill placed within a distance to the wall equal to the wall height should be compacted using light, hand-operated tampers or vibratory plates. This approach will reduce the risk of creating residual earth pressures, which would exceed the design values. These pressures also assume no build up of hydrostatic pressure behind the wall, a level backfill and no surcharge.

7.0. PAVEMENT DESIGN

7.1. *General*

Subgrade materials for the energy center's interior roads and parking areas will consist of topsoil (mixtures of silt and gravel), stiff clay or dense to very dense, sandy gravel and cobbles. If encountered, all plastic clay should be overexcavated from the subgrade and replaced with compacted granular fill. All other materials should be compacted to a minimum of 95% relative compaction prior to placing fabric and rock.

The maximum dry density of ASTM D 698 (Standard Proctor) should be used as the standard for estimating relative compaction. It should be understood that the native gravels are likely to be too coarse for conventional laboratory testing. Therefore, evaluation of adequate subgrade compaction may require field observation or proof rolling in lieu of field density tests.

7.2. *Traffic*

Truck traffic estimates were provided by Kerry Adams, P.E., of Burns & McDonnell. Mr. Adams indicated the truck traffic will be relatively light, consisting of five, two-axle and one, three-axle trucks per-day which corresponds to an equivalent 18-kip axle load (E-18) of ±11,000. We have also included anticipated employee vehicle traffic consisting of cars and pickup trucks.

7.3. *Design*

The anticipated subgrade materials contain aggregates too coarse for conventional, laboratory CBR testing. Therefore, we have assumed a CBR value based on information reported in the literature for compacted silty sand and gravel subgrades. A CBR value of 6 was selected for analysis, although the actual value is likely higher. A CBR value of 6 correlates to a subgrade resilient modulus (M_r) of 9,000 psi.

We used a computer program (AASHTO '86 method), the assumed traffic and assumed CBR value to estimate pavement sections for the access roads and parking areas constructed during dry weather. A rigid pavement analysis was also completed for areas subjected to truck traffic. Our analysis suggests a minimum PCC thickness of 5 inches over a 6-inch base. However, we recommend a minimum PCC thickness of 6 inches for the intended heavy use. A summary of the pavement design is provided in Table 4.

Table 4. Summary of Flexible and Rigid Pavement Design

Traffic Area	Minimum Asphalt/Base Rock Thickness	Minimum PCC/Base Rock Thickness
Interior Roads (autos and trucks)	3.5" AC over 8" Base	N/A
Parking Areas (autos only)	2.5" AC over 8" Base	N/A
Truck Loading Docks	Not recommended	6" PCC over 6" Base

A 20-year design life was assumed for the analysis. However, a nominal 2-inch overlay should be planned at about 12 years. The Asphalt Institute (TAI) recommends overlaying flexible pavements when 60% of the structure life is used. Research has shown that overlaying pavements at that time is more cost-effective than a full-depth repair after the pavement has failed. An experienced engineer should inspect the pavement every 3 to 5 years to determine its condition and need for rehabilitation.

8.0. CONSTRUCTION RECOMMENDATIONS

During the winter months, the ground water at the site lies within a couple of feet of the ground surface and, in some locations, is perched on the surface. The high ground water coupled with typical wet weather construction challenges will make mass grading operations during the winter months difficult or impractical. Therefore, the following recommendations assume the earthwork and mat foundation excavations extending below existing grades will occur during dry weather (i.e., typically mid-July to mid-October).

8.1. *General Earthwork Specifications*

1. Select fill as defined herein should consist of imported 1 or ¾-inch minus, well-graded, crushed gravel or rock with less than 3% passing the #200 U.S. Sieve. We should be provided a sample of the intended fill for approval, prior to delivery to the site.

2. Granular site fill as defined herein should consist of imported, 3-inch minus, well-graded, crushed gravel or rock with less than 3% passing the #200 U.S. Sieve. We should be provided a sample of the intended fill for approval, prior to delivery to the site.

3. General site fill as defined herein should consist of approved soil that is free of organics, construction debris or expansive (plastic) clay. General site fill may consist of on-site borrow or imported materials consisting of well-graded blends of cobbles, gravel, sand and silt.

4. Base rock beneath flexible and rigid pavements should consist of imported, 1½ or 1-inch minus, open-graded, angular, crushed rock. The open-graded material is recommended to provide a capillary break between the subgrade and pavement structure. We should be provided a sample of the intended fill for approval, prior to delivery to the site.

