AMENDMENT OF SOLICITATION/MODIFICATION OF CONTRACT

1. CONTRACT ID CODE: J	PAGE OF PAGES: 1 of 2		
2. AMENDMENT/MODIFICATION NO. 0004	3. EFFECTIVE DATE 30-Dec-2003	4. REQUISITION/PURCHASE REQ. NO. W68MD9-3311-5303	5. PROJECT NO.(If applicable)
6. ISSUED BY CODE W912DW USA ENGINEER DISTRICT, SEATTLE ATTN: CENWS-CT 4735 EAST MARGINAL WAY SOUTH SEATTLE WA 98134-2329		7. ADMINISTERED BY (If other than item 6) CODE **See Item 6**	

8. NAME AND ADDRESS OF CONTRACTOR (No., Street, County, State and Zip Code)

CODE | FACILITY CODE

[X] 9A. AMENDMENT OF SOLICITATION NO.
W912DW-04-R-0009

[X] 9B. DATED (SEE ITEM 11)
02-Dec-2003

[ ] 10A. MOD. OF CONTRACT/ORDER NO.

[ ] 10B. DATED (SEE ITEM 13)

11. THIS ITEM ONLY APPLIES TO AMENDMENTS OF SOLICITATIONS

[X] The above numbered solicitation is amended as set forth in Item 14. The hour and date specified for receipt of Offer [ ] is extended, [X] is not extended.

Offer must acknowledge receipt of this amendment prior to the hour and date specified in the solicitation or as amended by one of the following methods:
(a) By completing Items 8 and 15, and returning _______ copies of the amendment; (b) By acknowledging receipt of this amendment on each copy of the offer submitted; or (c) By separate letter or telegram which includes a reference to the solicitation and amendment numbers. FAILURE OF YOUR ACKNOWLEDGMENT TO BE RECEIVED AT THE PLACE DESIGNATED FOR THE RECEIPT OF OFFERS PRIOR TO THE HOUR AND DATE SPECIFIED MAY RESULT IN REJECTION OF YOUR OFFER. If by virtue of this amendment you desire to change an offer already submitted, such change may be made by telegram or letter, provided each telegram or letter makes reference to the solicitation and this amendment, and is received prior to the opening hour and date specified.

12. ACCOUNTING AND APPROPRIATION DATA (If required)

13. THIS ITEM APPLIES ONLY TO MODIFICATIONS OF CONTRACTS/ORDERS.
IT MODIFIES THE CONTRACT/ORDER NO. AS DESCRIBED IN ITEM 14.

[ ] A. THIS CHANGE ORDER IS ISSUED PURSUANT TO: (Specify authority) THE CHANGES SET FORTH IN ITEM 14 ARE MADE IN THE CONTRACT ORDER NO. IN ITEM 10A.

[ ] B. THE ABOVE NUMBERED CONTRACT/ORDER IS MODIFIED TO REFLECT THE ADMINISTRATIVE CHANGES (such as changes in paying office, appropriation date, etc.) SET FORTH IN ITEM 14, PURSUANT TO THE AUTHORITY OF FAR 43.103(B).

[ ] C. THIS SUPPLEMENTAL AGREEMENT IS ENTERED INTO PURSUANT TO AUTHORITY OF:

[ ] D. OTHER (Specify type of modification and authority)

E. IMPORTANT: Contractor [ ] is not, [ ] is required to sign this document and return _______ copies to the issuing office.

14. DESCRIPTION OF AMENDMENT/MODIFICATION (Organized by UCF section headings, including solicitation/contract subject matter where feasible.)

Solicitation No. W912DW-04-R-0009
Amendment No. R0004

Title: FIRE/CRASH RESCUE STATION
PORTLAND AIR NATIONAL GUARD BASE
PORTLAND, OREGON

SEE CONTINUATION PAGE

Except as provided herein, all terms and conditions of the document referenced in Item 9A or 10A, as heretofore changed, remains unchanged and in full force and effect.

15A. NAME AND TITLE OF SIGNER (Type or print)		16A. NAME AND TITLE OF CONTRACTING OFFICER (Type or print) TEL: EMAIL:	
15B. CONTRACTOR/OFFEROR (Signature of person authorized to sign)	15C. DATE SIGNED	16B. UNITED STATES OF AMERICA BY (Signature of Contracting Officer)	16C. DATE SIGNED 30-Dec-2003

SECTION SF 30 BLOCK 14 CONTINUATION PAGE

Amendment No. R0004 to Solicitation No. W912DW-04-R-0009, entitled “FIRE/CRASH RESCUE STATION , Portland Air National Guard Base, Portland, Oregon.”

A. This amendment provides for the following changes:

1.The enclosed “Geotechnical Investigation and Site-Specific Seismic Hazzards Study” dated March 20, 2003 is provided for informational purposes only.

2. The following information is provided for clarification of a discrepency noted Pertaining to the amounts of asphalt versus concrete between Drawing A001 and C002: **{Follow sheet C002 for pavement layout, remove notations “conc. drive way” on sheet A001.}**

B. PROPOSAL DUE DATE & TIME:
The Proposal due date and time remain unchanged: 05 January 2003, NLT 2:00 PM Local Time.

C. NOTICE TO OFFEROR’S:
Offerors must acknowledge receipt of this amendment by number and date on Standard Form 1442 block 19 submitted with proposal or by telegram.

D. All other terms and conditions of the solicitation remain unchanged.

Enclosures:

1. Geotechnical Investigation and Site-Specific Seismic Hazzards Study dated March 20, 2003

GRI
Geotechnical & Environmental Consultants
9725 SW Beaverton Hillsdale Hwy, Ste 140
Portland, Oregon 97005-3364
PHONE 503/641/3478 FAX 503/644/8034

March 20, 2003

3798 GEOTECHNICAL RPT (REV. 2)
(ISSUED 5/30/2003)

Helix Architecture, PS
6021 12th Street East, Suite 201
Tacoma, WA 98424

Attention: Erik Prestegaard, PE

SUBJECT: Geotechnical Investigation and Site-Specific Seismic Hazards Study
Fire Station at Portland Air National Guard (PANG)
Portland, Oregon

At your request, GRI has completed a geotechnical investigation and site-specific seismic hazards study for the proposed fire station at the Portland Air National Guard base in Portland, Oregon. The general location of the site is shown on the Vicinity Map, Figure 1. The investigation was conducted to evaluate subsurface conditions at the site and develop recommendations for use in the design of the proposed building. Our investigation consisted of subsurface explorations, laboratory testing, and engineering studies and analyses. This report describes the work accomplished and provides our conclusions and recommendations for the design and construction of the project.

PROJECT DESCRIPTION

Based on our conversations with Berger/ABAM Engineers, Inc., the project civil engineers, we understand the fire station will be a two-story structure located immediately south of existing Building 270. The location and configuration of the proposed building are shown on the Site Plan, Figure 2. The new building will have a footprint of about 12,700 ft^2. Maximum column loads for the structure will be about 170 kips. The new fire station is considered an essential facility according to the Oregon Structural Specialty Code. The proposed building will not have a basement or other significant below-grade structure, and the maximum depth of new underground utilities is anticipated to be less than 10 ft. In addition, only minor site grading will likely be necessary.

SITE DESCRIPTION

General

The site is located south of the existing Building 270 at the Air National Guard Base at Portland International Airport. The Site Plan, Figure 2, shows the location and configuration of the proposed fire station building. Existing site grades range from about elevation 16 to 22 ft, NGVD. The eastern portion of the site is undeveloped and covered with grass. The western portion of the site includes an existing concrete slab; vegetation in this portion of the site consists of grass.

Geology

The site is located on the former floodplain of the Columbia River. In this area, the ground surface is generally mantled with a variable thickness of fill. The fill is underlain by naturally occurring alluvial silt and sand that are underlain by gravel at depth. The depth to gravel generally increases from south to north across the base.

SUBSURFACE CONDITIONS

General

Subsurface conditions at the site were investigated on January 21, 2003, with one boring, designated B-1, and three cone penetration test (CPT) probes, designated P-1, P-2, and P-3. The locations of the explorations are shown on Figure 2. The field exploration and laboratory testing programs completed for this investigation are discussed in detail in Appendix A. Logs of the boring and CPT probes are provided in Appendix A. The terms used to describe the soils disclosed in the boring are defined in Table 1A.

Soils

The subsurface explorations for this investigation were advanced to a maximum depth of 110 ft. The explorations indicate the majority of the site is mantled with a layer of silt fill that is about 6 ft thick. The fill is underlain by alluvium that consists primarily of silt with varying percentages fine-grained sand and interbedded layers of silty sand and sand to a depth of about 30 ft. The soils below a depth of about 30 ft consist primarily of fine-grained sand. For the purpose of discussion, the soils disclosed during the subsurface investigation have been grouped into the following categories, based on their physical characteristics and engineering properties.

1. **FILL**
2. **SILT with layers of SAND**
3. **SAND**

1. FILL. The boring and CPT probes encountered a surface layer of silt fill that extends to a depth of about 6 ft. The silt is brown mottled rust and is clean or contains a trace of fine-grained sand. N-values of 2 to 4 blows/ft and CPT sleeve friction resistances ranging from about 0.05 to 0.65 tsf indicate the relative consistency of the silt fill ranges from very soft to medium stiff.

2. SILT with layers of SAND. The fill is underlain by gray alluvial silt with interbedded layers of sand that are up to several feet thick. The silt contains varying percentages of fine-grained sand and organic material. N-values ranging from 4 to 7 blows/ft and cone penetration resistances ranging from about 8 to 54 tsf indicate the relative consistency of the silt is medium stiff, and the relative density of the interbedded sand is loose to medium dense. The natural moisture content of the alluvial soils ranges from about 34 to 47%.

3. SAND. Alluvial sand was encountered at a depth of about 30 ft below the ground surface. The sand is typically gray and fine grained. N-values ranging from 0 to 45 blows/ft and cone penetration resistances ranging from about 10 to 265 tsf indicate the relative density of the sand ranges from loose to very dense and increases with depth. The sand becomes medium dense at about 35 ft; dense at 50 ft, and very dense at 100 ft. The natural moisture content of the sand ranges from about 26 to 41%. Boring B-1 was terminated in sand at a depth of 101.5 ft below the ground surface. CPT probes P-1, P-2 and P-3 were terminated in sand at depths of 100 to 110 ft.

Groundwater

Drainage canals and sloughs in this part of the floodplain are operated by the Multnomah County Drainage District to control the water levels in the floodplain. Pumping maintains the water level in the sloughs and drainages at about elevation +8 ft in the project area. Seasonal variations can occur during extreme highs and lows of the Columbia River and during the wet, winter months. Groundwater levels at locations away from the sloughs are commonly higher than the water level in the sloughs and may approach the ground surface during prolonged wet weather. The borings for this investigation were drilled using mud-rotary methods, which does not permit the measurement of groundwater levels during drilling. However, based on our previous work in this area, we anticipate the groundwater level ranges from about elevation +7 to +12 ft during the dry season.

CONCLUSIONS AND RECOMMENDATIONS

General

The subsurface explorations indicate the site is mantled with medium stiff silt fill to a depth of about 6 ft. Below the fill is a thick deposit of compressible silt that contains interbedded layers of sand and sandy silt, which is underlain by denser sand deposits. Based on the results of our investigation, which included a site-specific seismic hazard study, there is the potential for partial or complete liquefaction of the soils in the upper 40 to 55 ft of the site during a strong seismic event. This liquefaction will likely result in significant amounts of total and differential settlement across the site. Since the new fire station is considered an essential facility, which we understand is required to be functional after a strong seismic event. In our opinion, it will be necessary to construct the building on a deep pile foundation system established in the dense sand to limit risk of damage to the structure due to the relatively large amounts of liquefaction-induced settlement. In this regard, if some deformation of the first-floor slab-on-grade is acceptable, it will not be necessary to provide a pile-founded structural floor slab. However, if this option is selected, we recommend that you consider installing a thickened floor slab, similar to an impact panel for bridge approaches, on both sides of the fire truck doorways. In our opinion, this would limit the risk of liquefaction-induced settlement preventing the fire trucks from exiting the building. The following sections of this report provide our conclusions and recommendations concerning design and construction of foundations for the proposed structure and other geotechnical-related items.

Seismic Considerations

A site-specific seismic hazards study has been completed for this project. A detailed description of the seismic study is provided in Appendix B. Based on our studies, during a strong seismic event there is a moderate to high potential for partial or complete liquefaction within the fill and alluvial sands and silts that exist in the upper 40 to 50 ft at the site. The liquefaction potential for the site was evaluated based on the soil profile disclosed in boring B-1 and results of cone penetration test probes P-1 through P-3. Depending on actual river and groundwater levels at the time of the earthquake, we estimate liquefaction-induced settlements at the site could be in the range of a few feet, assuming groundwater at a depth of about 3 ft and earthquake magnitudes, focal distances, and accelerations consistent with design ground motions. Additionally, an earthquake with a larger magnitude producing the same acceleration at the site will result in larger settlements. It should be assumed that significant differential settlement could occur across the footprint of the building. We also anticipate that a significant percentage of the settlement will occur after the shaking stops.

