Appendix C

Field and Laboratory Testing

Professional Geotechnical Services

Foundation Engineering, Inc.

Foundation Engineering, Inc.
Turner Energy Center
Project 2011116

Table 1C. Piezometric Readings

Boring Number	Date	Surface Elevation (feet)	Water Depth (feet)	Water Elevation (feet)
BH-1	10/9/01	287.4	3.9	283.5
	1/2/02		0.9	286.5
BH-3	10/10/01	286.1	4.0	282.1
	1/2/02		1.3	284.8
BH-4	10/17/01	285.7	3.2	282.5
	1/2/02		0.0	285.7
BH-5	10/17/01	287.1	3.8	283.3
	1/2/02		0.9	286.2
BH-6	10/18/01	288.9	5.1	283.8
	1/2/02		2.1	286.8

Notes: Elevation provided by Northstar Surveying, Inc.
Surface elevation refers to the top of the Morris monument.
Water depth measured from the top of the Morris monument.

Table 1C. Piezometric Readings

Boring Number	Date	Surface Elevation (feet)	Water Depth (feet)	Water Elevation (feet)
BH-7	10/22/01	287.5	4.1	283.4
	1/2/02		1.4	286.1
BH-8	10/19/01	286.1	3.5	282.6
	1/2/02		0.8	285.3
BH-9	10/18/01	287.1	4.1	283.0
	1/2/02		1.4	285.7
BH-10	10/19/01	285.1	3.0	282.1
	1/2/02		0.6	284.5

Note: Elevation provided by Northstar Surveying, Inc.
Surface elevation refers to the top of the Morris monument.
Water depth measured from the top of the Morris monument.

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Project 2011116

Table 2C. Results of Field Resistivity Tests

Test[1] Location/Pin Direction	Test Depth (feet)	Average Resistivity (ohm-cm)
R-1/East-West	5	5,649
	10	14,746
	15	15,224
	20	17,235
	25	20,586
	30	22,406
R-2/East-West	5	28,821
	10	34,470
	15	34,470
	20	34,470
	25	34,470
	30	32,747
R-3/North-South	5	44,045
	10	34,470
	15	34,470
	20	31,023
	25	31,598
	30	31,598

[1] Refer to Figure 4A for a location of the resistivity tests.

Table 2C. Results of Field Resistivity Tests

Test[1] Location/Pin Direction	Test Depth (feet)	Average Resistivity (ohm-cm)
R-4/East-West	5	62,238
	10	45,960
	15	43,088
	20	42,130
	25	36,864
	30	32,747
R-5/East-West	5	50,748
	10	38,300
	15	37,343
	20	34,470
	25	34,470
	30	33,321
R-6/North-South	5	42,130
	10	42,130
	15	43,088
	20	42,130
	25	38,300
	30	37,917

[1] Refer to Figure 4A for a location of the resistivity tests.

Table 2C. Results of Field Resistivity Tests

Test[1] Location/Pin Direction	Test Depth (feet)	Average Resistivity (ohm-cm)
R-7/North-South	5	46,918
	10	59,365
	15	50,269
	20	53,620
	25	52,663
	30	48,833
R-8/North-South	5	44,045
	10	33,513
	15	37,343
	20	38,300
	25	42,369
	30	42,513

[1] Refer to Figure 4A for a location of the resistivity tests.

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Project 2011116

Table 3C. Summary of Corrosion Potential Constituents*

Analysis	Method	Sample	
		S-14-1 @ ±5 feet	S-19-1 @ ±5 feet
pH	Electrometric	5.9	5.6
Minimum Resistivity (Ω-cm)	Specific Conductance	11,100	8,930
Redox Potential (milivolts)	ASTM D 1498	595	560
Chloride (ppm)	Turbidimetric	1.7	2.4
Sulfate (ppm)	SM 4500-SO E	10	7
Sulfide (ppm)	Lead acetate paper	< 2	< 2

*Tests completed by MEI Charlton, Portland, Oregon.

Appendix D

Seismic Site Hazard Investigation

Professional Geotechnical Services

Foundation Engineering, Inc.