5. Drain rock as defined herein should consist of imported, 2-inch minus, clean (less than 2% passing the #200 sieve), open-graded, gravel or rock. We should be provided a sample of the intended fill for approval, prior to delivery to the site.

6. Compact all fill in loose lifts not exceeding 12 inches. Thinner lifts may be required if light or hand-operated equipment is used. Compact all exposed granular subgrade, on-site fill and imported fill to a minimum of 95% relative compaction. The maximum dry density of ASTM D 698 (Standard Proctor) should be used as the standard for estimating relative compaction. The moisture content of predominately fine-grain fill should be adjusted to within ±2% of its optimum value prior to compaction. Efficient compaction of fine-grained soils will typically require the use of a padfoot or kneading roller to achieve the required compaction. The granular subgrade and imported granular fill will compact more efficiently with a smooth drum, vibratory roller. Field density tests should be run frequently to confirm adequate compaction of granular fills having a maximum particle size of 1½ inches, or less.

Compaction of granular fills or subgrade that contain aggregates too coarse for density testing should be observed by an FEI representative. The completed subgrade or fill should be proof-rolled using a loaded, 10-yd^3 dump truck. Areas of pumping or deflection observed beneath the truck wheels should be overexcavated and replaced with compacted select fill and proof-rolled again.

7. The separation geotextile specified beneath pavements should have a Survivability Class 2 and may consist of one of the following woven fabrics: Amoco 2006, Synthetic Industries Geotex 300ST or LINQ GTF 300. Other fabrics with property values greater than or equal to those listed may be used upon approval by FEI.

8. Filter fabric should consist of a non-woven geotextile with a grab tensile strength greater than 200 lb., an apparent opening size (AOS) of between #70 and 100 (US Sieve) and a permitivity greater than 0.1 sec^{-1}.

9. Overexcavate all test pits that extend under the energy facility, support structures and pavements. Replace the test pit backfill with compacted granular site fill or select fill. The test pit locations should be shown in the project plans for the contractor's reference for future mitigation.

10. Inform contractors that water infiltration is likely for excavations that extend below existing grades. Trenches and foundation excavations should be pumped dry prior to placing the backfill. Trench backfill that extends beneath pavements, foundations and hardscape should consist of compacted select fill. Shoring will be needed in all trenches to protect workers from sloughing or caving soils.

8.2. *Site Grading*

Bulk site fills will consist of materials specified in Items 1 through 3. Our current understanding of the proposed grading plan is to raise the site in the power block area to ±El. 290, which will require ±2 to ±5 feet of fill. We also understand the top of the perimeter berms will lie at ±El. 290. Finish grade in the switchyard area is expected to lie at ±El. 286, thereby requiring up to ±2 to 3 feet of fill. Finish pavement grades were not yet known. However, we anticipate similar fill depths will be required.

We recommend the mass site grading and earth berm construction be completed during dry weather as follows:

11. Strip the existing ground as required to remove the vegetative layer. Dispose of all spoils outside of construction areas.

12. Complete the site grading using fills described in Items 1 through 3. Following stripping and prior to placing the first lift of site fill, moisture condition and compact the granular subgrade to a depth of at least 12 inches as specified in Item 6.

13. Moisture condition all fill during placement. Place all site fill in loose lifts not exceeding 12 inches. Compact and test each lift for adequate compaction as specified in Item 6. Each lift should be approved by an FEI representative prior to placing subsequent lifts.

14. Place a minimum of 6 inches of select fill in the respective building locations (after mass grading) to create the building pads, and beneath other isolated, lightly loaded structures. We recommend that the building pads be built up above the surrounding ground surface to promote site drainage away from the structures. Compact the building pad fill as specified in Item 6.

15. Place a minimum of 12 inches of select fill beneath slabs for water tanks, the cooling tower basin and transformer pads. Compact the fill as specified in Item 6.

8.3. ***Perimeter Earth Berm and Detention Pond Construction***

Earth berms are planned around the perimeter of the site to protect the area from potential flooding. Based on the draft grading plan (dated 2/20/02), the top of the berm will lie at El. 290. A maximum berm height of ±7 feet above the present grade will occur at the northwest portion of the site.

A portion of the north berm will be used to support a new side track that will spur from the Union Pacific Railroad line that runs along the east side of the site. Union Pacific Railroad has specific requirements for construction of earth berm supported side tracks. Those requirements were not available at the time this report was released and, therefore, will be addressed in a report addendum, if required.

