Geotechnical Investigation
Child Care Center
SW 13th Avenue at SW Market Street

January 25, 2002

Prepared For:
Portland State University
PO Box 751
Portland, Oregon 97207-0751

Prepared By:
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CDM Project No. 20542-34240
January 25, 2002

Mr. Burt Ewart
Portland State University
P.O. Box 751
Portland, Oregon 97207-0751

Subject: Geotechnical Investigation
Child Care Center
SW 13th Avenue at SW Market Street
Portland State University
Portland, Oregon

Dear Mr. Ewart:

CDM Jessberger, a division of Camp Dresser and McKee Inc. (CDM) is pleased to present the results of our geotechnical investigation for the Child Care Center project on the Portland State University campus in Portland, Oregon. Our scope of work for this project was outlined in our proposal dated September 26, 2001. The work was authorized by your October 10, 2001 amendment to our master retainer agreement with the Oregon University System.

Background Information

Site Description
As shown on the Vicinity Map, Figure 1, the project site is on the south side of SW Market Street between SW 12th and 13th Avenues immediately north of Parking Structure No. 3. The northeast corner of the site is developed with the existing child care facility, a three story brick structure with a basement. The northwest corner of the property is landscaped with grass and a berm. The south end of the property is developed with an asphalt parking lot and concrete walkway. Site grades vary from approximately +176 feet in the parking lot to +170 feet in the play area. The grass area is near elevation +173 feet.

Historical Information
CDM reviewed historical Sanborn Fire Insurance Maps to determine past development on the property. The maps indicate that as many as 7 buildings once occupied the site. The buildings consisted of dwellings and the current child care center. Copies of the Sanborn Fire Insurance Maps are attached to this report as Appendix C.
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**Project Description**
The proposed child care center project will consist of an addition on the west side of the existing structure. The addition will be a three-story, steel-framed structure. The first floor will match the existing building's first floor elevation, approximately elevation +170 feet. A partial basement will be located near the existing structure. Maximum columns loads of 108 kips (DL + LL) are planned.

**Field Investigation**
The subsurface conditions at the site were investigated with two borings drilled to 51.5 and 36.5-foot depths. The explorations are located approximately on the Exploration Location Plan, Figure 2. The borings were drilled using mud-rotary methods. Details of the field investigation and exploration methods are presented in Appendix A. Final logs of the explorations are presented on Figures A-2 and A-3.

**Laboratory Testing**
The laboratory testing program consisted of visual classification, water contents, unit weights, and a consolidation test. The test procedures are described in Appendix B.

**Subsurface Conditions**

**Soil Conditions**
Based on the explorations, the subsurface conditions at the site are described by three units. From the ground surface downward, these units are: 1) FILL, 2) Loose to medium-dense SILT and SAND, and 3) Medium-dense to dense fine SAND. A summary of the units is presented below. Please refer to the boring logs Figures A-2 through A-4 for details.

**FILL:** The site is mantled with 5 to 7 feet of undocumented FILL. We assume that randomly backfilled basements or partial basements account for the FILL. The FILL typically consists of sandy SILT with some organics, crushed rock, bricks, and plastic debris. Standard Penetration Test (SPT) blow counts ranged from 6 to 12 blows per foot (bpf) and averaged 10 bpf. Moisture contents ranged from 20 to 31 percent and averaged 25 percent.

**SILT and fine SAND:** Sandy SILT and fine SAND underlies the FILL near elevations +168 to +169 feet. The SILT near the top of this unit contains trace clay. The SILT grades sandy with depth. SPT blow counts ranged from 8 to 19 bpf and averaged 13 bpf. Moisture contents ranged from 21 to 43 percent and averaged 30 percent.
Dense fine SAND: Below approximately elevation +148 feet in boring CDM-1 and elevation +141 feet in boring CDM-2, the fine SAND soils grade medium-dense to dense. SPT blow counts ranged from 25 to 39 bpf and averaged 33 bpf. Moisture contents ranged from 18 to 32 percent and averaged 23 percent.