In our opinion, liquefaction induced settlement likely represents the most significant seismic design consideration for this project. However, to assist with the structural evaluation of the new building, we have estimated the ground response for the three seismic design events in this region. For the purpose of this study, we have assumed that a damping ratio of 5% is appropriate to characterize the planned structure. We have chosen three distinctly different types of design earthquakes to represent the seismic events that might affect the project. The basic parameters of the design earthquakes are tabulated below.

Earthquake Type	Magnitude	Focal Distance, km	Peak Horizontal Bedrock Acceleration, g
Subduction Zone	8.5 (Mw)	70	0.21
Subcrustal	7.0 (Mw)	50	0.24
Local Crustal	6.5 (ML)	9	0.28

Based on the results of our studies, as described in Appendix B, amplification of the seismic energy will occur as the seismic waves from earthquakes are propagated upward to the ground surface from the underlying bedrock. The results of our site-specific seismic study indicate that for the conditions at this site, the controlling peak horizontal ground acceleration is generated by the local crustal model, which resulted in a mean peak horizontal ground acceleration of 0.21 g. To assist the structural engineer in evaluating the response of the pile founded structure, we have provided site-specific acceleration response spectra for the ground surface and the top of the dense sand layer at a depth of 40 ft, immediately below the estimated zone of liquefaction. The results of these analyses indicate that for periods less than about 1.5 seconds there is significant amplification of the ground surface above the dense sand layer. Of course, it must be understood and acknowledged that the analytical methods of estimating seismic ground response do not include the effects of liquefied soil conditions. Therefore, the ground surface acceleration response spectra may not accurately model the actual ground shaking at this site.

Based on the criteria defined in the US Army Corps of Engineers TI 809-04, "Seismic Design of Buildings", and the potential for liquefaction at the site, the site classification is Class F, which requires site-specific evaluation. However, in our opinion, based on the results of our site-specific seismic study, Class E may be used for the seismic design of the structure.

Based on the site topography, it is our opinion that the risk for earthquake-induced slope instability is low. Based on the elevation and location of the site, the risk of damage by tsunamis and/or seiches at the site is absent. Based on our review of geologic maps and available subsurface information, no faults are mapped on or near the site, and it is our opinion that the potential for fault rupture at the site is low.

Foundation Support

In our opinion, foundation support for the fire station should be provided by a deep foundation system that extends into the lower sand layer that underlies the site. Based on previous experience at nearby sites and our understanding of the project, we anticipate that either steel pipe piles or augercast piles will be suitable for support of the fire station structure.

Allowable capacities for piles will depend on pile diameter and/or size and penetration into the lower sand layer. The following table summarizes our recommended allowable compression (downward) and tension (uplift) criteria for steel pipe piles and augercast piles.

Pile Type	Estimated Minimum Penetration into Lower Sand, ft	Allowable Capacity (compression), tons	Allowable Capacity (tension), tons
PP 12.75 x 0.375 (open-end)	30	60	30
16-in.-diameter augercast	30	70	35

The above design capacities assume the piles will be installed to a tip elevation of about -62 ft. The estimated penetrations into the lower sand are based on subsurface explorations and liquefaction analysis of the site. In this regard, the allowable capacities refer to real loads, i.e., the total of dead load plus frequently or permanently applied live loads, including transient seismic loads, and include a reduction for downdrag. The allowable pile capacities are based on soil support considerations and include an estimated factor of safety of at least 2 for static loading conditions and a factor of safety of 1.5 for all loads, including seismic loading and downdrag. The structural strength of the pile may limit the allowable capacities to lower values.

We acknowledge that other pile types and capacities may be appropriate for this site depending on the specific pile design load requirements. The above pile capacities are intended to provide you with pile types and capacities that have been commonly used in these types of soils.

We anticipate that the settlement of steel pipe or augercast piles installed in accordance with the criteria presented herein will be relatively small and limited to the elastic shortening of the pile.

Lateral Load Analysis

Lateral structural loads can be resisted by piles in bending and the passive resistance of the soil adjacent to the pile cap. Lateral loading of pipe piles was analyzed with the aid of the computer software program L-Pile, version 4.0, by Ensoft, Inc. of Austin, Texas. The program computes deflection, shear, bending moment, and soil response with respect to depth. The soil behavior was modeled with p-y curves internally generated by the software following published recommendations for various types of soils.

The steel pipe piles used in the analysis have a diameter of 12.75 in. and a wall thickness of 0.375 in. For the seismic case, the group effects were ignored. This is based on the assumption that the cyclic loading during a seismic event will remold the soil around the pile causing the weakened soils to become less effective in transferring the induced stresses to the neighboring piles. This effect would be most significant for relatively small pile groups with a minimum spacing of 3D driven in soft and loose soils, typical of the conditions present at the site. The piles were analyzed for the fixed- and free-head conditions. The lateral loads were applied at the head of the pile.

The soil profile was developed using the results of the subsurface explorations. Soil properties were determined based on the results of our field investigation, laboratory testing, review of existing information for the area, and our experience with other projects in the vicinity of the site, particularly the Portland International Airport.

For the site, groundwater was assumed to occur at the ground surface. The seismic case was analyzed, and information from our liquefaction study was used to identify layers that are likely to liquefy. These layers were assigned residual shear strength values based on published data. The typical residual shear strength values ranged from 100 to 200 psf. The lower sand layer was assigned an effective shear strength, ϕ', of 35°. The cyclic loading option in L-Pile was used. The results of lateral load analyses for the steel piles are tabulated below.

SUMMARY OF LATERAL LOAD ANALYSIS FOR DRIVEN PILES

Pile Type	Lateral Single Pile Capacity, kips Fixed Head 1/2 -in. deflection	Lateral Single Pile Capacity, kips Free Head 1/2 -in. deflection
PP12.75 x 0.375	6.0	2.5
16-in.-diameter augercast	6.6	3.0

Additional resistance to lateral forces can be provided by passive earth pressure against the pile cap. The magnitude of the passive earth pressure will depend, in part, on the anticipated limiting deflection. In our opinion, passive resistance can be evaluated on the basis of an equivalent fluid having a unit weight of 250 lb/ft^3 for deformations of 0.5 in. This value assumes the excavations for the pile caps will be backfilled with compacted granular fill.

Installation Criteria for Driven or Vibratory Piles

Steel pile piles should be installed with a center-to-center spacing of at least three pile diameters. The piles may be driven with an air, steam, or diesel hammer exerting at least 32,000 ft-lb of energy per blow, or as necessary to achieve the required pile tip elevation. A vibratory pile hammer/extractor may be used to install the open-end pipe piles. The vibratory hammer must be capable of installing the piles to the required tip elevations.

In this regard, we recommend that all pile-driving operations be observed on a full-time basis, and a continuous record of pile installation resistance versus depth of penetration should be maintained for each pile. All pile driving records should be reviewed by the geotechnical engineer as the work progresses.

Installation Criteria for Concrete Augercast Piles

Augercast piles are constructed by rotating a hollow-stem auger into the ground to the desired depth. As the auger is slowly withdrawn, grout is pumped through the hollow auger stem and out the bottom of the auger. It is essential that the tip of the auger be maintained at least 4 to 6 ft below the surface of the grout as the auger is withdrawn. Augercast piles should only be installed by an experienced contractor with a proven record of pile installation in similar conditions. Concrete augercast pile installation should be observed on a full-time basis by a qualified geotechnical engineer. Augercast piles typically include a rebar cage in the upper portion of the pile and a full-length reinforcing bar to transmit uplift (tension forces).

To minimize disturbance of recently grouted piles that have not yet established their initial set, we recommend that a 24-hour waiting period be specified for installing piles spaced closer than five pile diameters. In addition, particular care should be taken during installation of the augercast piles to prevent the inclusion of any spoils or cuttings within the grouted column. This requires withdrawing the auger and grouting the column in a continuous operation, while maintaining a positive head of grout within the stem of the auger at all times. Consideration should be given to carefully centering rebar cages and using spacers along the length of the cage to maintain proper cover for the steel around the perimeter of the pile. Difficulties encountered while grouting the column or placing the rebar may require redrilling the pile to full depth before the grout sets. Provisions should be made to top off the grout column if the grout settles before it sets and to protect the heads of freshly completed piles.

Site Preparation and Grading

We recommend that all organic material, existing AC, and concrete be removed from within the limits of the proposed building and areas outside the new building that will be paved. To minimize disturbance of the near-surface fine-grained subgrade soils, we recommend performing site stripping and making all excavations with backhoes equipped with smooth cutting edges. In general, we anticipate that stripping to a depth of about 6 in. will be required in areas of existing landscaping. However, greater or lesser amounts of stripping may be required locally. In our opinion, spoil materials should be removed from the site, or in the case of topsoil, stockpiled on site for use in landscaped areas. Upon completion of site stripping and excavation to subgrade level, the resulting subgrade in areas to receive structural fill should be observed by a qualified geotechnical engineer. Any soft areas or areas of unsuitable material should be overexcavated to firm undisturbed soil and backfilled with structural fill. It should be anticipated that some overexcavation will be necessary due to the presence of fill as disclosed by some of the borings.

The moisture content of the silty subgrade materials will likely be significantly above optimum. When the moisture content of these soils is in excess of 4 to 5% of the optimum moisture content, they typically become weak and unstable when disturbed and remolded by construction traffic. For this reason, we recommend that, if possible, all site preparation and earthwork be accomplished during the dry summer months, typically extending from mid-May to mid-October of any given year. We also recommend that in areas to receive structural fill, such as the new roadway and parking areas, construction equipment not traffic the fine-grained subgrade soils. This will require the placement of granular fill for a working pad to protect the subgrade. In addition, if construction is performed during the wet winter and spring months, it will likely be necessary to place a granular work pad in the building area. In our opinion, a 12-in.-thick granular work pad should be sufficient to prevent disturbance of the subgrade by lighter construction equipment. A granular work pad on the order of 18 to 24 in. thick is typically required to protect fine-grained subgrade soils from disturbance by repetitive heavy construction loads. If the subgrade is disturbed during construction in areas that will be paved outside the pile founded building footprint, the soft disturbed soils should be overexcavated to firm soil and backfilled with granular structural fill.

Structural Fill

In our opinion, imported, relatively clean, granular material approved by the geotechnical engineer may be used to construct structural fills. Imported granular materials used to construct structural fills or work pads should consist of material with a maximum size of up to 6 in. and with not more than about 5% fines passing the No. 200 sieve (washed analysis). The first lift of granular fill material placed over the silt subgrade should be in the range of 12 to 18 in. thick (loose). Subsequent lifts, if needed, should be placed 12 in. thick (loose). All lifts should be compacted with a medium-weight (48-in.-diameter drum), smooth, steel-wheeled, vibratory roller until well keyed. Generally, a minimum of four passes with the roller are required to achieve compaction.

All backfill placed in utility trench excavations within the limits of the buildings and paved areas should consist of sand, sand and gravel, or crushed rock with a maximum size of up to 1½ in., and with not more that 5% passing the No. 200 sieve (washed analysis). In our opinion, the granular backfill should be placed in 9-in.-thick lifts (loose) and compacted using vibratory plate compactors or tamping units to at least 95% of the maximum dry density as determined by ASTM D 698. Flooding or jetting the backfilled trenches with water to achieve the recommended compaction should not be permitted.

Floor Support

If you elect not to use a structural floor slab, we recommend the subgrade be prepared as recommended in the Structural Fill section of this report. We anticipate the finish floor elevation will be established near existing site grades. Therefore, we recommend placing a minimum 6-in.-thick granular base course beneath the concrete floor slab. Base course material should be compacted to at least 95% of the maximum density as determined by ASTM D 1557. In addition, it may be appropriate to install a suitable vapor-retarding membrane beneath slab-on-grade floors in the office building areas where damp-proofing may be needed. The details of the vapor-retarding membrane are shown on Figure 3, which shows a minimum 6-in.-thick granular base course beneath the concrete floor slab which serves as a capillary break. The base course material should consist of crushed rock with a maximum size up to about 1 in. and less than about 2% passing the No. 200 sieve (washed analysis). Assuming the floor slab subgrade and base course are suitably prepared, we recommend using a modulus of subgrade reaction of 225 pci for the design of concrete slabs that are subjected to heavy floor loads.