Seismic Site Hazard Investigation

Turner Energy Center

Turner, Oregon

Prepared for:

Calpine Corporation
Pleasanton, California

February 8, 2002

Foundation Engineering, Inc.
Professional Geotechnical Services

Jim McLucas
Calpine Corporation
6700 Koll Center Parkway, Suite 200
Pleasanton, California 94566

February 8, 2002

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

Project 2011116

Dear Mr. McLucas:

We have completed the requested seismic hazard study for the above-referenced project. The study was completed to identify potential seismic hazards and evaluate the effect those hazards might have on the proposed site. Our work 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. The guidelines have been adopted by the Oregon Department of Geology and Mineral Industries (DOGAMI) who will peer review this study for the Oregon Department of Energy.

Our report includes a description of our work, a discussion of site conditions, and provides our final conclusions regarding seismically-induced geologic hazards at the proposed site. A report summarizing the geotechnical investigation and recommendations for foundation design and construction is in progress.

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

REGISTERED PROFESSIONAL ENGINEER
60776 PE
OREGON
MAY 17, 1999
WILLIAM L. NICKELS JR.
EXPIRES: 12/31/02

James K. Maitland, P.E.
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Foster Gambee Geotechnical, Inc.

CERTIFIED
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TURNER ENERGY CENTER
TURNER, OREGON

1.0. PURPOSE AND SCOPE

The purpose of this study is to identify potential seismic hazards and evaluate the effect those hazards may have on the proposed site in Turner, Oregon. Our work is based on available scientific literature, subsurface conditions as identified from information obtained in 10 borings and 24 test pits completed on-site, and from geophysical data collected in one of the borings. In general, work related to the seismic investigation included the following tasks:

1. A literature review of published papers, maps, open-file reports, earthquake records and catalogs addressing the tectonic setting, regional and local geology, and historical seismic activity in the vicinity of the proposed site.

2. An evaluation of site-specific subsurface information and geophysical data obtained by Foundation Engineering, Inc. (FEI). Based on the site characterization, a conceptual geologic cross-section was prepared and used in the dynamic analyses.

3. Identification of the potential seismic activity that may affect the site. The magnitude of the events were compared to the minimum requirements outlined in the 1998 State of Oregon Structural Specialty Code (OSSC), Chapter 18 for selecting three scenarios or design seismic events for evaluation.

4. Based on the information obtained in Tasks 1, 2 and 3, analyses and evaluations were made regarding:

   - seismically-induced geologic hazards that may affect the site
   - the risk of cyclically induced soil liquefaction
   - dynamic soil response and possible ground motion amplification
   - site-specific bedrock and soil surface acceleration response spectra for the three postulated earthquakes
   - water related hazards (tsunami and seiche) as required by DOGAMI

2.0. PEER REVIEW

Information gathered from our literature search of the project site's geologic setting, faulting and seismicity was peer reviewed by Kevin Foster, P.G., C.E.G., P.E. of Foster Gambee Geotechnical, Inc. Technical issues contained herein were reviewed by Stephen Dickenson, Ph.D., Associate Professor of Civil Engineering at Oregon State University.

3.0. PROJECT BACKGROUND

Calpine Corporation (Calpine) plans to construct a 620-megawatt natural-gas fired combustion turbine energy plant on the southwest edge of Turner. The area is made up of Tax Lots: 500, 600, 700, 800, 900 and 1000 that are generally located within T8S, R2W, NE quarter of Section 32 – Willamette Meridian, Marion County. We understand Calpine will purchase ±97 acres, but the bulk of the plant will be sited on the northern ±44 acres. The ±44 acres located east of Wipper Road within the Turner city limits will be referred to from hereon as the site. The project location is shown on the Vicinity Map, Figure 1A, Appendix A.

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.

Calpine Corporation is the project owner. FEI was retained by Calpine to complete a geotechnical investigation and seismic site hazard investigation for the site.

4.0. FIELD EXPLORATION

The field exploration program consisted of 10 borings and 24 test pits. The borings and test pits TP-1 through TP-8 were completed on 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.

4.1. *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 seismic and shear wave velocity survey. All installations were capped at the ground surface with a locking Morris monument set in concrete.

4.2. *Test Pits*

The test pits were dug with a rubber-tired, Case 580 extend-a-hoe. The deeper test pits extended to maximum depths of ±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.

5.0. DISCUSSION OF SITE CONDITIONS

5.1. *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.

5.2. *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.

5.2.1. *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 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).

5.2.2. *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.

5.2.3. *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.

5.3. ***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 north bound 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.

5.4. ***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 (Appendix C).

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

6.0. GEOLOGIC SETTING

6.1. *Literature Review*

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

6.2. *Regional Geologic and Tectonic Setting*

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).