Groundwater
Mud-rotary drilling methods preclude the direct measurement of groundwater during drilling. A review of previous auger borings and well logs from the Portland State University campus indicates that groundwater lies at depths greater than 100 feet below the ground surface.

Seismic Hazards
Tectonic Setting
Regionally, the site is located at the north end of the Willamette Valley, a broad north-south trending basin between the Coast Range to the west and the Cascade Mountains to the east. The surface expression of the Cascadia Subduction Zone is located approximately 100 km off the coast of Oregon (approximately 200 km west of the Portland area). This zone represents the active plate boundary where the Juan de Fuca plate subducts beneath the west edge of the North American continental plate.

Locally, the site is located at the west edge of the Portland Basin. The Portland Basin is a deep, northwest to southwest trending basin underlain by up to 300 meters of Miocene flood basalt and Pliocene to Pleistocene lavas that are interfingered with alluvial sediments. The basin is generally capped with alluvial soils deposited on Quaternary catastrophic flood sediments. Deep borings on the PSU campus indicates that catastrophic flood sediments and the Troutdale Formation overlie basalt, the latter of which was logged at approximately 230 feet.

Seismogenic Zone
General
In Oregon, three seismogenic zones have been identified. Two zones relate to the Cascadia Subduction Zone (CSZ) at and near the plate boundaries and the third zone relates to crustal deformation within the North American plate (crustal zone). The CSZ is divided into the eastward-dipping interface between the two plates (plate interface) and seismicity within the subducting Juan de Fuca plate (intraslab sources).
Plate Interface

Earthquakes greater than magnitude 8 (moment magnitude) are predicted for the plate interface event. These events would occur offshore at depths of 10 to 20 kilometers at the interface between the North American plate and the subducting Juan de Fuca plate. Such events occur as a sudden rupture along the Juan de Fuca subduction zone with possible rupture lengths ranging from 250 to 450 kilometers. Although no subduction zone events are recorded historically, geologic evidence suggests that recurrence intervals may range from 300 to 2,000 years (mean recurrence of 350 to 550 years in Oregon). Based on historical tsunami records in Japan and carbon dating of coastal deposits, scientists recently determined that the last subduction zone event occurred in the year 1700.

Intraslab Sources

Earthquakes with maximum moment magnitudes of 7.0 to 7.5 are suggested for the intraslab event. Intraslab events occur beneath the Coast Range at depths of 40 to 70 kilometers. Intraslab events that confirm the potential for such earthquakes in the Pacific Northwest were recorded in 1949 (Olympia, Washington), 1965 (Puget Sound), and 2001 (Olympia, Washington). In Oregon, observations indicate a possible 150-year recurrence time for magnitude 6.5 events and 1 to 50 year recurrence intervals for smaller events.

Crustal Zone

Crustal earthquakes are shallow, occurring within 30 km of the ground surface. The maximum earthquake generating potential of crustal faults in Oregon is not yet known, however, worldwide experience has shown that earthquakes to magnitude 6.5 can occur at locations where active faulting is not known. Oregon has recently experienced two, relatively large crustal earthquakes, the Scotts Mills (magnitude 5.6) on March 25, 1993, and the Klamath Falls earthquake (magnitude 5.9) on September 20, 1993.

Design Earthquake

Based on available information, we considered the following earthquakes (representing 500-year recurrence interval events) for seismic design. The peak site bedrock accelerations were estimated using a deterministic approach and commonly accepted attenuation relationships.
A study of seismicity in Oregon was completed for ODOT in January 1995 ("Seismic Design Mapping, State of Oregon," Geomatrix Consultants). The study generated sets of contour maps showing predicted peak bedrock accelerations throughout the state for earthquakes with return periods of 500, 1,000, and 2,500 years. For this site, the maps indicate a peak horizontal bedrock acceleration of 0.195g for a 500-year return period or a 10% chance of exceedance in a 50-year design life. The peak bedrock accelerations were based on a probabilistic analysis of all earthquake sources. The 0.195g value compares favorably to the range of values noted in the table for the three probable earthquake sources.