Material for the earth berms will be generated from a proposed storm water detention pond which will be located in the northwest corner of the site. The proposed bottom pond elevation is El. 265, which means most of the excavation will take place below the ground water table, presently ±2 to 3 feet below present grade (±El. 280). Therefore, most of the excavation will occur below water which will make grading the interior slopes difficult. It is likely the slope material will slough into the excavation and become segregated prior to removal from the pond. Since the excavated material will consist of variable mixtures of silt, sand, gravel and cobbles, the material will need to be temporarily spread, drained and blended prior to placement and compaction.

Berm construction will require frequent density tests on each lift to verify adequate compaction. The initial lift should be placed on terrain that is properly stripped of vegetation, moisture conditioned and compacted. The compacted ground surface should be left in a rough condition to promote blending with the first lift of berm fill. We recommend overbuilding the slopes to provide compaction at the face of the finish slope. Finish slopes should be no steeper than 1.5(H):1(V). The slopes should be seeded, watered and maintained as soon as possible to provide erosion protection prior to the onset of wet weather. In the event the outside berm slopes require protection from turbulent flood water, riprap should be considered in these areas. Sizing of the riprap should be based on the anticipated flow velocity.

16. Excavate materials from the storm water detention pond. Spread the material in an area to allow the water to drain. Prior to placement in the perimeter berms (or other structural areas), the material should be blended to achieve a good grain size distribution.

17. Construct the perimeter berms using general site fill (Item 3). This will likely be stockpile materials generated from Item 16. The berm materials should be placed in maximum 12-inch thick lifts. Compact each lift in accordance with Item 6. Each lift should be proof rolled.

18. Immediately seed and hydrate the new slopes. Water as required to produce a mature cover of grass on the slopes prior to the onset of wet weather.

8.4. ***Mat Foundation Construction***

The CT, HRSG and STG units will be placed on mat foundations. Design criteria are provided in Section 6.2. We have assumed the base of the mats will extend a minimum of 3 feet below finish grade, but should also be kept as high as possible to minimize the dewatering effort. The contractor should anticipate dewatering these large excavations and be prepared to install a sufficient number of sumps and length of perimeter drain pipe to maintain a dry excavation. General dewatering recommendations are provided in this section, but may require modification depending on actual field conditions. Mat foundation construction and dewatering should be in general accordance with the following recommendations.

19. Design all mat foundations as specified in Section 6.2. The design criteria assume that the granular site fill underlying the mats will extend to dense to very dense alluvium. We recommend keeping the base of the mats as high as possible to reduce the dewatering effort, but no higher than 3 feet below finish grade.

20. Excavate for the mat foundations. The excavation should extend below the base elevation to accommodate a minimum of 12 inches of compacted, granular site fill. Grade the bottom of the excavation to drain towards the perimeter. The excavations should extend a minimum of 3 feet beyond the edge of the mat to allow for formwork and dewatering equipment.

21. Slope the perimeter of the excavation to maintain stable cut slopes that are in accordance with Oregon OSHA (OR-OSHA) requirements. There is a high risk of caving in locations where ground water seeps along the face of the cut. It is the contractor's responsibility to maintain stable cut slopes throughout construction. Recommendations for cut slopes can be made by FEI, if desired, once final excavation depths are known.

22. Do not compact the base of the excavation unless disturbed during excavating and requested by an FEI representative.

23. Install temporary sumps and collection pipes, as required, to dewater the excavation prior to placing granular site fill.

    Each contractor will likely have their own dewatering plan. However, one approach discussed with the design team is to install a rigid, perforated collection pipe around the perimeter of each excavation. The pipe should be wrapped in a fabric meeting the requirements of Item 8. The pipe should contain the required amount of risers that will extend above the granular pad. Sumps can be placed in the risers for dewatering.

24. Place the required amount of granular site fill to raise the base of the excavation to the required grade. Compact as specified in Item 6.

25. Backfill around the mats with compacted select fill or granular site fill and compact as specified in Item 6.

8.5. <u>*Shallow Foundation Construction*</u>

Shallow foundations and slabs will be used to support the remaining structures (except the switchyard towers). Design criteria are summarized in Section 6.3 (shallow foundations) and Section 6.4 (slabs). Shallow foundation construction should be in general accordance with the following recommendations.

26. Provide a minimum footing width of 24 inches for all footings and a leveling course of at least 6 inches of compacted select fill under all footings. The fill under the footings should extend at least 6 inches beyond the edges of the footings. Place the base of all footings at least 24 inches below the finished grade or paved surface.

    In some instances, the base of the footings will lie in the imported fill used to raise the site. The 6-inch leveling coarse of compacted select fill is still required for these areas.