Pavement Design

We anticipate that pavement areas around the proposed building will be subjected to automobile and heavy emergency vehicle traffic. It is our understanding the heavy emergency vehicle traffic is estimated to be about four trips per day. All areas to be paved should be prepared as recommended in previous sections of this report. Prior to placing base course materials, all paved areas should be proof rolled with a fully loaded 10 yd^3 dump truck. Any soft areas that are detected by the proof rolling should be overexcavated to firm ground and backfilled with compacted structural fill.

	Minimum Flexible Pavement Thickness, in.	Minimum Rigid Pavement Thickness, in.	Minimum Crushed Rock Base Thickness, in.
Areas Subject to Fire Truck Traffic	N/A	8.5	6
Areas Subject Primarily to Automobile Traffic and Parking	3	N/A	8

The recommended thicknesses assume that all pavement sections will be constructed during the dry season. If wet-weather pavement construction is considered, it will likely be necessary to increase the thickness of crushed rock base course for both pavement sections to support construction equipment. We would also welcome the opportunity to review the above-recommended sections when traffic estimates become available.

In those areas where the pavement will be placed over a granular work pad, it will probably only be necessary to remove the contaminated surface material, i.e., the upper few inches, and replace this with the crushed rock base course prior to paving. However, prior to any grading or paving, the granular work pad should be proof rolled with a fully loaded dump truck. Any soft and/or wet areas should be overexcavated and backfilled with compacted structural fill.

Properly installed drainage is an essential aspect of pavement design. All paved areas should be provided with positive drainage to remove surface water and water within the base course. This will be particularly important in cut sections or at low points within the paved areas, such as at catch basins. Effective methods to prevent saturation of the base course materials include roadside drainage ditches in communication with and below the base course, providing weep holes in the sidewalls of catch basins, subdrains in conjunction with utility excavations, and separate trench drain systems. To provide quality materials and construction practices, we recommend that the pavement work conform to the "Standard Specifications for Highway Construction" used by the Oregon Department of Transportation.

Design Review and Construction Services

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that all construction operations dealing with earthwork and foundations should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in this report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

LIMITATIONS

This report has been prepared to aid the engineer in the design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the proposed structure. In the event that any changes in the design and location of the structure as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the boring and CPT probes made at the locations indicated on Figure 2 and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between subsurface explorations. If, during construction, subsurface conditions different from those encountered in the explorations are observed or encountered, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

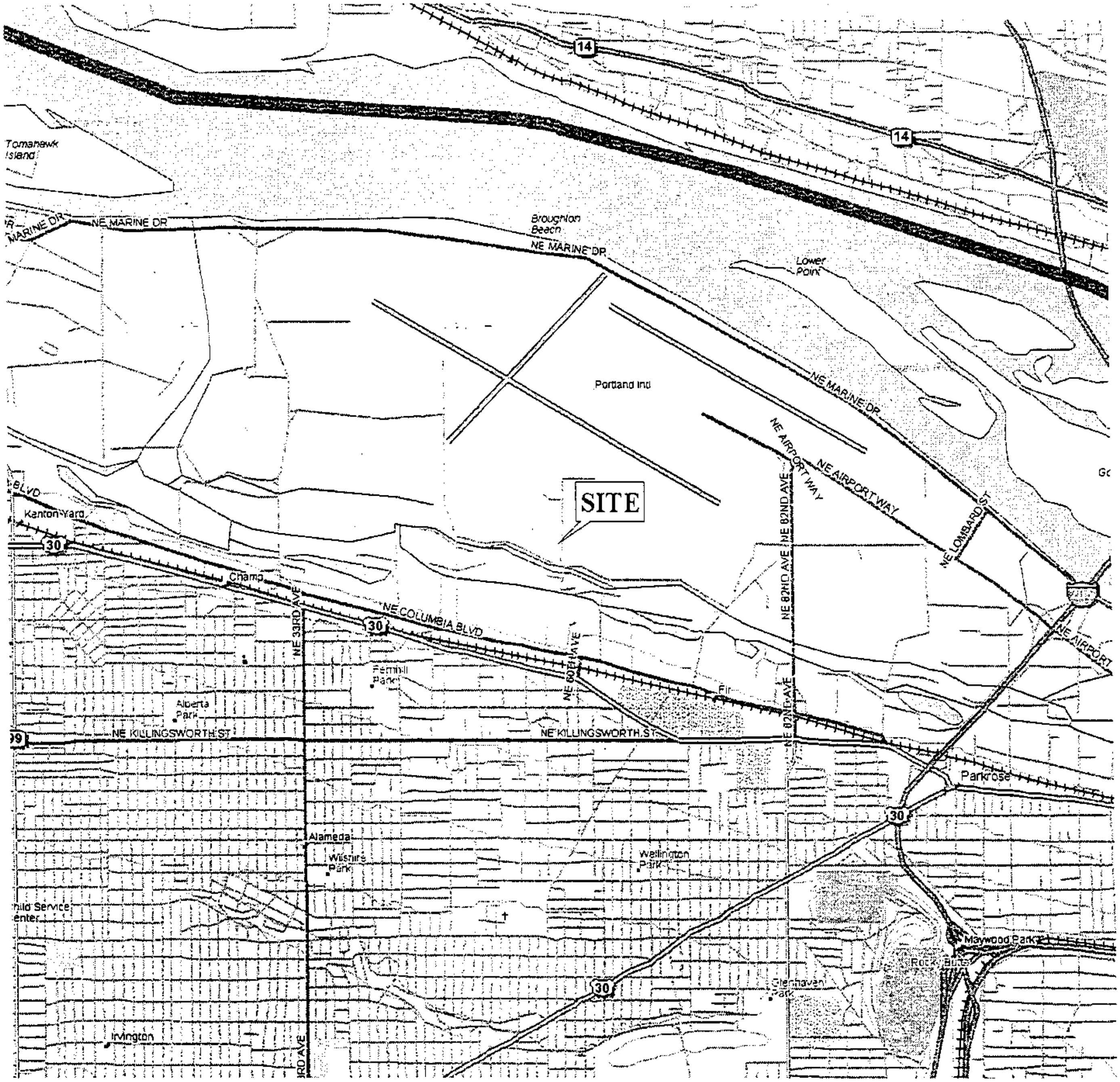
Submitted for GRI,

REGISTERED PROFESSIONAL ENGINEER
8045
OREGON
JULY 22, 1974
H. STANLEY KELSAY

H. Stanley Kelsay
6/05

H. Stanley Kelsay, PE
Principal

Tova R. Peltz
Staff Engineer



DELORME 3-D TOPOQUADS, OREGON WEST
MOUNT TABOR, OREG. (2cd) 1999

North

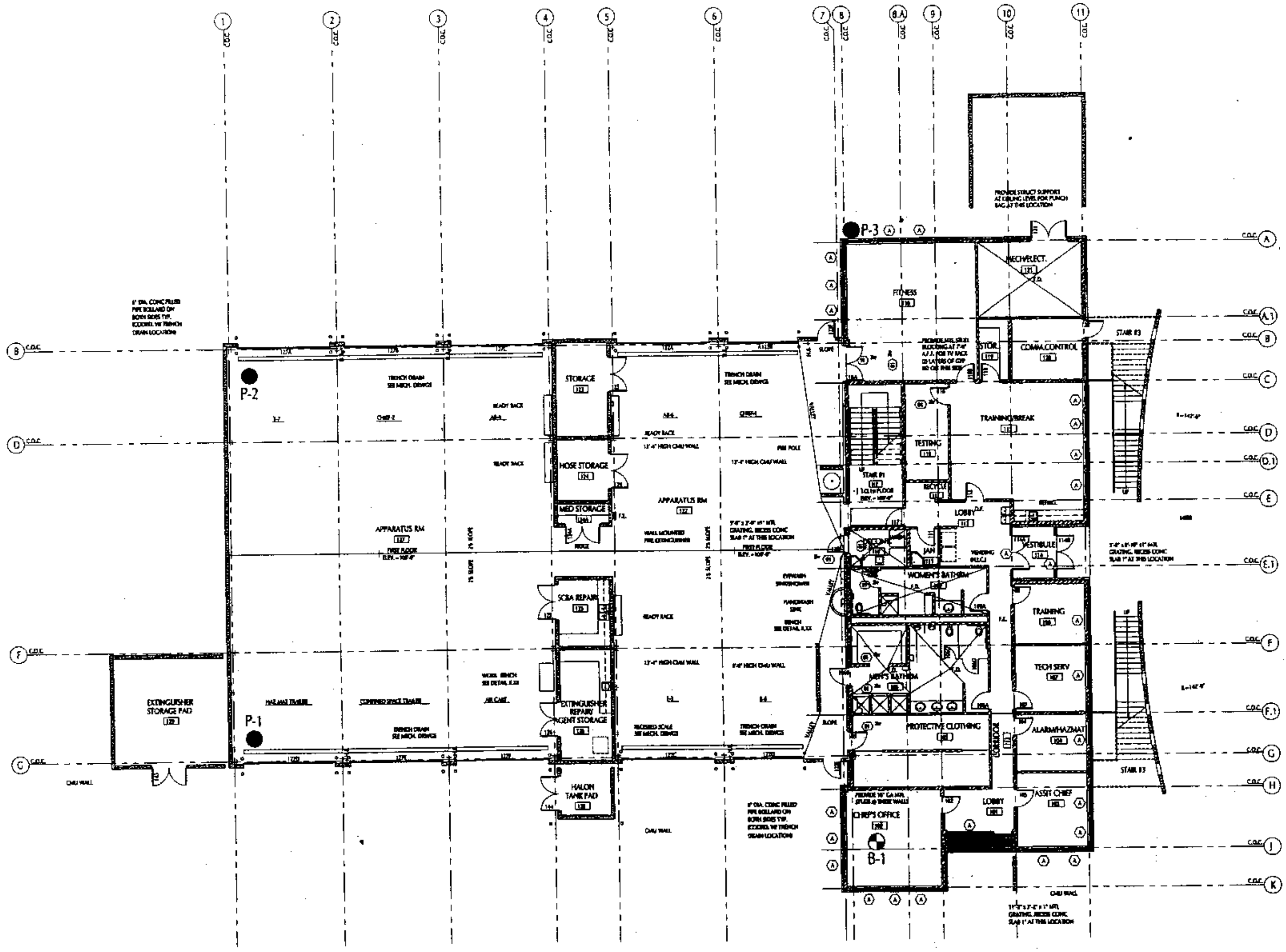
0 1 2 MILES

GRI HELIX ARCHITECTURE PS
PORTLAND AIR NATIONAL GUARD FIRESTATION

VICINITY MAP

MAR. 2003 | JOB NO. 3798 | FIG. 1

R0004



● CONE PENETRATION TEST MADE BY GRI (JANUARY 21, 2003)

BORING MADE BY GRI (JANUARY 21, 2003)

SITE PLAN FROM FILE BY BERGER ABAM (UNDATED)

North

0 20 40 FT

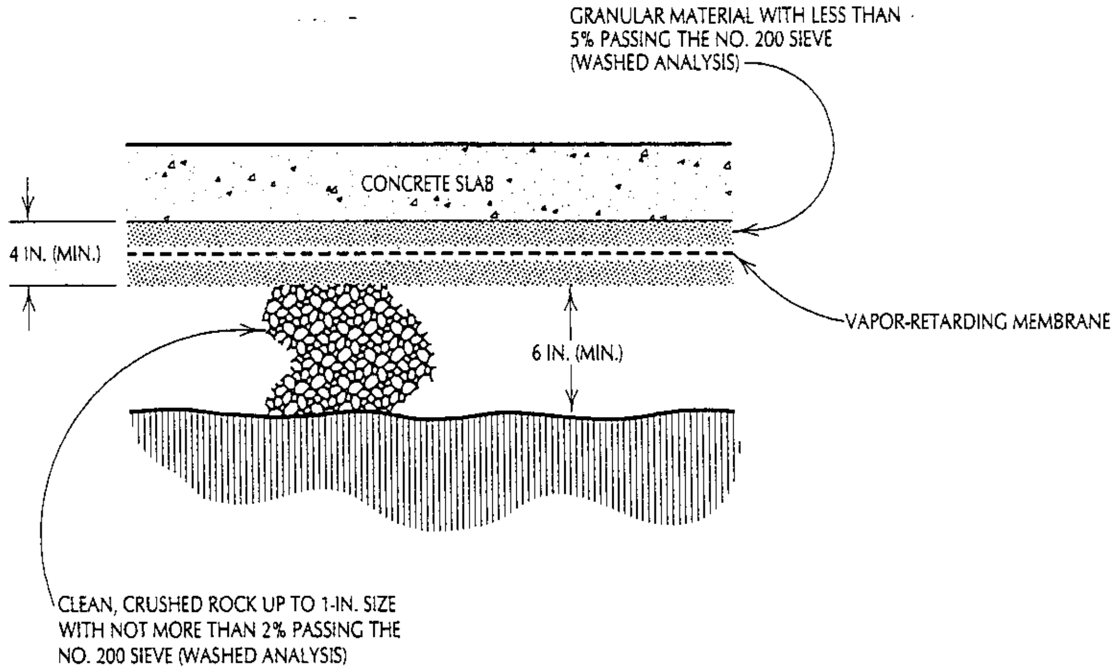
GRI

BERGER ABAM
PANG FIRESTATION SEISMIC STUDY

SITE PLAN

MAR. 2003 JOB NO. 3798 FIG. 2

R0004



NOT TO SCALE

GRI

HELIX ARCHITECTURE PS
PORTLAND AIR NATIONAL GUARD FIRESTATION

UNDERSLAB DRAINAGE DETAIL

MAR. 2003 | JOB NO. 3798 | FIG. 3

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

General

Subsurface materials and conditions at the site were investigated January 21, 2003, with one boring, designated B-1, and three cone penetration test (CPT) probes, designated P-1, P-2, and P-3. The locations of the explorations are shown on Figure 2. The boring was drilled to a depth of 101.5 ft with mud-rotary techniques using a truck-mounted drill rig provided and operated by Subsurface Explorations of Banks, Oregon. The CPT probes extended to depths of 100 to 110 ft. An experienced geotechnical engineer provided by our firm observed the drilling and sampling and maintained a detailed log of the materials disclosed during the course of the work.