**Ground Shaking Amplification**

The strength of ground shaking within bedrock generally attenuates with distance from the earthquake source. The earthquake motions experienced at the ground surface may be greater or less than the underlying bedrock motions because of soil conditions or topography. We evaluated the influence of strong ground shaking on site soil conditions using the computer program PROSHAKE.

PROSHAKE is a one-dimensional, columnar analysis of the earthquake motion propagation from the bedrock through the soil profile. For our analysis, we developed a typical soil profile at the PSU campus using our boring logs and previous deep borings logs. Shear wave velocities were based on average values for similar soil types determined by the DOGAMI Earthquake Hazard Maps of the Portland Quadrangle (GMS-79) and downhole seismic testing performed on the campus by Shannon and Wilson in 1972. Input motions from the Mexico City subduction zone earthquake and a magnitude 6.5 synthetic crustal earthquake were used. These earthquake motions were scaled to the peak bedrock accelerations presented in the Design Earthquake section of this report. Only the crustal and subduction zone interface earthquake were analyzed because their close proximity and size will likely have the greatest influence at the site.
The PROSHAKE analyses indicate that the soil profile at the site will tend to amplify both earthquakes, in particular the low frequency subduction zone quake. The results indicate that the bedrock accelerations are amplified 2 and 1.7 times for the subduction zone and crustal earthquakes, respectively. The Ground Motion Amplification Map, Portland Quadrangle (DOGAMI GMS-79) indicates an amplification factor of 1.0 to 1.4 for the site.

**Fault Rupture**

The closest potentially active fault mapped in the Geomatrix report is the Portland Hills Fault. The Geologic Map of the Portland Quadrangle (DOGAMI GMS-75) maps this fault along the base of the Portland Hills escarpment north of Burnside Street. At Burnside Street, the mapped fault line shifts and passes through downtown Portland approximately 1800 feet east of the site. If the fault continued along the escarpment base, it would pass very close to the PSU campus.

Although there was no definitive evidence for activity on the Portland Hills Fault Zone, the Geomatrix report classified the fault zone as having relatively high probability of activity. Slip rate estimates ranged from 0.07 to 1.0 mm/year. Because no fault traces have been identified at the site, it is our opinion that the potential for fault rupture at the site is low.

**Liquefaction**

Liquefaction, or the loss of shear strength, can occur in loose saturated sands and some fine-grained soils during an earthquake. Plastic silts and clays usually do not liquefy. Liquefaction commonly occurs in geologically young alluvial or fill soils. Evidence of liquefaction commonly includes "sand boils" at the surface, an increase in soil density and a corresponding decrease in volume manifested by surface subsidence.

The Liquefaction Susceptibility Map, Portland Quadrangle (DOGAMI GMS-79) maps no liquefaction susceptible soils at the site. Based on the deep groundwater level and the relative density if the sandy soils, it is our opinion that the site is not susceptible to liquefaction.

**Lateral Spreading/Seismic Slope Stability**

The Lateral Spread Displacement and Dynamic Slope Instability Map, Portland Quadrangle (DOGAMI GMS-75) maps no potential lateral spreading or dynamic slope instability on the site. Based on the relatively level site and no potentially liquefiable soil, it is our opinion that the site will remain stable during a seismic event.
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**Tsunami/Seiche**  
A tsunami is produced when a major fault under the ocean floor moves vertically and shifts the water column above it. A seiche is a periodic oscillation of an inland body of water resulting in changing water levels, sometimes caused by an earthquake or earthquake induced landslide. The site is far removed from the Pacific Ocean and lies about 150 feet above the Willamette River, therefore the risk of tsunami or seiche flooding is non-existent.

**Comments and Recommendations**

**General**  
The site is mantled with 5 to 7 feet of undocumented FILL from past site development. SILT and fine SAND soils underlie the FILL to the maximum explored depth of 51.5 feet. The principal geotechnical site issues include the support capability of the undocumented FILL and wet weather grading. These issues and others are discussed in the following sections.