27. Construct the granular pads for the respective structures as specified in Items 14 and 15. Excavate for footings using a hoe equipped with a smooth bucket to reduce subgrade disturbance. Dewater the excavations prior to placing backfill, if required. Backfill the footings with a minimum of 6 inches of compacted select fill. Compact the footing backfill using light, hand-operated equipment.

28. Overexcavate any organics, debris or plastic clay encountered in footing excavations. The overexcavation should extend to native alluvial gravel and the depth should be evaluated by us in the field. Backfill the overexcavated areas using compacted, granular site fill or select fill.

29. Provide a suitable vapor barrier under building slabs that is compatible with the proposed floor covering (if any) and the method of slab curing. The type and placement of the vapor barrier depends on the method of slab curing and schedule for installing the floor surfaces. Therefore, this item should be reviewed by the flooring manufacturer, contractor and project engineer and/or architect.

30. Provide a minimum of 6 inches of compacted select fill under all other isolated concrete slabs and sidewalks for dry weather construction.

8.6. *Retaining Wall Construction*

Design criteria for retaining walls are summarized in Section 6.5. Retaining wall construction should be in general accordance with the following recommendations.

31. Excavate for the retaining wall footings. The footing depths should accommodate a minimum of 6 inches of select fill beneath the footing. Place and compact the select fill as specified in Item 6.

32. Install a drainage system behind all walls to alleviate hydrostatic pressure buildup. The system should consist of a 3 or 4-inch diameter, perforated PVC pipe wrapped in a geotextile fabric meeting the requirements in Item 8. The pipe should be bedded in 12 inches of drain rock (Item 5) that is wrapped in a similar geotextile and laps 12 inches at the top.

33. Backfill the remaining area behind the wall using select fill compacted to 92% relative compaction. The maximum dry density of ASTM D 698 (Standard Proctor) should be used as the standard for estimating relative compaction.

34. Keep heavy rollers and other construction equipment a safe distance back from all walls. To avoid compaction surcharge loads, heavy equipment should be kept back a distance equal to the wall height. Compaction within this zone should be completed on relatively thin lifts (6 inches or less) with light compactors such as vibratory plates or walk behind rollers.

35. Backfill placed in front of the wall toe should consist of compacted select fill if the passive toe resistance is to be used for design, or if pavements or other structures extend over the wall toe. Otherwise, the area may be backfilled with compacted, general site fill meeting the requirements of Item 3.

8.7. *Pavement Construction*

Pavement and base rock thicknesses for several pavement sections are summarized in Table 4, Section 7.3. Subgrade preparation beneath interior road alignments and parking areas should be in general accordance with the following recommendations.

36. Prepare the pavement subgrade and/or place fill material to raise the grade in accordance with Section 8.2.

37. Proof-roll the subgrade using a loaded 10-yd^3 dump truck prior to placing fabric and rock. FEI should be retained to provide the proof-rolling observation. Areas of pumping or deflection should be overexcavated and replaced with compacted, granular site fill or select fill.

38. Place a separation geotextile on the exposed subgrade meeting the requirements of Item 7. The geotextile should be laid smooth, without wrinkles or folds in the direction of construction traffic. Overlap adjacent rolls a minimum of 2 feet.

39. Construct pavement areas using the appropriate section from Table 5. We should be contacted to re-evaluate the recommended sections if the anticipated traffic changes from that stated in Section 7.3.

8.8. *Foundation Drains*

40. Install foundation drains along the perimeter of all buildings and other structures supported on spread footings. The drains should consist of 3 or 4-inch diameter, perforated or slotted, PVC pipe wrapped in a non-woven filter fabric with an AOS of between 70 and 100. The flowline of the pipe should be set along the outside edge of the footing base. The pipe should be bedded in at least 4 inches of drain rock and backfilled to within 12 inches of the ground surface. The entire mass of drain rock should be wrapped in a filter fabric (meeting the requirements of Item 8) and lapped at least 12 inches at the top.

41. Provide clean-outs at appropriate locations for future maintenance of the drainage system.

42. Discharge the water from the drain system into the nearest catch basin, manhole or storm drain.

9.0. DESIGN REVIEW/CONSTRUCTION OBSERVATION/TESTING

We should be provided the opportunity to review all drawings and specifications that pertain to site preparation, earthwork, foundation construction and pavements. Site preparation and earthwork will require field confirmation of topsoil stripping, subgrade preparation and fill placement in accordance with recommendations provided herein. Mitigation of any subgrade pumping or persistent ground water infiltration will also require engineering review and judgment. That judgment should be provided by one of our representatives. Frequent field density tests should be run on all imported granular fill and base rock. Fills too coarse or variable for density testing should be proof-rolled as recommended above. We recommend that we be retained to provide the necessary construction observation and testing.