Boring

Disturbed and undisturbed samples were obtained from the boring at 2.5-ft intervals of depth in the upper 15 ft and at 5-ft intervals below this depth. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the standard penetration resistance, or N-value. The N-values provide a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as silt. The soil samples obtained in the split-spoon sampler were carefully examined in the field and representative portions were saved in airtight jars for further examination and physical testing in our laboratory.

A log of the boring is provided on Figure 1A. The log provides a descriptive summary of the various types of materials encountered in the boring and notes the depth at which the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, N-values are shown graphically, along with the natural moisture contents. The terms used to describe the soils encountered in the boring are defined in Table 1A.

Cone Penetration Test Probes

The cone penetration test consists of forcing a hardened steel cone vertically into the soil at a constant rate of penetration. The thrust required to cause penetration at a constant rate can be related to the bearing capacity of the soil immediately surrounding the point of the penetrometer cone. This value is known as the cone penetration resistance. After making the cone thrust measurement, a measurement is obtained of the magnitude of thrust required to force a special friction sleeve, attached above the cone, through the soil. The thrust required to move the friction sleeve can be related to the undrained shear strength of fine-grained soils. The dimensionless ratio of sleeve friction to point bearing capacity provides an indication of the type of soil penetrated. The cone penetration resistance and the sleeve friction are determined at about 8-in. intervals in the probe hole and can be used to evaluate the relative density of cohesionless soils and the relative consistency of cohesive soils, respectively.

Logs of the cone penetration test probes are provided on Figures 2A through 4A. Each log presents a descriptive summary of the various types of materials encountered in the explorations and notes the depth at which the materials and/or characteristics of the materials change.

LABORATORY TESTING

General

All samples obtained from the boring were returned to our laboratory where the physical characteristics of the samples were noted and the field classifications were modified where necessary. The laboratory testing program included determinations of natural moisture content and grain size. The following paragraphs describe the testing program in more detail.

Natural Moisture Content

The natural moisture content of the soil samples was determined in substantial conformance with ASTM D 2216. The results are summarized on Figure 1A.

Grain Size Analyses

Grain size analyses of selected samples were performed in general conformance with ASTM D 421 and 422 to determine the percentage of material passing the No. 200 sieve. The test results are tabulated below.

SUMMARY OF GRAIN SIZE ANALYSES

Boring	Sample	Depth, ft	Percent Passing No. 200 Sieve	Soil Description
B-1	S-3	10	30	SAND; fine grained , some silt
	S-4	12.5	67	Sandy SILT; fine-grained sand
	S-5	15	24	SAND; some silt
	S-7	25	88	SILT [illegible]ne fine-grained sand
	S-8	30	70	SILT; some fine-grained sand to sandy
	S-10	40	12	SAND; fine grained, some silt
	S-13	55	9	SAND; fine grained, trace silt

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane Undrained Shear Strength, tsf
very soft	2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Sandy silt materials which exhibit general properties of granular soils are given relative density description.

Grain-Size Classification

Boulders
12 - 36 in.

Cobbles
3 - 12 in.

Gravel
1/4 - 3/4 in. (fine)
3/4 - 3 in. (coarse)

Sand
No. 200 - No. 40 sieve (fine)
No. 40 - No. 10 sieve (medium)
No. 10 - No. 4 sieve (coarse)

Silt/Clay - pass No. 200 sieve

Modifier for Subclassification

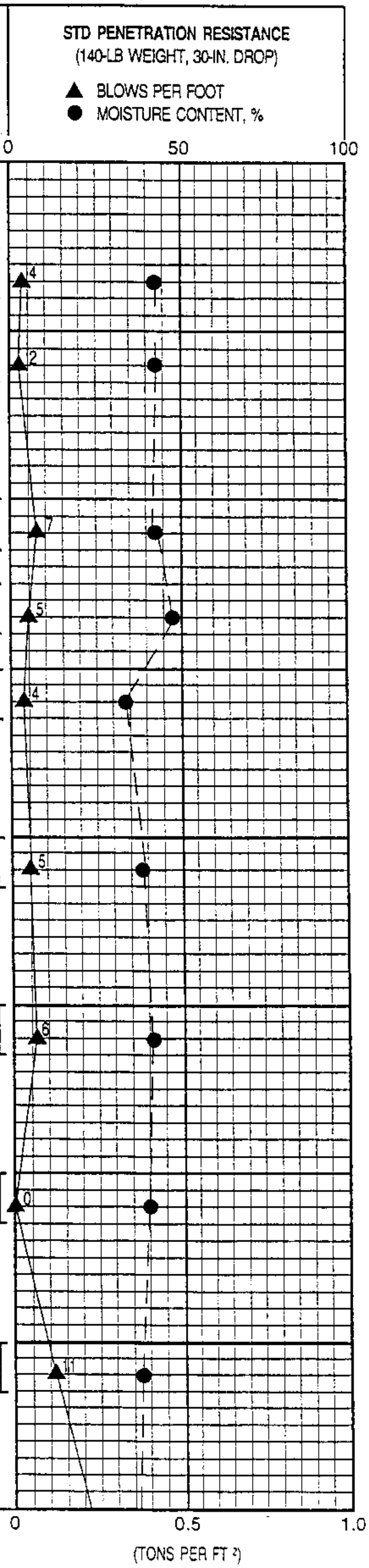
Adjective	Percentage of Other Material In Total Sample
clean	0 - 2
trace	2 - 10
some	10 - 30
sandy, silty, clayey, etc.	30 - 50

GRI

BORING B-1

SURFACE ELEVATION NOT AVAILABLE

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLES	BLOWS PER FOOT
0–6.0	FILL: Medium stiff, brown mottled rust SILT; clean to trace fine-grained sand, scattered roots, 3-in.-thick heavily rooted zone at the ground surface		S-1	4
5	--------very soft below 5 ft	6.0	S-2	2
6.0–12.5	Very loose to loose, brown to gray SAND; fine grained, trace silt		S-3	7
12.5–15.0	Medium stiff, gray, sandy SILT; fine-grained sand	12.5	S-4	5
15.0–26.0	Loose, gray SAND; fine grained, some silt to silty	15.0	S-5	4
			S-6	5
26.0–30.0	Medium stiff, gray SILT; occasional interbeds of fine-grained sand	26.0	S-7	6
30.0–40	Very loose, gray SAND; fine grained, some silt, occasional interbeds of silt	30.0	S-8	0
35	--------medium dense below 35 ft		S-9	11



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

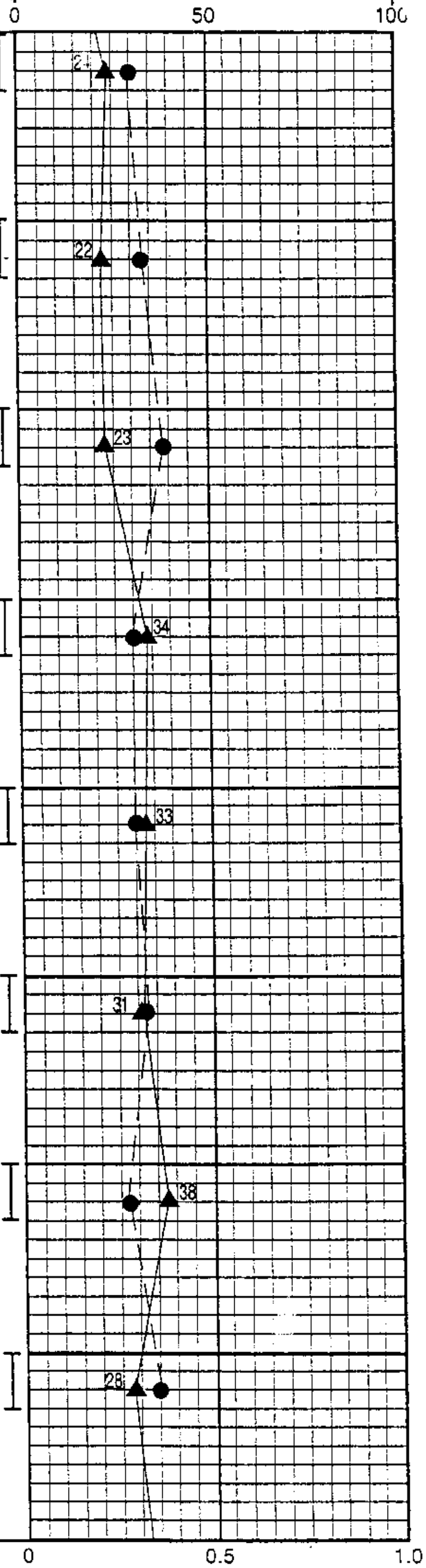
BORING B-1

MAR. 2003 JOB. NO. 3798 FIG. 1A

R0004

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION NOT AVAILABLE			
40		Medium dense, gray SAND; fine grained, some silt			S-10
45					S-11
50		--------some silt to silty at 50 ft			S-12
		--------dense below 51 ft; 6-in.-thick layer of silt at 51 ft			
55		Dense, gray SAND; fine to medium grained, trace silt	53.0		S-13
60					S-14
65					S-15
70					S-16
75		--------medium dense at 75 ft			S-17
80					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▌ NX CORE RUN
|— SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-1 (cont.)

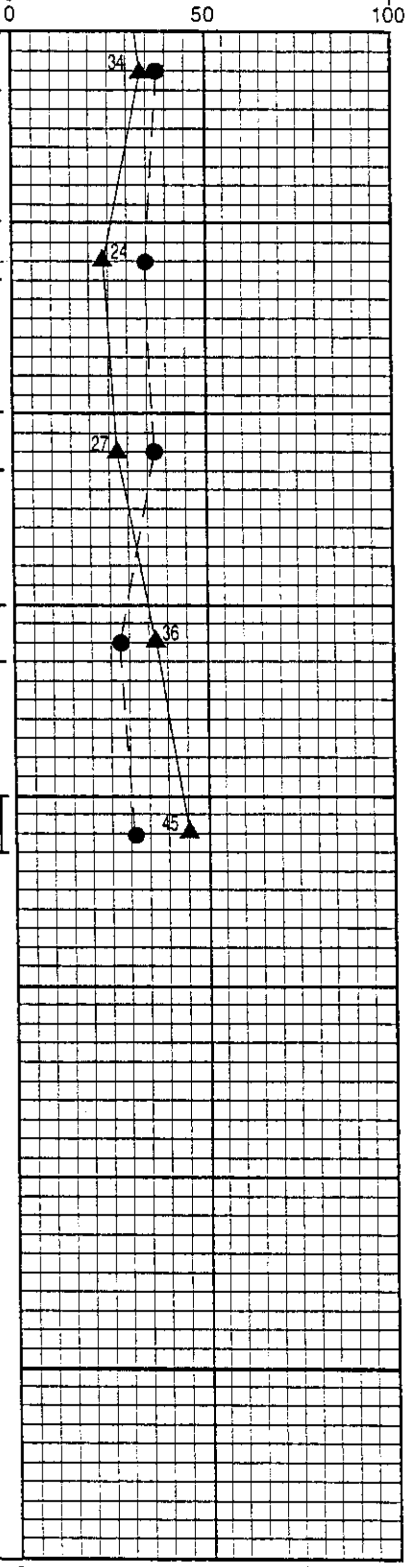
MAR. 2003 JOB. NO. 3798 FIG. 1A

R0004

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION NOT AVAILABLE	DEPTH, FT	GROUNDWATER	SAMPLES
80		Dense, gray SAND; fine to medium grained, clean to trace silt			S-18
85		--------medium dense below 85 ft			S-19
90		--------occasional thin (<1 in.) interbeds of silt at 90 ft			S-20
95		--------dense below 95 ft			S-21
100		Very dense, gray SAND; fine grained, scattered fine, rounded gravel	100.0		S-22
		(1/21/2003)	101.5		
105					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY

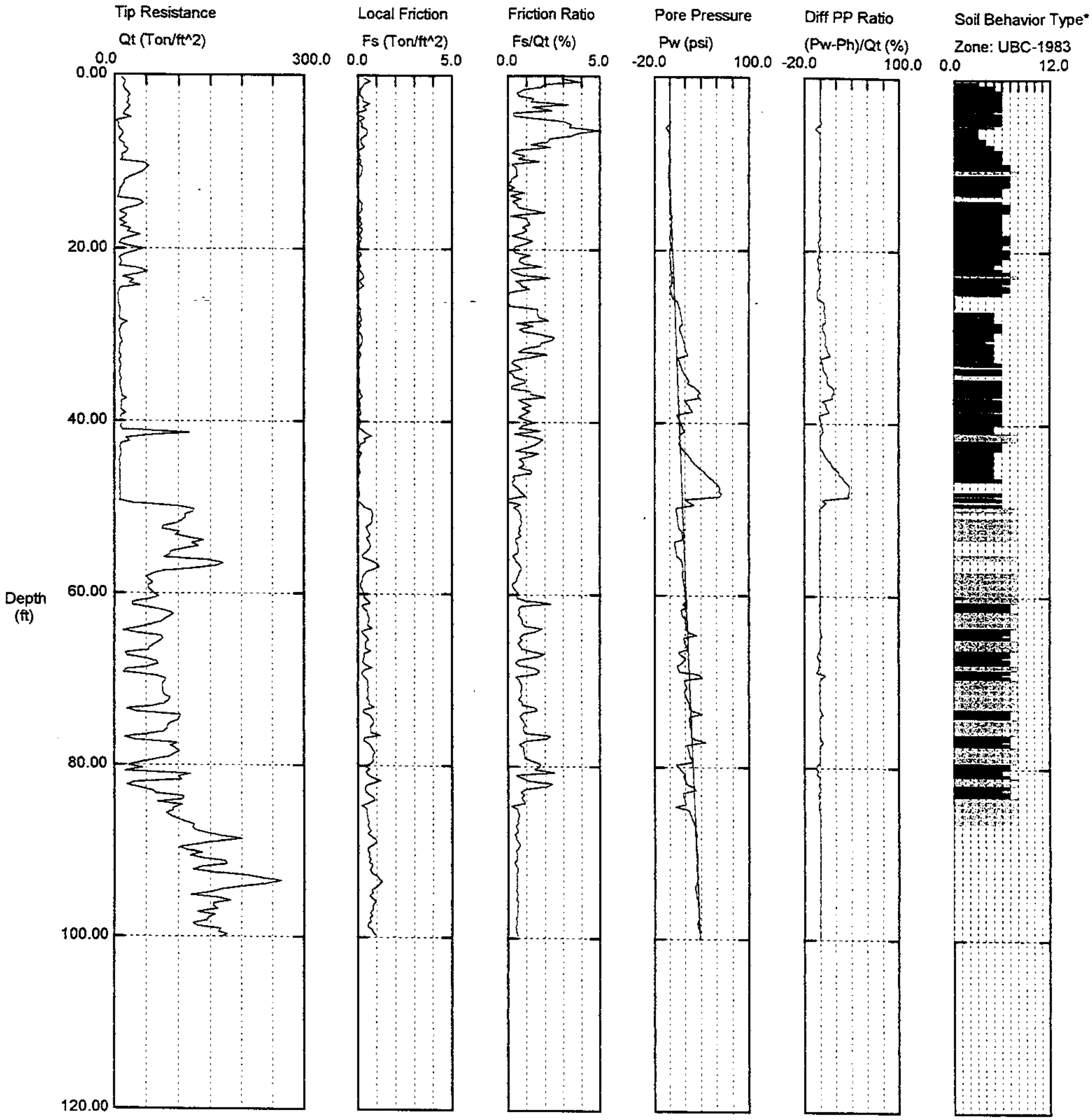
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-1 (cont.)

MAR. 2003 JOB. NO. 3798 FIG. 1A

R0004



Maximum Depth = 100.07 feet

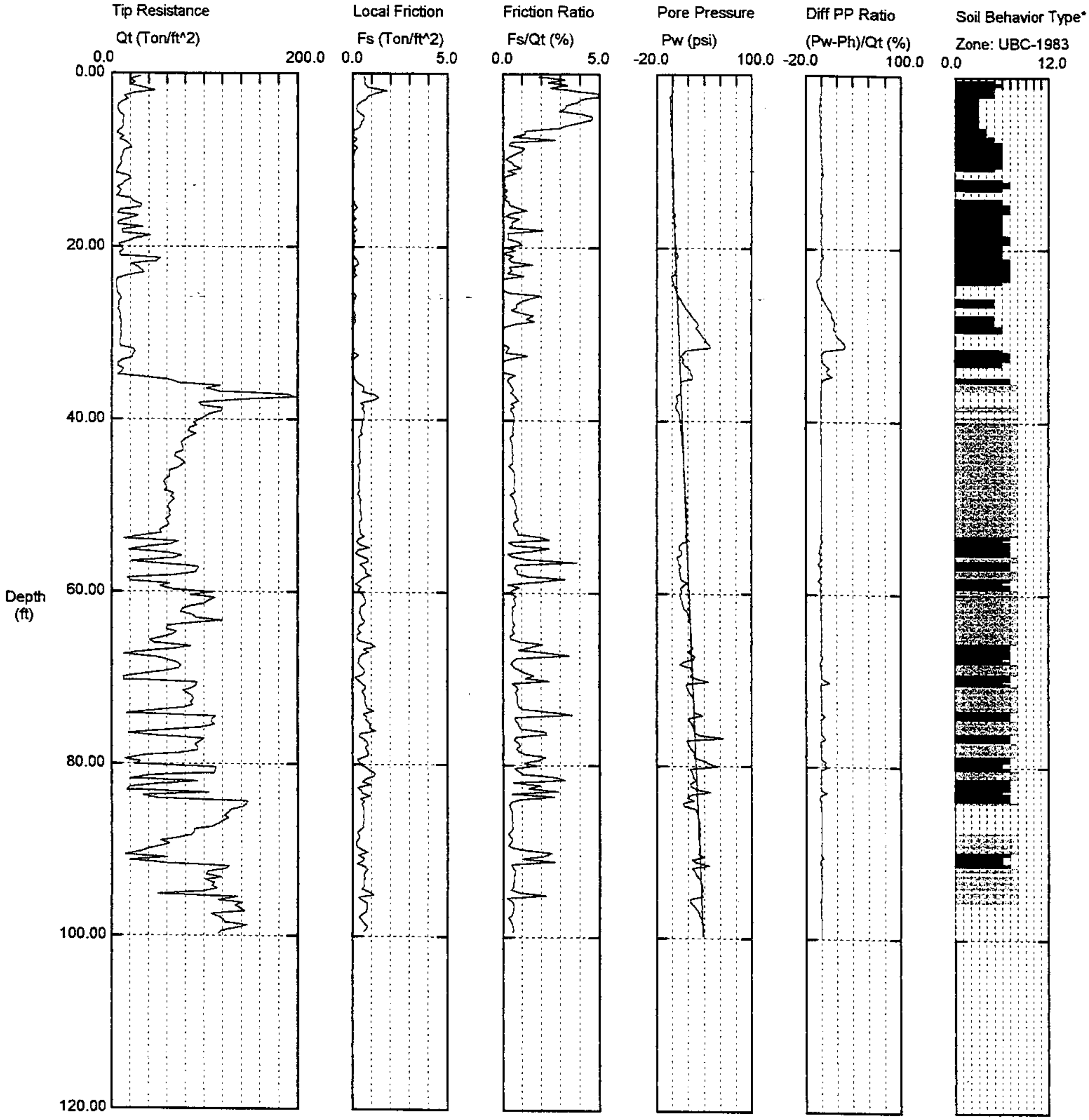
Depth Increment = 0.33 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

CONE PENETRATION TEST P-1

MAR. 2003 JOB NO. 3798 FIG. 2A

R0004



Maximum Depth = 99.74 feet

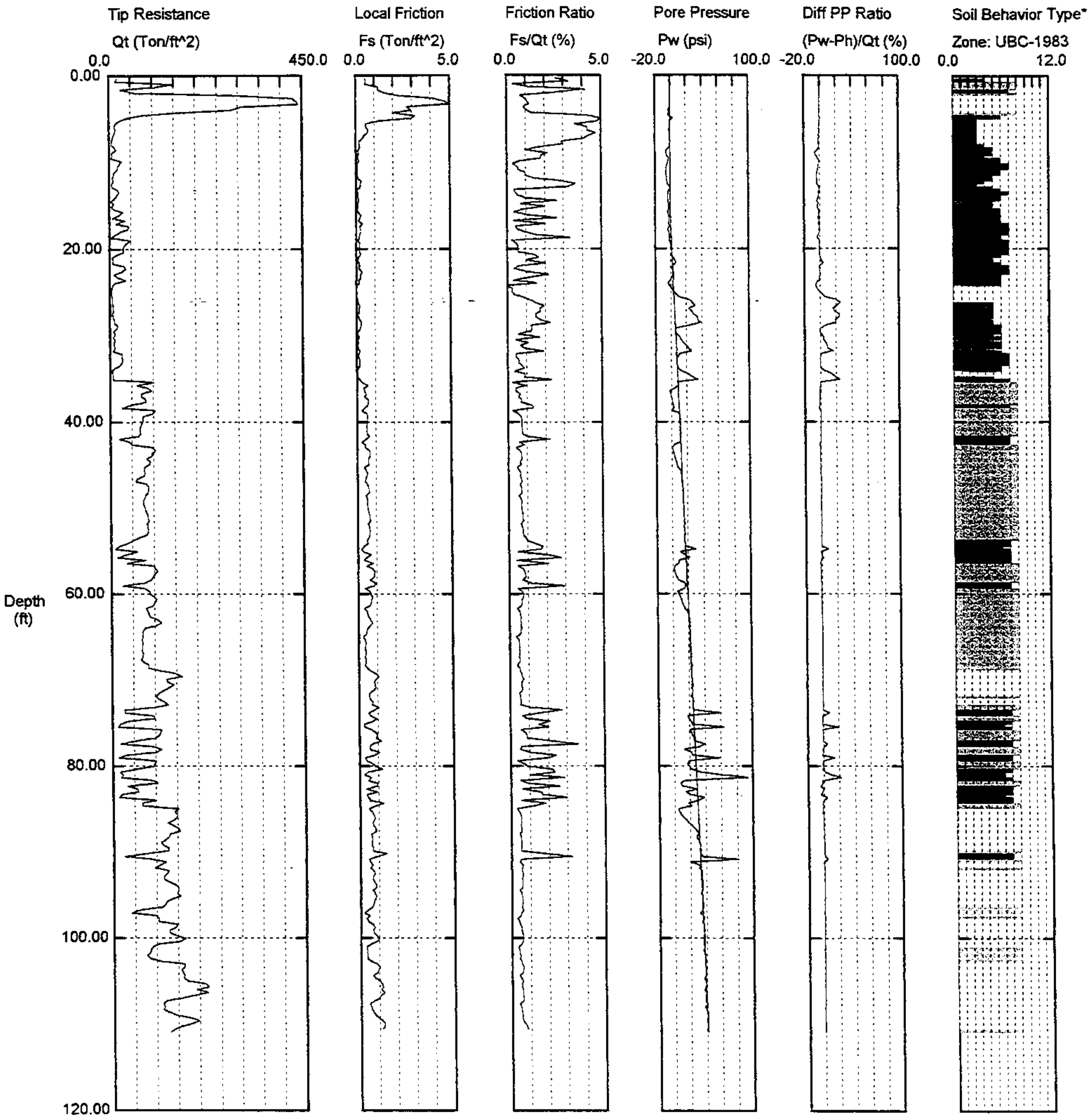
Depth Increment = 0.33 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

GRI

CONE PENETRATION TEST P-2

MAR. 2003 | JOB NO. 3798 | FIG. 3A



Maximum Depth = 110.89 feet

Depth Increment = 0.33 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

GRI

CONE PENETRATION TEST P-3

MAR. 2003 JOB NO. 3798 FIG. 4A

APPENDIX B

SITE-SPECIFIC SEISMIC HAZARD STUDY

General

GRI completed a site-specific seismic hazard study for the proposed fire station at the Portland Air National Guard base in Portland, Oregon. The general location of the site is shown on the Vicinity Map, Figure 1. The Site Plan, Figure 2, shows the proposed location and configuration of the building. We understand the fire station will be a two-story structure and will not have a basement or other significant below-grade structure.