**Weather Constraints**  
The native soils are fine-grained and sensitive to excess moisture. The native SILT/SAND cannot be compacted to structural fill specifications during wet weather periods. Instead, imported granular soils must be used. Additionally, the on-site soils may become very soft under construction traffic during wet weather, requiring a significant thickness of crushed rock in order to provide trafficability. Our experience indicates that 18 inches of crushed rock placed on a geotextile will typically support heavy, repeated truck traffic during wet weather periods.

**Site Preparation/Grading**  
We recommend stripping all grass, root mats, loose surficial and buried organic soils beneath structures, walls, pavements, and areas to receive structural fill. The borings encountered fills to approximately elevation +168 to +159 feet. Deeper fills may occur where buried basements or old tanks exist. The Sanborn Fire Insurance Maps (Appendix C) may assist in identifying backfilled basements.

The exploration data indicates that 1 to 2 feet of undocumented FILL occurs below proposed finished floor grade (+170 feet). The slab and base aggregate will place the floor subgrade at approximately +169 feet, leaving approximately one foot of FILL. Because the FILL consistency varies significantly, we recommend removing all FILL beneath the floor slabs, footings, and retaining walls, and backfilling the overexcavation with compacted structural fill.
When stripping and overexcavation is complete, we recommend that the engineer review the subgrade to verify the removal of the surface organics and FILL.

**Excavations**

FILL and stiff/medium-dense SILT and SAND underlies the site. The FILL and SILT/SAND are most similar to the OSHA description of Type C soils. Type C soils are allowed temporary excavations slopes of 1.5H:1V. In our opinion, all temporary on-site slopes should be limited to 1.5H:1V. Adjustments to the temporary slope inclinations may be necessary depending on the actual conditions encountered (bedding inclinations, the presence of water seepage, control of surface water, etc.). Slopes must be protected from erosion during wet weather.

Excavations should not penetrate a 2H:1V line extended down from existing footings unless the footing is underpinned.

If permanent slopes are used, we recommend a 2H:1V maximum slope inclination, adequately vegetated to prevent erosion.

**Temporary Shoring**

Proposed excavations may require maximum 8-foot cuts. If temporary sheet-pile or soldier pile walls are required, use the pressure diagram presented on Figure 3 for the design.

**Spread Footing Foundations**

The proposed child care center project may be supported on conventional foundations bearing entirely on the native SILT and SAND soils or a uniform thickness of structural fill. Individual footings should not bear on a combination of native soils and structural fill. We recommend a maximum allowable design bearing capacity of 3,000 pounds per square foot (psf). The allowable bearing pressure may be increased by 1/3 for short-term seismic loading conditions.

**Figures 4 and 5** present our settlements estimates for column and continuous footings bearing on the native soils. We suggest that footing settlements be limited to 0.5 to 0.6 inches under real live loads. The estimated settlements can be reduced by placing the footings on compacted, granular structural fill. **Figure 6 and 7** present our settlement estimates for column and continuous footings placed on 2 feet of compacted structural fill. The bearing pressures presented on the figures are "real" pressures defined as the sustained service loads (i.e. the dead load plus sustained live loads).
We note that the soils conditions and properties for this project are similar to those at the adjacent Parking Structure No. 3, although the design loads are less. Therefore, if 3,000 psf design bearing pressure is used, some overexcavation and replacement structural fill may be required to meet suggested settlement criteria. Alternatively, a lower allowable bearing pressure (say 2,500 psf) could be considered.

The settlement estimates assume that there is no stress influence from adjacent footings. Footings located within 2 times the footing width (2B) from each other will increase the stress beneath the adjacent footing, resulting in increased settlement.

We recommend minimum foundation dimensions of 18-inches wide for continuous footings and 24 inches for column footings. Footings must be located a minimum 18 inches below the lowest adjacent grade for frost protection and bearing considerations. Any structural fill placed beneath footings must extend beyond the footing at a 0.5H:1V slope as shown on Figure 8.

Stress/Settlement Influence
New footings constructed adjacent to existing foundations will result in increased stress and settlement beneath the existing structure. We recommend placing new footings at least 2 footing widths (2B) away from existing footing to reduce stress influences.