The Willamette Valley lies approximately 90 miles inland from the surface expression of the Cascadia Subduction Zone, a converging plate boundary where the Juan de Fuca plate is being subducted beneath the western edge of the North American continent. Available information indicates this subduction zone is capable of generating earthquakes within the descending Juan de Fuca plate (intraplate earthquakes), along the inclined interface between the two plates (interplate or subduction zone earthquakes), or within the overriding North American Plate (crustal earthquakes) (Weaver and Shedlock, 1996).

6.3. ***Local Geology, Hazards and Faulting***

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.

A number of faults lie within ±20 miles of the site. A map showing the locations of known local, crustal faults in the Mid-Willamette Valley is presented on Figure 4A, Appendix A. A brief discussion of each fault is provided in the following subsections.

6.3.1. *Mill Creek Fault*. The closest, potentially active fault lies less than 1 mile north of the site. The ±10-mile long Mill Creek Fault (combined with the Turner Fault) lies along two linear segments of the southern front range of the Waldo Hills. The Mill Creek Fault is not exposed at the surface, rather, it is identified from water well and oil logs (Geomatrix, 1995). Although the northeasterly-trending, normal fault shows no evidence of late Pleistocene or Holocene movement, earthquakes occurring along the fault may have long recurrence intervals. Therefore, Geomatrix (1995) suggests the fault may still be active. No other faults have been identified in the Stayton Basin.

6.3.2. *Waldo Hills Fault*. The northeasterly-trending Waldo Hills Range-Front Fault traces ±4 miles north of Turner. This ±7-mile long, buried reverse fault has shown no evidence of movement within the last 28,000 years (late Pleistocene to Holocene) (Geomatrix Consultants, 1995).

6.3.3. *Mt. Angel Fault*. The ±15 to 20-mile long Mt. Angel Fault is located ±15 miles northeast of Turner. This northwest-trending, oblique fault is concealed beneath valley alluvium (Yeats et al., 1996). The Mt. Angel Fault has shown Quaternary to late Quaternary movement (within the last 780,000 to 1.6 million years) (Geomatrix Consultants, 1995). The 1993 Scotts Mills earthquake occurred ±5 miles south of the southern end of the Mt. Angel Fault. It is not known if the earthquake occurred on an extension of the Mt. Angel Fault or if it occurred on a parallel structure.

6.3.4. <u>Corvallis Fault</u>. The northern extent of the ±34-mile long, northeast-trending Corvallis Fault is located ±20 miles southwest of Turner (Yeats et al., 1996). Faulting has been ongoing since the Eocene, with the most recent detectable movement occurring more than ±28,500 years ago (Geomatrix, 1995). There have been three historic earthquakes with intensities of V, III and III-IV located near the Corvallis Fault trace (Bela, 1979).

6.3.5. <u>Other Faults</u>. Other concealed faults, including the East Albany and Beaver Creek faults, lie within ±20 miles of the site (Yeats et al., 1996). However, they are not considered active (Geomatrix, 1995).

The trace of the Waldo Hills Range-Front Fault aligns with the Mill Creek and Turner Faults, and the potentially active Corvallis Fault to the southwest, possibly forming a ±58-mile long rupture zone (Geomatrix, 1995). Although these faults appear connected along the same northeasterly trend, the potential for simultaneous rupture along the entire length of the compound fault is thought to be low (Geomatrix, 1995).

Although there is no indication of current faulting beneath the site, hidden and/or deep-seated active faults could remain undetected. Additionally, recent crustal seismic activity cannot always be tied to observable faults. In the event of a catastrophic earthquake with a large seismic moment, inactive faults could potentially become reactivated.

6.4. <u>**Seismicity**</u>

Because the geologic and seismologic information available for identifying the nature of the seismicity at the site is incomplete, it is difficult to accurately predict the probable magnitude, location, and frequency of earthquakes that might affect the site.

No significant interplate (subduction zone) earthquakes have occurred in historic times, however, several large-magnitude subduction zone earthquakes are thought to have occurred in the past few thousand years. Interplate earthquakes are believed to have an average return period of 400 to 700 years (Nelson and Personius, 1996), with the last event occurring ±300 years ago (Nelson et al., 1995). The maximum estimated magnitude of a subduction zone earthquake ranges from M 8.5 to M 9.0 (Wang and Leonard, 1996).