10.0. VARIATION OF SUBSURFACE CONDITIONS, USE OF THIS REPORT AND WARRANTY

The analyses, conclusions and recommendations contained herein are based on the assumption that the soil profiles and the ground water levels encountered in the borings and test pits are representative of overall site conditions. The above recommendations assume that we will have the opportunity to review final drawings and be present during construction to confirm assumed foundation conditions. No changes in the enclosed recommendations should be made without our approval. We will assume no responsibility or liability for any engineering judgment, inspection or testing performed by others.

This report was prepared for the exclusive use of Calpine Corporation and their design consultants for the Turner Energy Center project in Turner, Oregon. Information contained herein should not be used for other sites or for unanticipated construction without our written consent. This report is intended for planning and design purposes. Contractors using this information to estimate construction quantities or costs do so at their own risk. Our services do not include any survey or assessment of potential surface contamination or contamination of the soil or ground water by hazardous or toxic materials. We assume that those services, if needed, have been completed by others.

Our work was done in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

11.0. REFERENCES

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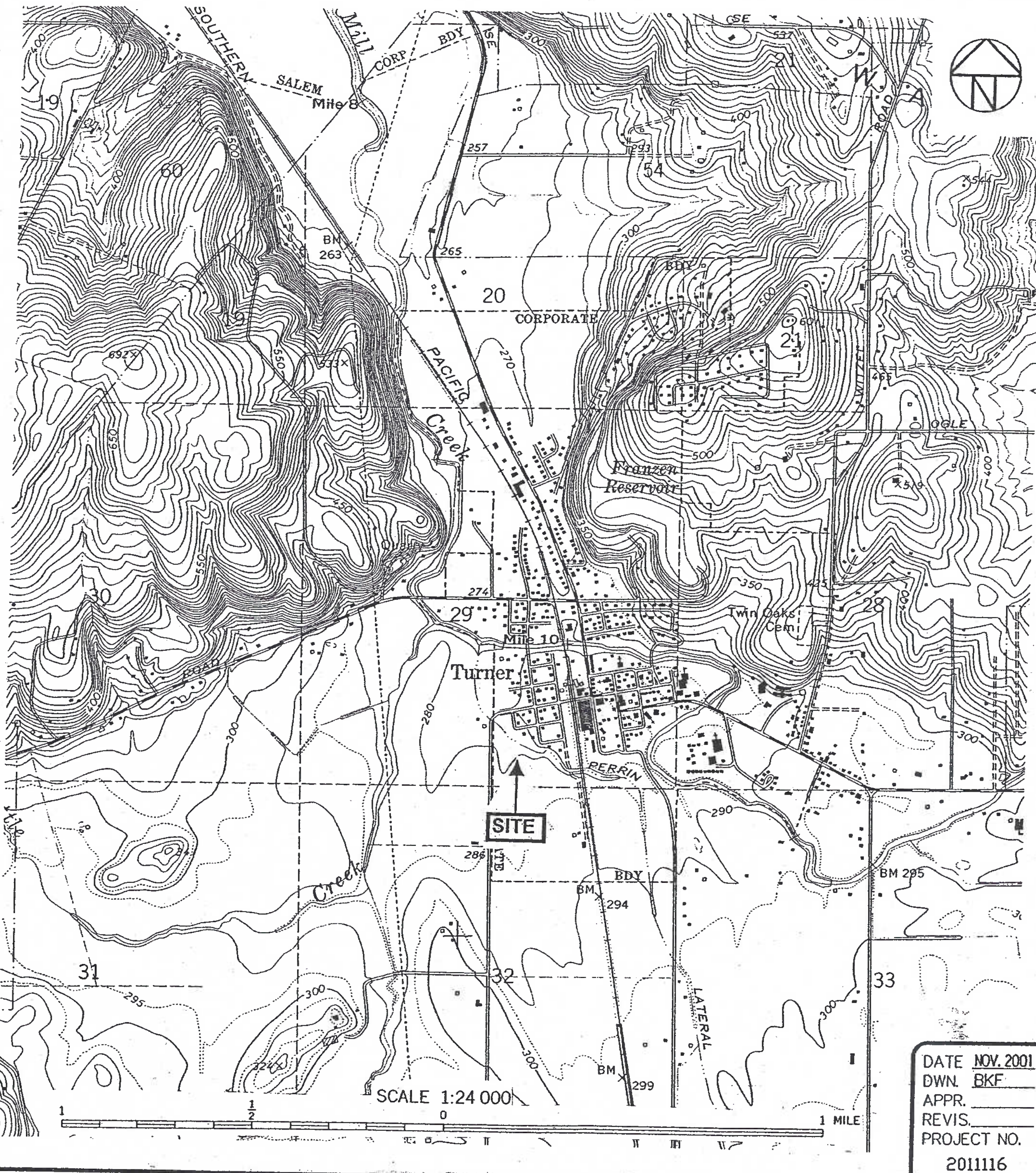
Appendix A