Purpose and Scope

The purpose of our study was to evaluate the potential seismic hazards associated with regional and local seismicity and to estimate the effect those hazards might have on the site. Our work was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and on the subsurface conditions at the site, interpreted from geotechnical explorations and geophysical measurements made in the vicinity of the site. Specifically, our work included the following tasks:

1) A detailed review of the literature, including published papers, maps, open-file reports, seismic histories and catalogs, works in progress, and other sources of information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

2) Compilation, examination, and evaluation of existing subsurface data gathered at and in the vicinity of the site, including classification and laboratory analyses of soil samples. This information was used to prepare a generalized subsurface profile for the site.

3) Identification of the potential seismic events appropriate for the site and characterization of those events in terms of a series of generalized design events.

4) Office studies, based on the generalized subsurface profile and the generalized design earthquakes, resulting in conclusions and recommendations concerning:

    a) specific seismic events that might have a significant effect on the site,

    b) the potential for seismic energy amplification at the site, and

    c) site-specific acceleration response spectra for each of the design earthquakes.

This report describes the work accomplished and summarizes our conclusions regarding the nature of the seismic hazards at the site.

Geologic Setting

On a regional scale, the site lies at the north end of the Willamette Valley; a broad, gently deformed, north-south-trending topographic feature separating the Coast Range to the west from the Cascade Mountains to the east. The valley lies approximately 150 km inland from the surface expression of the Cascadia Subduction Zone, an active plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent. The configuration of these plates and the location, extent, and geometry of the surface expression of the subduction zone are shown schematically on the Tectonic Setting Summary, Figure 1B(a). The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs of the Gorda, Juan de Fuca, and Explorer plates and the over-riding North America Plate, as shown schematically on Figure 1B(b).

On a local scale, the site lies in the western margin of the Portland Basin, a large, well-defined, northwest-trending structure bounded by high-angle, northwest-trending, right-lateral strike-slip faults generally considered to be seismogenic. The distribution of these faults relative to the site is shown on the Regional Geologic Map, Figure 2B. Within the basin, some faults have been mapped on the basis of stratigraphic offsets and geophysical evidence, but accurate information regarding the precise location and extent of these faults is lacking, due largely to the scale at which geologic mapping in the area has been conducted and the presence of thick, relatively young basin-filling sediments that obscure underlying structural features. Other faults may be present within the basin, but clear stratigraphic evidence regarding their location and extent is not presently available.

Because of the proximity of the site to the Cascadia Subduction Zone and its location within the Portland Basin, three distinctly different sources of seismic activity contribute to the potential for the occurrence of damaging earthquakes. Each of these sources is generally considered to be capable of producing damaging earthquakes. Two of these sources are associated with deep-seated tectonic activity related to the subduction zone; the third is associated with movement on the local, relatively shallow faults within and adjacent to the Portland Basin.

Precise, quantitative information regarding historic seismic activity in the Pacific Northwest and in northwestern Oregon is sparse. Events that may have occurred in the region prior to settlement of the Oregon Territory in the mid-nineteenth century are speculative and have not been clearly identified in terms of location, magnitude, or frequency. From the mid-nineteenth century to the time of the installation of the first dependable seismometers in the area (about 1940), reliable information regarding location and magnitude is not available, although rough estimates of these parameters have been based on records of eyewitness accounts. Since about 1940, seismographic records of increasing sophistication and accuracy are available for local events larger than about 3.5 (M_L). We have examined a catalog (Open File Report 0-94-04) obtained from the Oregon Department of Geology and Mineral Industries (DOGAMI) containing a list of those earthquakes known to have occurred in Oregon during the period 1883-1993. We searched this catalog for all known earthquakes within the area bounded by 46°00′ N latitude on the north, 45°00′ N latitude on the south, 122°11′ W longitude on the east, and 123°11′ W longitude on the west. This area includes a 50-km radius of the project site. The only recent events that may have generated measurable accelerations in the vicinity of the project site are the 1962 Portland Earthquake and the 1993 Scotts Mills Earthquake. Peak horizontal bedrock accelerations at the project site due to these events were probably substantially less than 0.1 g.

Subsurface Conditions

The general area occupied by the subject site is underlain by a thick sequence of unconsolidated, fine- to coarse-grained sediments of Miocene to Quaternary age. Locally, these deposits consist predominantly of sandy silt, silty sand, and gravel in interbedded lenses and layers of limited lateral extent. These deposits are underlain by mudstone, siltstone, claystone, and sandstone beds of the Sandy River Mudstone. The Sandy River Mudstone is underlain by flood deposits of the Columbia River Basalt Group. The boundary between the overlying sedimentary materials and the underlying basalt is unconformable, indicating that a considerable period of time elapsed between the solidification of the last of the basalt flows and the deposition of the overlying sedimentary materials. The thickness of the overlying unconsolidated sediments varies widely within the basin, but is thought to be approximately 1,400 ft in the vicinity of the site, based on the logs of geophysical borings (MTD-1 and MTP-4) made by DOGAMI in 1993 in the vicinity of the Portland International Airport (Mabey and Madin, 1995).

Based on this information and the subsurface investigation conducted by GRI, we have developed a model of subsurface conditions for the project site. This model consists of a series of horizontal layers of semi-infinite extent, with the following characteristics:

Material Type	Average Thickness, ft	Total Unit Weight, pcf	Shear Wave Velocity, ft/sec
SILT	10	100	350
SAND	15	120	2,500
SILT	10	100	470 to 575
SAND	85	120	1,150 to 3,370
GRAVEL	1100	130 to 135	1,150 to 3,770
Sandy River MUDSTONE	120	145	2,500
Weathered BASALT	80	155	2,625
BASALT	Undefined	165	4,000

Seismicity

The geologic and seismologic information available for identifying the nature of the seismicity at the site is incomplete, and large uncertainties are associated with any estimates of the probable magnitude, location, and frequency of occurrence of earthquakes that might affect the site. The information that is available indicates that the seismic hazards at the site can be grouped into three independent categories: *subduction zone events* related to sudden slip between the upper surface of the Juan de Fuca plate and the lower surface of the North American plate, *subcrustal events* related to deformation and volume changes within the subducted mass of the Juan de Fuca plate, and *local crustal events* associated with movement on shallow, local faults within and adjacent to the Portland Basin. Based on our review of currently available information, we have developed generalized design earthquakes for each of these categories. The design earthquakes are characterized by three important properties: size, location relative to the subject site, and the peak horizontal bedrock accelerations produced by the event. In this study, size is expressed in Richter (local) magnitude (M_L), surface wave magnitude (M_S), Japanese Meteorological Association magnitude (M_{JMA}), or moment magnitude (M_W); location is expressed as epicentral or focal distance, measured radially from the subject site in kilometers; and peak horizontal bedrock accelerations are expressed in gravities (1 g = 980.6 cm/sec/sec).

Probabilistic Considerations. The probability of occurrence of an earthquake of a specific magnitude at a given location is commonly expressed by its return period, i.e., the average length of time between successive occurrences of an earthquake of that size or larger at that location. The return period of a design earthquake can be calculated once a project design life and some measure of the acceptable risk that the design earthquake might occur or be exceeded are specified. The US Army Corps of Engineers' document TI 809-04, "Seismic Design for Buildings," based on the USGS-prepared probabilistic spectral acceleration maps, evaluates the Maximum Considered Earthquake (MCE) ground motion with a 2% probability of exceedance in 50 years, resulting in a return period of about 2,500 years. This document recommends using three quarters of the MCE as the ground motion for the design of essential facilities, which corresponds to a ground motion with about 5% probability of exceedance in 50 years, resulting in a return period of about 1,000 years. The most recent probabilistic study for the Portland Metropolitan area (Wong, et. al., 2000) estimated the potential magnitude of ground shaking, and yielded peak ground accelerations at the site area of approximately 0.3 to 0.4 g for a 2,500-year return period.

Subduction Zone Event. Since subduction zone events have not occurred in the Pacific Northwest in historic times, estimates of their probable size, location, and frequency are generally based on comparisons of the Cascadia Subduction Zone with active convergent plate margins in other parts of the world and on geologic evidence suggests that seismic events of this type may have occurred in the Pacific Northwest in the geologic past. Published estimates of the probable maximum size of subduction zone events range from moment magnitude Mw = 8.0 to > 9.0. Published information regarding the location and geometry of the subduction zone indicates that minimum focal distances (measured from Portland) of 60 to 80 km are probable (Weaver and Shedlock, 1989). Published recurrence intervals, plus and minus one standard deviation, for these events range from 260 to 1,490 years (Adams, 1984 and 1990; Atwater, 1987 and 1988; Peterson and Darienzo, 1989 and 1991). We have chosen to represent the subduction zone event by a design earthquake of Mw = 8.5 at a focal distance of 70 km. This corresponds to a sudden rupture of half of the length of the Juan de Fuca-North American plate interface, placed at the closest approach of the interface, due west of Portland. It should be noted that this choice of a design earthquake is based primarily on an estimate of the capability of the subduction zone to produce a large earthquake, not on a probabilistic analysis of a demonstrated seismic history.

Based on the attenuation relationship published by Youngs and others (1997), a subduction zone event of this size at a focal distance of 70 km would result in a peak horizontal bedrock acceleration of approximately 0.21 g at the site.

Subcrustal Event. Estimates of the probable size, location, and frequency of subcrustal events in the Pacific Northwest are generally based on comparisons of the Cascadia Subduction Zone with active convergent plate margins in other parts of the world and on the historical seismic record for the region surrounding Puget Sound, where significant events known to have occurred within the subducting Juan de Fuca plate have been recorded. Published estimates of the probable maximum size of these events range from moment magnitude Mw = 7.0 to 7.5. Published information regarding the location and geometry of the subduction zone indicates that minimum focal distances of 40 to 60 km (measured from Portland) are probable (Weaver and Shedlock, 1989). Estimates of recurrence intervals applicable to the Portland area are not available. We have chosen to represent the subcrustal event by a design earthquake of magnitude Mw = 7.0 at a focal distance of 50 km. As with the subduction zone event, this choice is based on an estimate of the capability of the source region rather than on a probabilistic analysis of a historical record of events of this type.

Based on the attenuation relationship published by Youngs and others (1997), a subcrustal event of this size at a focal distance of 50 km would result in a peak horizontal bedrock acceleration of approximately 0.24 g at the site.

Local Crustal Event. Sudden crustal movements along relatively shallow, local faults in the Portland area, though rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood, since few of the faults in the area are expressed at the ground surface, and the foci of the observed earthquakes have not been located with precision. The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1841), it can serve as a guide for estimating the potential for seismic activity in the area.

Another method of estimating the magnitude to be expected of local crustal events involves an analysis of the lengths of local faults. The empirical relationship between fault rupture length and the magnitude of the resulting earthquake has been studied extensively (Matthiesen, 1984; Wells and Coppersmith, 1994). Based on the most recent study by Wong and others (2000), the longest mapped fault in the Portland Basin is the Portland Hills Fault, with a mapped extent of about 60 km and a corresponding characteristic earthquake magnitude ranging from about $M_L = 6.5$ to about $M_L = 7.1$, depending somewhat on the geometry of the fault at depth and the degree to which the fault is segmented, neither of which is well understood. Based on mapping by Beeson and others (1991), the fault passes within about 9 km of the project site.

In consideration of the size and use of the proposed structures at the site, and published information about fault characteristics, we have chosen to represent the local crustal seismicity by a $M_L = 6.5$ event occurring on the Portland Hills Fault, located 9 km from the site. The peak horizontal bedrock accelerations associated with an event of this size and epicentral distance, using the attenuation relationship of Boore, Joyner, and Fumal (1997), are on the order of 0.28 g.