Intraplate earthquakes occur within the Juan de Fuca Plate at depths of 25 to 40 miles. The maximum estimated magnitude of an interplate earthquake is about M 7.5. The Puget Sound region has experienced three intraplate events in modern times, magnitudes M 7.1 in 1949 and M 6.5 in 1965 (Wang and Leonard, 1996), and M_w 6.8 in 2001 (USGS National Earthquake Information Center).

Crustal earthquakes dominate Oregon's seismic history. Crustal earthquakes occur within the North American Plate typically at depths of 6 to 12 miles. The estimated maximum magnitude of a crustal earthquake is about M 6.5 (Wang and Leonard, 1996). Only three of the crustal major events have reached Richter local magnitude (M_L) 6, with the majority in the M_L 4 to 5 range. Table 2 lists crustal earthquakes greater than or equal to M_L 3.5 that have occurred within a ±50-mile radius of Turner over the last 150 years (Johnson et al., 1994).

Table 2. Historic Earthquakes within a 50-mile Radius of Turner

Year	Month	Day	Hour	Latitude	Longitude	Depth (km/mi)	Magnitude (M_L)
1995	2	8	9	45.1	122.7	31.7/19.7	3.6
1993	6	8	1	45.0	122.6	20.2/12.6	3.7
1993	3	25	13	45.0	122.6	20.6/12.8	5.6
1963	3	2	23	44.9	123.5	47.0/29.2	4.6
1962	9	5	5	44.5	122.9	Unknown	3.5
1961	8	19	4	44.7	122.5	Unknown	4.5

A sample of distant strong earthquakes felt in the Turner area include the following (Modified Mercalli Intensities (MMI) in parentheses): the 2001 Nisqually, Washington earthquake (II to III); the 1993 Scotts Mills earthquake (VI); the 1965 Puget Sound earthquake (V); the 1962 Portland earthquake (I to IV); the 1961 earthquake northeast of Lebanon and Albany (V); the 1957 earthquake near Salem (I to IV); the 1949 Olympia, Washington earthquake (VI) (USGS, 2001; Wong and Bott, 1995; Bott and Wong, 1993).

7.0. SEISMIC DESIGN

7.1. *Design Earthquakes*

The OSSC, Section 1804, requires that structures classified as essential or hazardous facilities, and major and special-occupancy structures be evaluated for at least three different earthquakes with the following magnitudes:

Crustal: M_W = 6.0 minimum
Intraplate: M_W = 7.0 minimum
Interplate: M_W = 8.5 minimum

We reviewed current seismic information for the Turner area and defined the scenario or design earthquakes for this specific area (Weaver and Shedlock, 1996). The following earthquake magnitudes and source-to-site distances were selected:

1. Crustal Earthquake Source: M_w 6.5 at a depth or distance of 6.2 miles from the site.
2. Subduction Zone (Intraplate) Source: M_w 7.0, at a depth of 28 miles and a distance of 24 miles west of the site.
3. Subduction Zone (Interplate) Source: M_w = 8.8, at a depth of 24 miles and a distance of 49 miles west of the site.

The three earthquake sources described above were used to establish the design earthquake parameters for the site response analysis.

8.0. SITE RESPONSE ANALYSIS

8.1. *Approach*

FEI developed acceleration response spectra based on the three design earthquakes at the proposed site. Our approach consisted of the following steps:

1. Using the geologic and seismicity data provided above, we established ground motion characteristics for the following three earthquake sources: Random crustal, Intraplate (Subduction Zone), and Interplate (Subduction Zone).
2. Determined the maximum bedrock acceleration (A_{max}) and predominant period (T_p) for each design earthquake using empirical relationships.
3. Developed a target rock spectrum for each design earthquake.
4. Identified and selected recorded motions (i.e., earthquake acceleration time-histories) that either resemble the target spectrum shape or could be slightly modified to correspond to the target spectrum.
5. Used the SHAKE91 computer program for 1-D dynamic soil response (integrated with SHAKE2000 pre- and post-processor software) and developed a pseudo-acceleration response spectrum for each event using an average soil profile and available earthquake records.

8.2. *Design Earthquake Parameters (Item 1)*

The design earthquake parameters presented in Table 3 were used in the respective dynamic analysis. In addition, the OSSC states that the maximum bedrock acceleration for the design crustal earthquake should not be less than the Seismic Zone Factor (Z) which is 0.30 for the proposed site. Based on the Joyner & Boore predictive relationship, a magnitude 6.5 earthquake at a source-to-site distance of 6.2 miles will produce an A_{max} of ±0.30g. Therefore, a magnitude 6.5 earthquake was used to evaluate site response for the random crustal event.