Figures

Professional Geotechnical Services

Foundation Engineering, Inc.



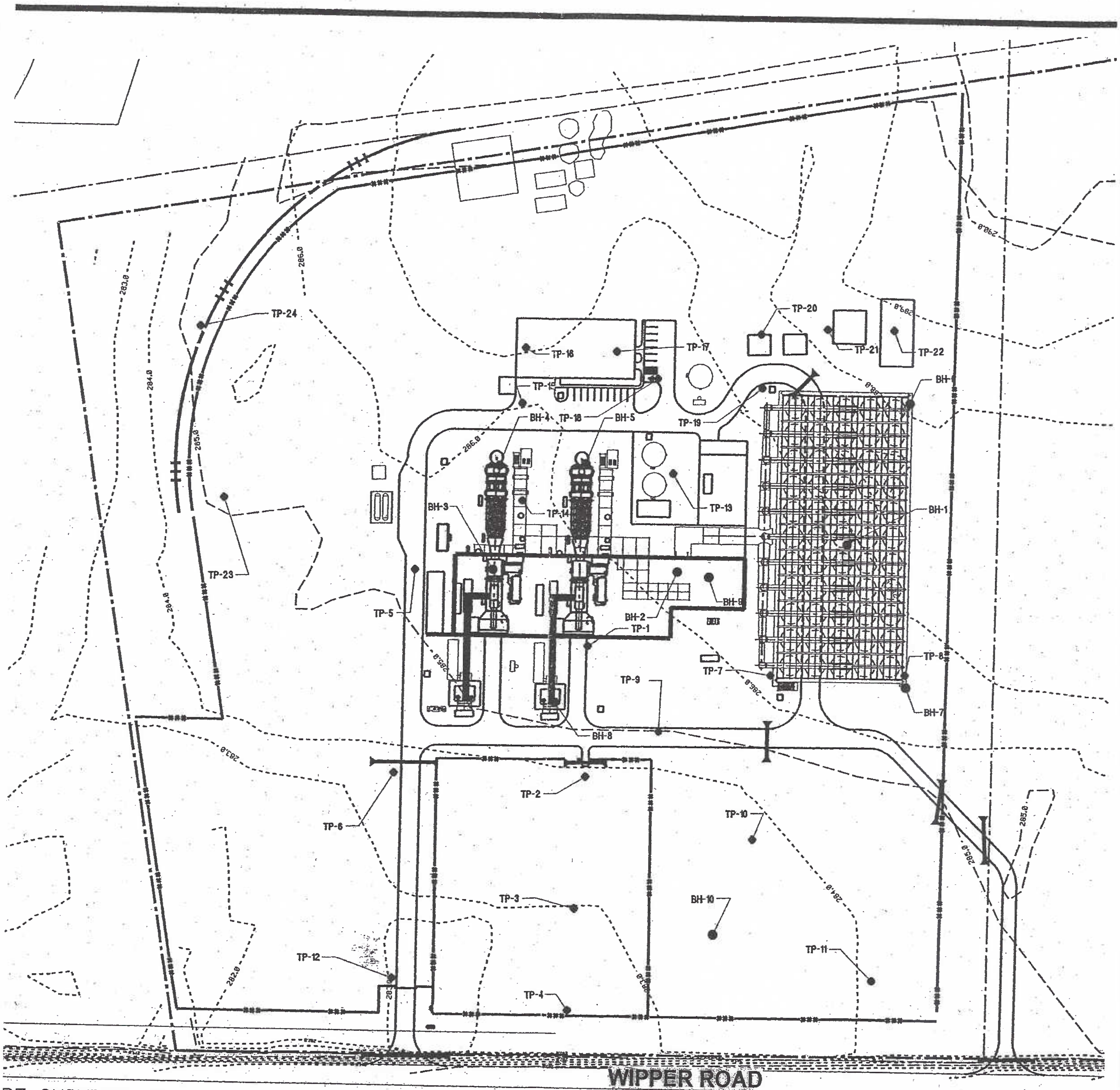
DATE NOV. 2001
DWN. BKF
APPR.
REVIS.
PROJECT NO.
2011116