Summary of Design Earthquake Parameters

In summary, we have chosen design events of three distinctly different types to represent the seismic events for this project. The subduction zone and subcrustal events were chosen on the basis of the seismic capability of the structures on which they are expected to occur, rather than on the more classic probabilistic approach, due to the lack of a reliable historic record of earthquakes of these types. The local crustal event was based on analyses of the best available record of historic crustal earthquakes in the Portland area and on the distribution of local, shallow faults and their proximity to the site. The peak accelerations that would be expected on bedrock exposed at the site due to the occurrence of these earthquakes were estimated using published attenuation relationships. The basic parameters of the design earthquakes are as follows:

Design Earthquake	Magnitude	Focal Distance, km	Mean Peak Bedrock Acceleration, g
Subduction Zone	8.5 (M_w)	70	0.21
Subcrustal	7.0 (M_w)	50	0.24
Local Crustal	6.5 (M_L)	9	0.28

Earthquake Motions

A series of acceleration-time histories (commonly referred to as "accelerograms") of well-studied earthquakes have been selected to represent each of the design events described above. These events were selected from the current inventory of the National Geophysical Data Center (NGDC) in Boulder, Colorado, and from the records available from the California Division of Mines and Geology in Sacramento, California. From the available records, corrected free-field and basement/ground floor accelerograms recorded at rock and stiff soil sites at focal distances similar to those chosen for the design events during earthquakes with magnitudes similar to the design events were chosen. These records were checked for obvious errors, missing data points, and other anomalies and were transformed into a uniform data format. The selected strong-motion records are as follows:

SUBDUCTION ZONE EVENT

Earthquake	Recording Station	Magnitude	Focal Distance, km	Peak Bedrock Acceleration, g
Michoacan	La Union	8.1 (M_s)	82	0.17
El Salvador	San Miguel	7.6 (M_s)	92	0.14
Lima	Arequipa	7.6 (M_s)	84	0.20
Valparaiso	Llolleo	7.8 (M_s)	73	0.45
Santiago	Univ of Chile	7.9 (M_s)	132	0.16
Nihonkai	Frofushi	7.7 (M_{JMA})	74	0.21

SUBCRUSTAL EVENT

Earthquake	Recording Station	Magnitude	Focal Distance, km	Peak Bedrock Acceleration, g
Adak	US Naval Station	6.8 (M_s)	69	0.19
Alaska	Kodiak	6.8 (M_s)	68	0.02
Puget Sound	Olympia	6.7 (M_s)	88	0.20
Nisqually	Olympia	6.8 (M_s)	53	0.22
W Washington	Olympia	7.1 (M_s)	62	0.28

LOCAL CRUSTAL EVENT

Earthquake	Recording Station	Magnitude	Focal Distance, km	Peak Bedrock Acceleration, g
Coalinga (Aft)	Oil Fields Fire Sta	6.0 (M_L)	11	0.22
Coyote Lake	San Ysidro	5.8 ([illegible]	12	0.42
Sierra Madre	Mt. Wilson	5.8 (M_L)	13	0.28
Morgan Hill	Anderson Dam	6.2 (M_L)	17	0.42
Helena	Carroll College	6.0 (M_L)	17	0.15
Whitter	Norwalk	5.9 (M_L)	16	0.24
Suruga Bay	Shimuza Factory	6.1 (M_J)	22	0.16

Estimated Site Response

The effect of a specific seismic event on the site is related to the type and depth of soil overlying the bedrock at the site and to the amount and type of seismic energy delivered to the bedrock beneath the site by the earthquake. For the project site, the characteristics of the soil and rock beneath the site were estimated by reviewing representative samples of soil and rock from the site, evaluating results from shear wave velocity measurements, and laboratory testing. The amount of energy associated with the design events has been described above. Subsurface information was combined with information regarding the type of energy involved (primarily its frequency content), derived from strong-motion records of similar earthquakes and used, in conjunction with PROSHAKE (EduPro Civil Systems, Inc.), to produce site-specific response spectra.

Peak Horizontal Ground Accelerations

Pseudoacceleration response spectra, based on the generalized subsurface profiles described above, the peak bedrock accelerations estimated for the design events, and the strong-motion records listed in the preceding tables have been prepared using PROSHAKE. However, these methods of estimating ground acceleration do not include the effects of liquefied soil conditions. The spectra were produced for the estimated ground surface of the proposed building foundation and at a depth 40 ft below the ground surface, immediately below the potentially liquefiable zone, damped at 5% of critical damping, from the larger horizontal component of each of the strong-motion records, scaled to match the estimated peak horizontal bedrock accelerations of the design events. For each of the design events, summary plots showing the mean response spectra at the ground surface and at a depth of 40 ft were prepared. The results of these analyses for the three design events are shown on Figure 3B and 4B.

Seismic Hazards

Field and laboratory studies have demonstrated that if saturated, loose to medium dense sands and soft to medium stiff, low-plasticity silts, are subject to cyclic shear stresses of a sufficient magnitude and duration, an increase in porewater pressure can result. As porewater pressure increases, the effective stress decreases, which results in a corresponding loss of shear strength in the material. The limiting case being if the porewater pressure ratio, r_u (i.e., the ratio of the porewater pressure to the total vertical soil pressure), approaches 100%, the material will lose most of its shear strength and deform as a viscous fluid (complete liquefaction). For porewater pressure ratios between 0 and 100%, a partial loss of shear strength can occur in response to the decrease in effective stress (partial liquefaction). As strength is lost, there is an increased risk of settlement.

In our opinion, during a strong seismic event there is a moderate to high potential for partial or complete liquefaction within the fill and alluvial sands and silts that exist in the upper 40 to 50 ft at the site. The liquefaction potential for the site was evaluated based on the soil profile disclosed in boring B-1 and results of cone penetration test probes P-1 through P-3. Depending on actual river and groundwater levels at the time of the earthquake, we estimate liquefaction-induced settlements at the site could be in the range of a few feet. Additionally, an earthquake with a larger magnitude producing the same acceleration at the site will result in larger settlements.

Based on the site topography, it is our opinion that the risk for earthquake-induced slope instability is low. Based on the elevation and location of the site, the risk of damage by tsunamis and/or seiches at the site is absent. Based on our review of geologic maps and available subsurface information, no faults are mapped on or near the site and it is our opinion that the potential for fault rupture at the site is low.

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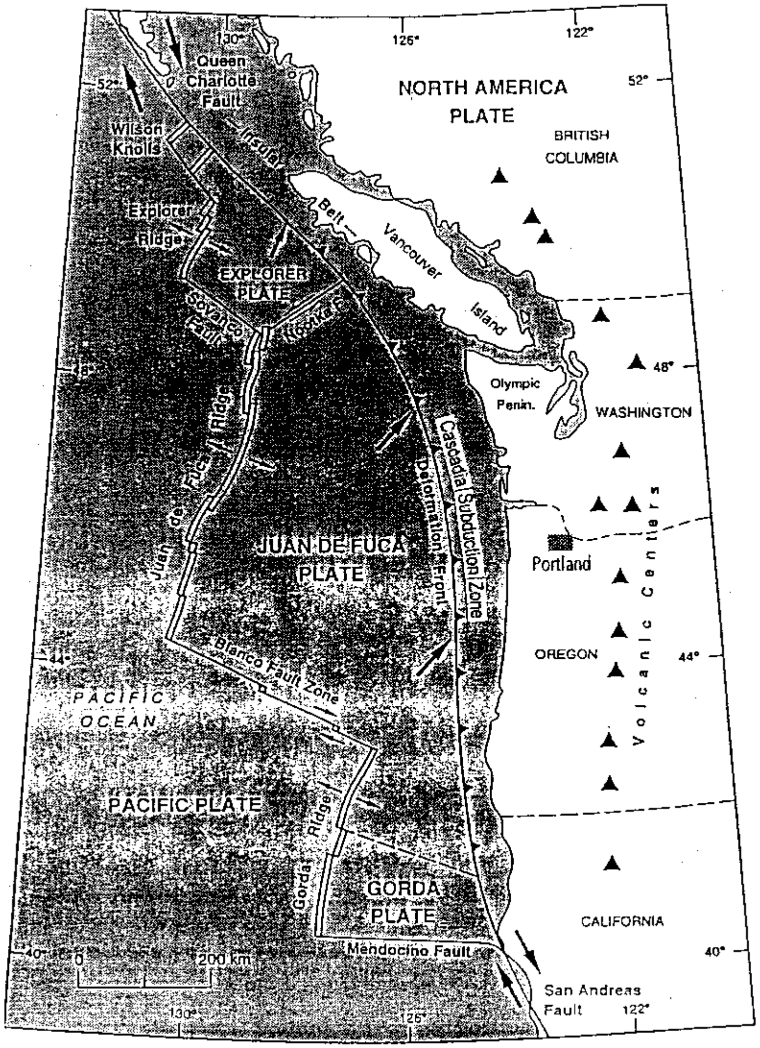
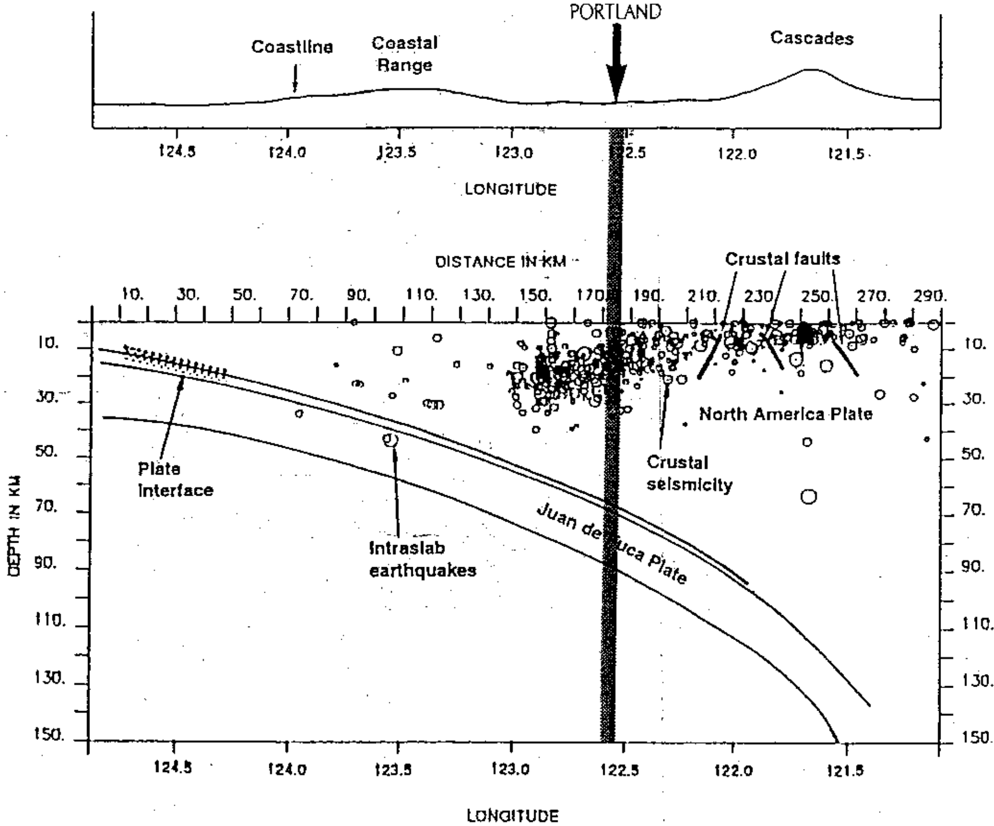


A) TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE (MODIFIED FROM DRAGERT AND OTHERS, 1994)