According to the 1997 Uniform Building Code (UBC), the site is located in Seismic Zone 3 which has a Z of 0.30. Based on our subsurface investigation, interpretation of the site geology and geophysical data, we have classified the soil/rock profile type extending below the site as S_C. In addition, near Source Factors (N_a and N_v) of 1.0 are appropriate for the site.

Table 3. Design Earthquake Parameters

Postulated Earthquake Source	Design Magnitude	Source-to-Site Distance (miles)	Max. Bedrock Acceleration (g)
Random Crustal	6.5	6.2	0.30
Subduction (Intraplate)	7.0	24	0.18
Subduction (Interplate)	8.8	49	0.22

For comparison, Table 4 summarizes maximum accelerations (on rock) for the Turner area for a 500, 1,000 and 2,500-year design earthquake as published by Geomatrix (1995) and the USGS. These values suggest that for an event with a 500-year return period the maximum acceleration of 0.30g used for our random crustal analysis is conservative.

Table 4. Summary of Maximum Acceleration (on Rock) for the Turner Area as Published by Geomatrix and the USGS

Literature Source	500-year return	1,000-year return	2,500-year return
Geomatrix (1995)	0.18g	0.23g	0.33g
USGS (2002) (Web Site)	0.16g	0.22g	0.33g

Note: Accelerations are based on uniform, aggregate hazards from probabilistic studies.

8.3. *Attenuation Relationships (Items 2 and 3)*

Peak bedrock accelerations (PGA or A_{max}), predominate periods (T_p) and target (bedrock) response spectra were estimated using predictive ground motion equations or attenuation relationships. The selected relationships were derived using similar site-specific conditions such as: earthquake magnitude, source-to-site distance and focal depth. A target (rock) response spectrum was then developed using the selected relationships.

The random crustal design earthquake with a M_w 6.5 is a shallow or crustal-type event. Therefore, the empirical relationship presented by Joyner and Boore (1988) was chosen to estimate A_{max} and T_p, and to develop a target (crustal) spectrum for the site.

The M_w 8.8 and M_w 7.0 subduction zone earthquakes are assumed to be interplate and intraplate, respectively. For these larger magnitude and deeper earthquakes, the attenuation relationship by Youngs et al. (1997) was chosen to estimate A_{max} and T_p, and to develop the target response spectra.

8.4. *Selection of Strong Motion Records (Item 4)*

The target response spectrum for each earthquake has a unique shape, depending on the type of event, earthquake magnitude and distance to source. These spectra were compared to spectral shapes produced by available strong motion records that are similar in magnitude, distance, A_{max} and T_p. Based on the comparable shapes, suitable time-history records were selected that best represented the overall characteristics of each design earthquake. The input motions were scaled in SHAKE91 to match the estimated peak bedrock accelerations of the design events. The suite of selected time histories is listed in Table 5.

Table 5. Summary of Selected Strong Motion Records

Earthquake	Recording Station (orientation)	Magnitude	Distance (km)
Subduction Zone Events (Interplate)			
Michoacan (1985)	La Union (N90E)	8.1	82
Miyagi-Oki (1978)	Ofunato Bochi (E41S)	7.6	116
Valparaiso (1985)	Llolleo (160 deg.)	7.8	73
Subduction Zone Events (Intraplate)			
Earthquake	**Recording Station (orientation)**	**Magnitude**	**Distance (km)**
Adak, Alaska (1971)	Naval Station (N090E)	6.8	69
Loma Prieta (1989)	Agnews (0 deg)	7.1	44
Olympia (1949)	Station Not Listed (N86E)	7.1	56
Puget Sound (1965)	Olympia Hwy Test Lab (Orien. Not Listed)	6.8	89
Crustal Events			
Earthquake	**Recording Station (orientation)**	**Magnitude**	**Distance (km)**
Coalinga AS (1983)	Anticline Ridge (0 deg.)	5.3	13
Coalinga AS (1983)	Sulphur Baths (0 deg.)	6.0	13
Helena (1935)	Carroll College (SOOW)	6.0	8
Mammoth Lakes (1980)	Long Valley Dam (0 deg.)	6.2	16
San Fernando	Lake Hughes #4 (S69E)	6.6	26
San Fernando	CalTech Seismo. Lab (S90W)	6.6	37