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES
820 NW CORNELL AVENUE
CORVALLIS, OR 97330-4517
BUS. (541) 757-7645 FAX (541) 757-7650

VICINITY MAP

TURNER ENERGY CENTER
TURNER, OREGON

FIGURE NO.
1A



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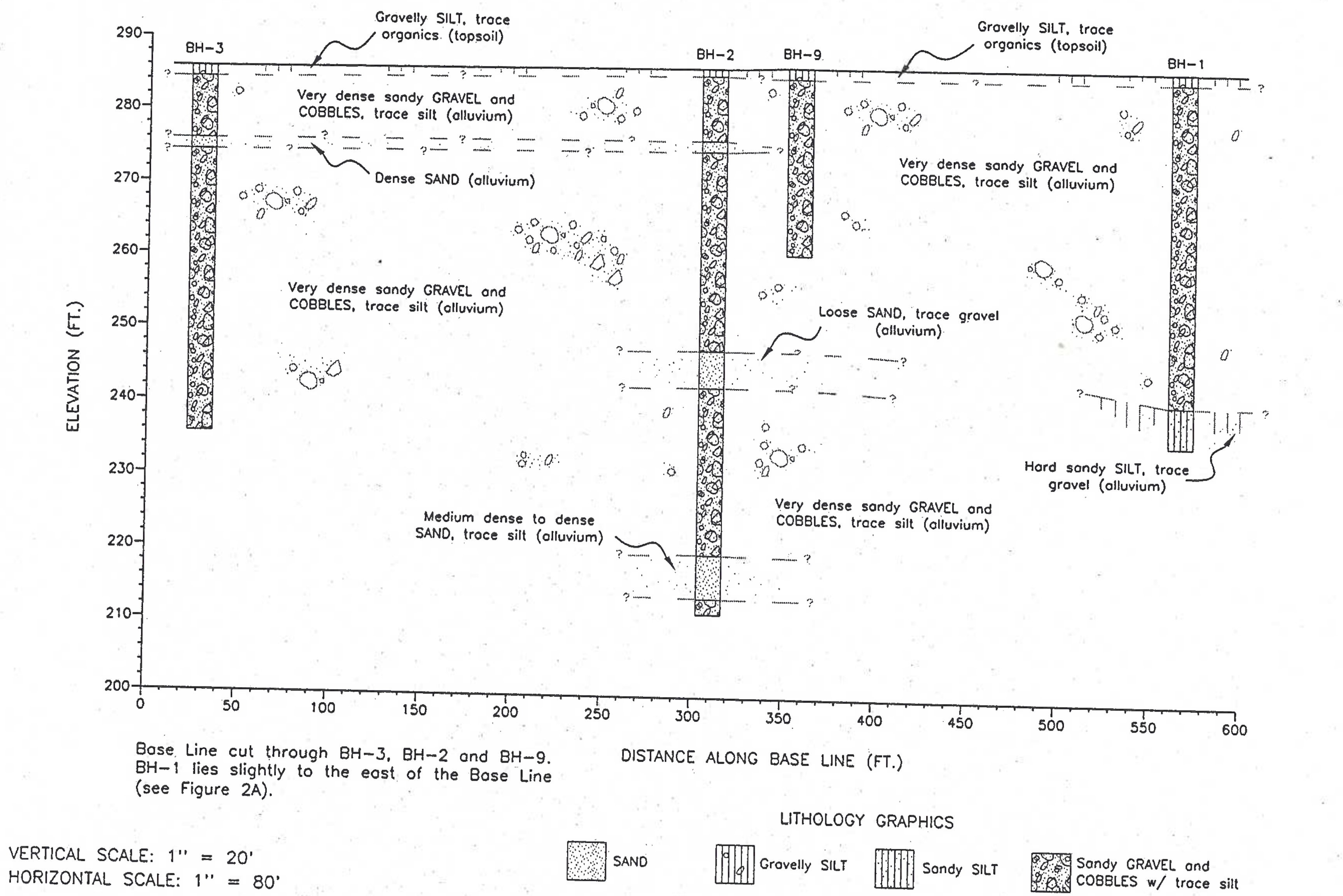
BORING AND TEST P