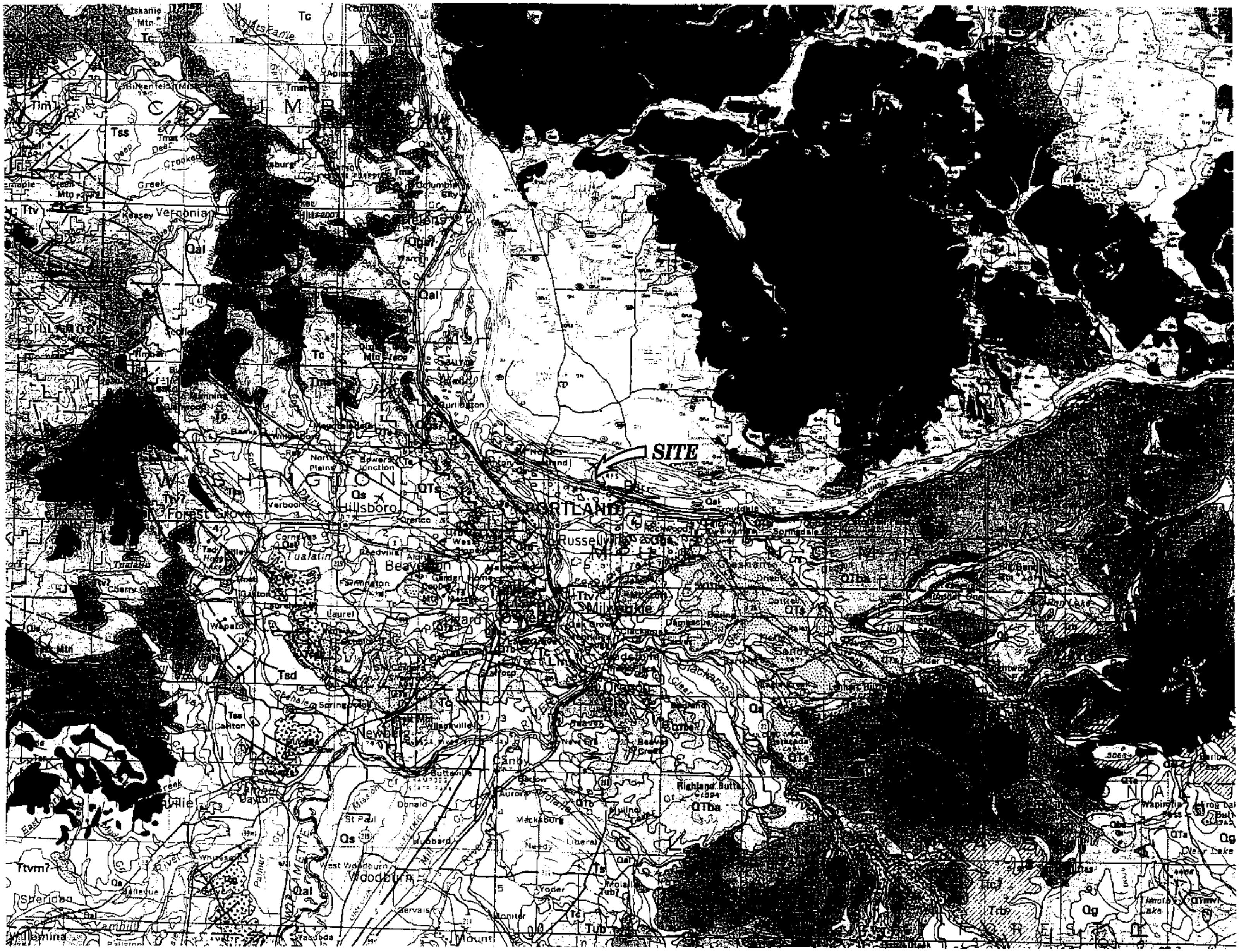
B) EAST-WEST CROSS-SECTION THROUGH WESTERN OREGON AT THE LATITUDE OF PORTLAND, SHOWING THE SEISMIC SOURCES CONSIDERED IN THE SITE-SPECIFIC SEISMIC HAZARD STUDY (MODIFIED FROM GEOMATRIX, 1995)

GRI

TECTONIC SETTING
SUMMARY

MAR. 2003 JOB NO. 3798 FIG. 1B

R0004



Contact — Approximately located

Fault — Dashed where inferred; dotted where concealed; queried where doubtful; ball and bar on downthrown side

Thrust fault — Dashed where inferred; dotted where concealed; queried where doubtful; sawteeth on upper plate

Strike and dip of bed

BASE MAP FROM: WALKER, G.W., AND MACLEOD, N.S., 1991, GEOLOGIC MAP OF OREGON: U.S. GEOLOGICAL SURVEY

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North

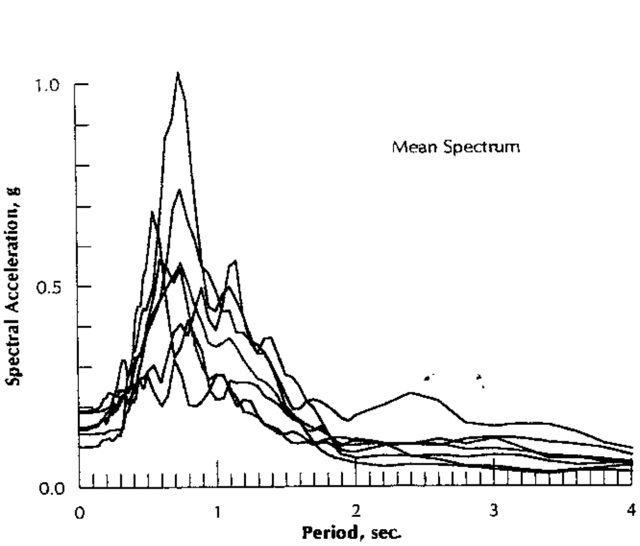
0 10 20 MILES

0 10 20 KILOMETERS

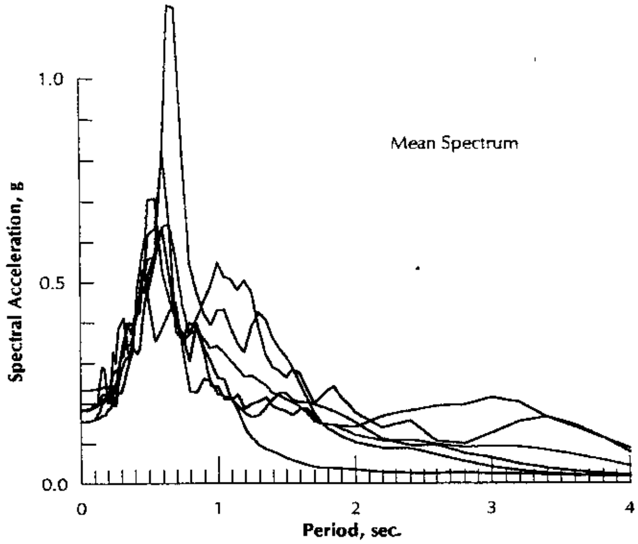
GRI

REGIONAL GEOLOGIC MAP

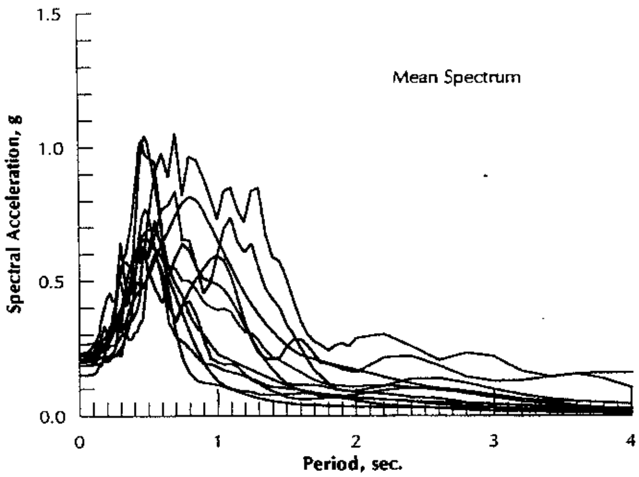
MAR. 2003 | JOB NO. 3798 | FIG. 2



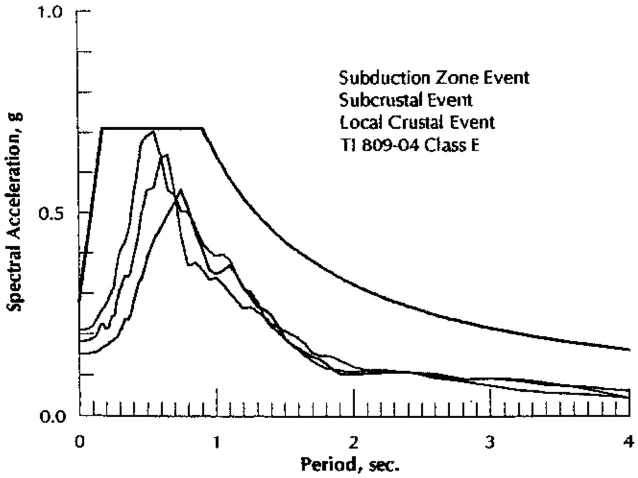
a) Subduction Zone Event Response Spectra



b) Subcrustal Event Response Spectra



c) Local Crustal Event Response Spectra

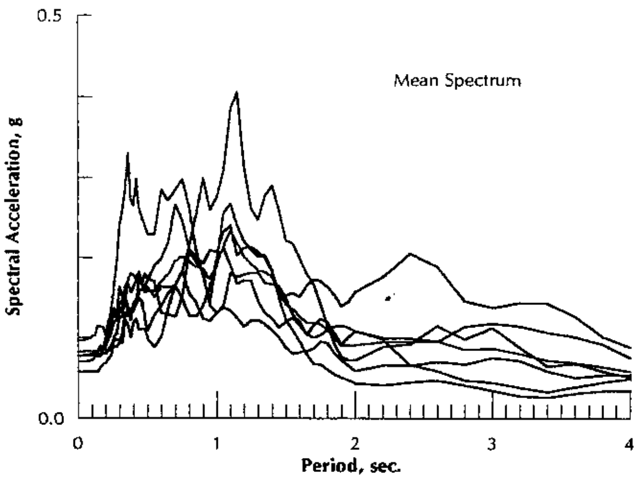


d) Summary of Mean Event Response Spectra and Army Corps of Engineers TI 809-04 Design Response Spectrum for Soil Class E

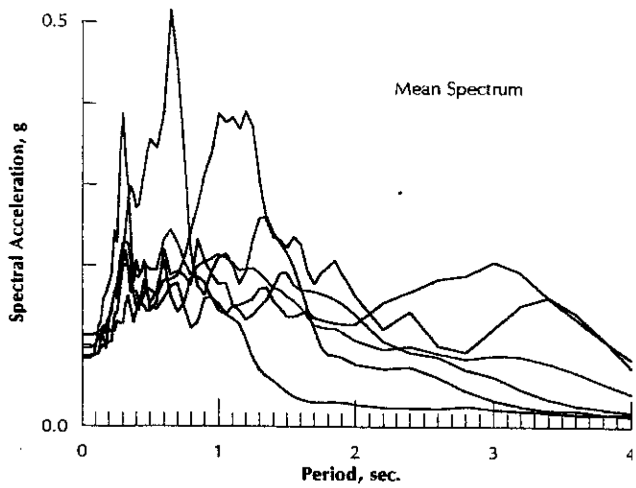
GRI

INDIVIDUAL AND MEAN RESPONSE SPECTRA
(AT THE GROUND SURFACE)

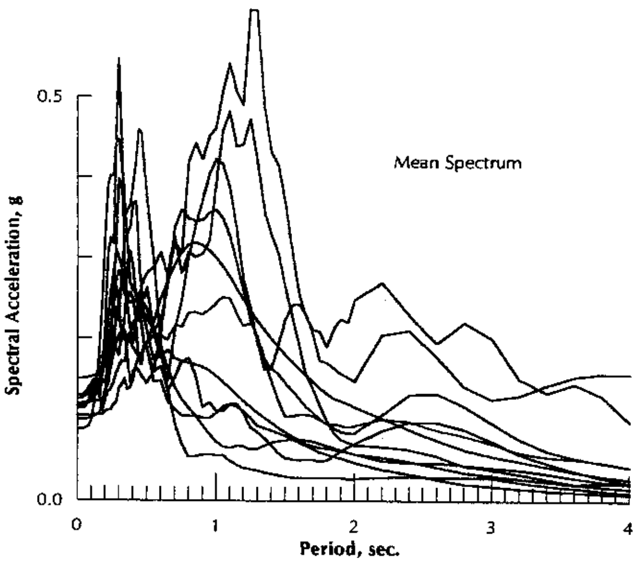
MAR. 2003 | JOB NO. 3798 | FIG. 3B



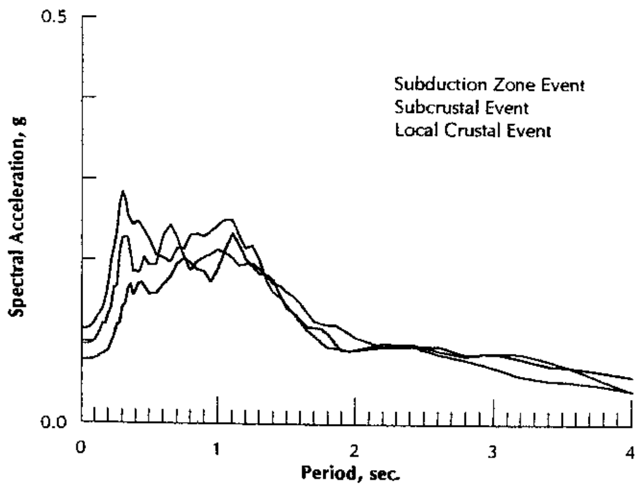
a) Subduction Zone Event Response Spectra, d - 40 ft



b) Subcrustal Event Response Spectra, d - 40 ft



c) Local Crustal Event Response Spectra, d - 40 ft



d) Summary of Mean Event Response Spectra, d - 40 ft

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INDIVIDUAL AND MEAN
RESPONSE SPECTRA
(AT A DEPTH OF 40 FT BELOW GROUND SURFACE)

MAR. 2003 | JOB NO. 3798 | FIG. 4B