8.5. *SHAKE91 (Item 5)*

SHAKE91 was used to evaluate the effects of local soil and rock conditions on the ground response at the current ground surface. The program determines the ground response by modeling the propagation of shear waves from bedrock to the ground surface. The generalized soil profile (i.e., SHAKE column) consists of the estimated thickness of each soil layer, shear wave velocities and estimated (total) unit weights. Shear wave velocities to a depth of 70 feet are shown in Table 1. Velocities below 70 feet were assumed to be 2,250 ft/sec (gravel) and 3,200 ft/sec (basalt) based on information reported in the literature (Wang and Leonard, 1996). Finish site grades will be raised above existing grades. Therefore, after site filling, we have assumed the base of the footing or mat foundation will lie a minimum of 1-foot below the existing grade. The data considered for use in our analysis is summarized in Table 6.

Table 6. Shake Column Data

Soil Type (Layer)	Thickness of Individual Sublayers (ft) (Number of Sublayers)	Unit Weight (lb/ft^3)	Shear Wave Velocity (ft/s)
Gravel	3, 4 and 5 ft. (3 Sublayers)	125	920
Gravel	5 ft. (2 Sublayers)	130	1,710
Gravel	5 ft. (3 Sublayers)	130	2,330
Sand	4 ft. (2 Sublayers)	125	1,710
Gravel	5 ft. (2 Sublayers)	135	3,350
Gravel	5 ft. (3 Sublayers)	135	2,180
Gravel	5, 10, 15 ft. (3 Sublayers)	135	2,250
Gravel	20 ft (10 Sublayers)	135	2,250
Columbia River Basalt	Half-Space		3,200

Notes: 1. The total depth of the SHAKE column is 300 feet.
2. Strain-dependent soil stiffness and damping curves for the respective soil types and rock were obtained from the SHAKE2000 database.

8.6. Analysis Results

8.6.1. *Random Crustal.* A bedrock A_{max} of ±0.30g with a T_p of ±0.15 seconds was estimated for the design random crustal earthquake based on the attenuation relationship from Joyner and Boore (1988). Therefore, the input motions used in SHAKE91 were scaled to 0.30g. The computed response spectra for the crustal earthquake is shown in Figure 1D, Appendix D. The corresponding soil surface response spectrum generally lies within the UBC response spectrum for an S_C profile.

8.6.2. *Subduction Zone (Interplate).* The relationships derived by Youngs et al. (1997) are based on recorded ground motion data from subduction zone earthquakes with source to site distances of between 10 to 500 km. An A_{max} of ±0.17g with a T_p of ±0.20 seconds was estimated for the design interplate earthquake. The computed response spectra for the Interplate earthquake is presented in Figure 2D, Appendix D. The selected input motions resulted in soil response within the 1997 UBC envelope for an S_C profile.

8.6.3. *Subduction Zone (Intraplate).* An A_{max} of ±0.22g with a T_p of ±0.20 seconds was estimated for the design intraplate earthquake (Youngs et al., 1997). The computed response spectra for the intraplate earthquake is shown in Figure 3D, Appendix D. The selected input motions resulted in soil response within the response spectra derived using the 1997 UBC criteria for an S_C profile.

8.7. Ground Shaking, Amplification and Response

Results of the dynamic soil response analyses suggest that a magnitude 6.5, randomly oriented crustal earthquake located at a depth or distance of 10 km from the site will generate moderate ground shaking. The spectral acceleration at the ground surface peaks just outside of the UBC spectrum at a period of ±0.15 seconds. However, the portion lying outside of the UBC envelope is negligible, and if a dynamic structural analysis is performed, we recommend using the UBC spectrum for design.

The amplification ratios between the bedrock and the ground surface were reviewed for all three design events at T=0 and the frequency domain to identify general trends. Average ratios at 0 period were less than 1.2 for the three events.

Deamplification was observed at frequencies of 15 Hz to 25 Hz. Amplification ratios of 1.3 to 1.5 were typical at lower frequencies. The maximum ratio was observed at the predominate site period (±0.7 seconds) where the ratio spiked to ±1.8. Although amplification was observed at lower frequencies, the ground response for all three events generally lies within the UBC envelop.