FIGURE 2-A

TURNER ENERGY
TURNER, ORE



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DATE Jan 2002
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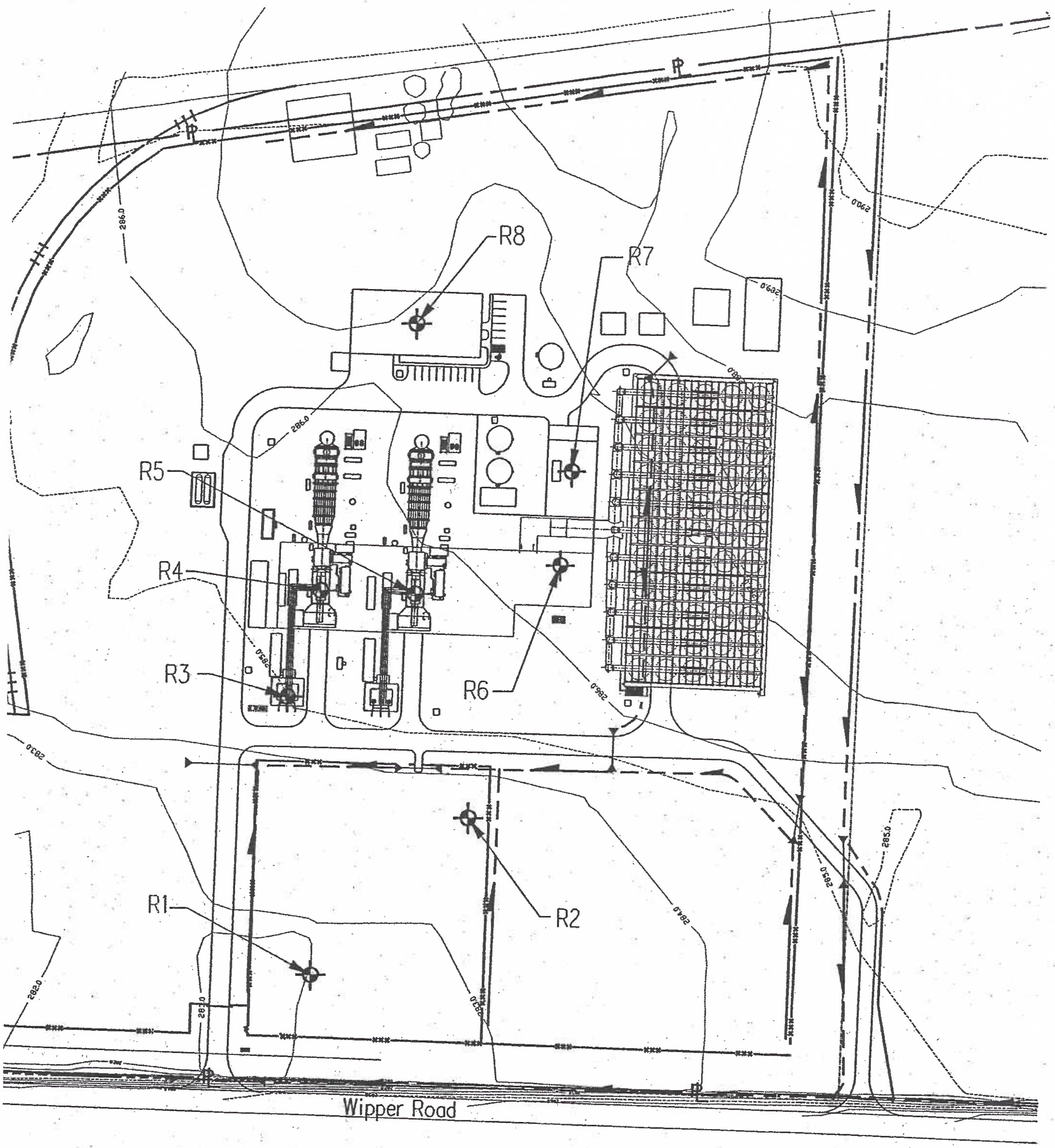
REVIS.
PROJECT NO.
201-1-116

INTERPRETED SUBSURFACE PROFILE

North - South Cross-Section

Turner Energy Center
Turner, Oregon

FIGURE NO.
3A



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DATE Feb. 2002
DWN. wln
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2011116

RESISTIVITY TEST LOCATIONS

Turner Energy Center
Turner, Oregon

FIGURE NO.
4A