8.8. Liquefaction

The Turner site is underlain by dense to very dense deposits of sandy gravel and cobbles with saturated sand interbeds of varying densities (loose to dense) and thicknesses (±1 to ±5 feet). Where SPT data is available, the N-values suggest that most of the interbeds are medium dense to dense. However, a ±5-foot thick zone of loose sand was encountered at a depth of ±39 feet in BH-2. Although the sand is relatively deep and has a limited lateral extent, a qualitative and quantitative evaluation of liquefaction risk and post-liquefaction ground subsidence is provided.

Nearly 100 SPT values were completed in 10 borings and only one was low enough to suggest a liquefaction risk. However, evaluating risk cannot be adequately assessed using one SPT value. Rather, the collective site data needs to be considered and then the relative liquefaction risk evaluated. First, we noted that the horizontal extent of the sand layer in question is limited. The sand was not observed in BH-3 (±280 feet north of BH-2), in BH-1 (±260 feet south of BH-2) or in BH-10 to the west. A ±2-foot thick interbed was noted at a depth of ±41½ feet in BH-4 (±300 feet northeast of BH-2). The absence of the thicker sand strata in the surrounding borings suggests the zone is not continuous across the site and may be limited to a relatively small area in the vicinity of BH-2. Second, the sand layer is relatively thin and lies at a depth greater than 39 feet, and the non-liquefiable confinement layer (very dense sand, gravels and cobbles) overlying the sand is relatively thick. The thickness of the overlying layer required to prevent level-ground liquefaction related damage for sites subjected to a range of maximum accelerations was estimated using Figure 9.48 (Kramer, 1996). For a 5-foot thick sand layer confined by 39 feet of non-liquefiable soil, subjected to a maximum acceleration of 0.4g, the figure shows that liquefiable induced ground damage is highly unlikely.

We also note there are well documented cases supporting qualitative increases in liquefaction resistance with geologic age (Seed, 1979; Youd and Hoose, 1977; Youd and Perkins, 1978). Studies show that sediments deposited within the last several hundred years are generally much more susceptible to liquefaction than older Holocene sediments; Pleistocene sediments, reported to underlie the Turner site are even more resistant; and pre-Pleistocene sediments are essentially insusceptible to liquefaction. Although qualitative increases in liquefaction resistance have been well documented, it is noted in the NCEER workshop proceedings (edited by Youd and Idriss, 1996) that insufficient quantitative data have been assembled from which to incorporate a correction factor based on geologic age into the simplified procedure.

In a quantitative approach, the factor of safety against the triggering of liquefaction is the capacity of the soil to resist liquefaction (expressed in terms of cyclic resistance ratio (CRR)) divided by the seismic demand placed on a soil layer (expressed in terms of cyclic stress ratio (CSR)). The latest consensus for quantifying liquefaction susceptibility is summarized in the above-referenced NCEER workshop proceedings. Procedures presented in the proceedings were used, in part, during this study. The CSR was estimated using uniform average shear stress values. Two methods were used to estimate the CRR. The first method uses SPT data and the second is based on in-site shear wave velocities (Andrus and Stokoe, NCEER Proceedings, 1996).

A field SPT value of 10 (corresponding $(N_1)_{60}$ of 9) was recorded in the sand layer or lens. The method using SPT data for evaluating the loose sand indicates that the seismic demand exceeds the resistance which implies the risk for triggering liquefaction is high. Alternatively, the shear wave velocity recorded at depth was 1,710 ft/s and the factor of safety against liquefaction was greater than 3, suggesting the risk for triggering liquefaction is highly unlikely.

The analysis based on one SPT value suggests there is a risk of liquefaction triggering in the isolated, deep lens of loose sand. Evaluations completed on the remaining strata using SPT data revealed a factor-of-safety of at least 1.5 against liquefaction. In addition, the qualitative information and quantitative shear wave data presented above also suggest liquefaction is highly unlikely. Therefore, it is our professional opinion that a cyclically induced liquefaction hazard is negligible, and the risk of significant ground subsidence or bearing capacity failure due to liquefaction is also negligible. Seismically-induced lateral spread is also unlikely due to the flat terrain.

8.9. *Other Seismic Hazards*

There is a low potential for ground rupture due to the lack of known faulting beneath the site. Seismically induced landslide hazards do not exist due to the site's distance from the surrounding hills. The site cannot be affected by tsunami or seiche flooding due to its location.

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

The analysis and conclusions 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.

This report was prepared for the exclusive use of Calpine Corporation and their design consultants for the Turner Energy Center project in Turner, Oregon. This report is intended to present our conclusions regarding seismic hazards at the subject site.

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

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