Retaining Walls
For free-standing retaining walls, designed to deflect at the top, we recommend using a 35 pcf active equivalent fluid pressure for design. For walls restrained from top deflections, a 55 pcf equivalent fluid pressure is recommended. These recommendations assume that the retaining wall backfill is granular soil, and no surcharge effects exist. Significantly higher lateral earth pressures will occur if the walls are backfilled with cohesive soils and/or surcharge loads exist. Traffic surcharges can be accommodated by increasing the effective wall height by 2 feet. Please advise this office of differing design assumptions for a revision of the above recommendations.

We recommend installing a drainage system behind all retaining and basement walls to preclude hydrostatic pressures. A retaining wall drain detail is shown on Figure 9.

Lateral Resistance
Lateral loads may be resisted by friction between the foundation and the underlying soils, and by passive earth pressure on the face of the footing or buried walls. The ultimate passive resistance of compacted, level backfill may be assumed equal to a 350 pcf equivalent fluid
pressure for both the static and dynamic condition. A friction coefficient of 0.35 may be used for footings placed on undisturbed SILT/SAND or structural fill.

**UBC Design Criteria**
Based on the boring logs, it is our opinion that the following UBC seismic design factors best represent the subsurface conditions at the site.

- Seismic Zone Factor Z = 0.3
- Soil Profile Type = S_0
- Seismic Coefficient Ca = 0.36
- Seismic Coefficient Cv = 0.54
- Near-Source Factor Na = 1.3
- Near-Source Factor Nv = 1.6

**Floors**
For typical floor loads less than 100 psf, slab-on-grade floor systems may be founded over a 6-inch zone of compacted crushed rock similar to the structural fill specifications. This recommendation assumes that the native SILT/SAND subgraded is in a firm, non-yielding and undisturbed condition at the time of slab construction.

If moisture sensitive coverings (tile, sheet vinyl, carpet, etc.) are placed over the slab, we recommend vapor barrier protection beneath all slabs-on-grade. The vapor barrier should be a high quality material designed for long-term stability when buried in a soil environment. We recommend placing the vapor barrier over the crushed rock and pouring the concrete slab directly on the vapor barrier. Minimum reinforcing (No. 4 rebars at 12 inch eachway) is recommended.

**Drainage**
A continuous footing drain is recommend around the perimeter of the building. A recommended footing drain detail is presented on Figure 9. Connect the footing drain to the storm system at an elevation that will preclude storm water from entering the footing drains. Final site surface grades should be designed to slope away from structures. Connect roof drain systems to the storm drainage system.
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**Structural Fill**
The on-site soils are silty and fine-grained. These soils are sensitive to excess moisture and will be difficult to moisture condition during wet weather. Because of this fact, we recommend using imported granular material for structural fill (i.e. fill placed below pavements, future building areas, sidewalks, etc.).

For this project, we recommend using ¼-inch minus crushed aggregate (ODOT Section 00680) for imported granular fill. If you desire to consider other materials, they should consist of free-draining (generally less than 5% passing the No. 200 sieve) granular material (sand, gravelly sand, sandy gravel, or well-graded gravels), free of organics, debris, or other deleterious matter. The maximum particle size should not exceed 2 inches. The material may be derived from naturally occurring geologic deposits or from crushing operations on bedrock or larger gravel, cobbles, or boulders. All particles of the fill should be sound, durable, unweathered and be capable of withstanding compaction, as specified, and construction traffic without crushing, fracturing, or otherwise altering the original gradation.

**Compaction Requirements**
Structural fill should be spread in maximum 9-inch loose lifts for compaction by heavy, self-propelled or tractor-towed compactors and maximum 6-inch loose lifts for light, manually guided compactors. Each lift should be thoroughly compacted to the required criterion with equipment suitable to the soil types being compacted. We recommend that the structural fill attain minimum dry densities based on ASTM D 1557 (AASHTO T180), modified Proctor, as noted below. Prior to compacting each lift, the fill should be properly moisture conditioned by the addition of uniform applications of water or by drying, as required, to achieve a moisture content which is within + 2 percent of the optimum moisture content as determined by ASTM D 1557 (AASHTO T180). All fill surfaces should be firm and yield only slightly beneath rubber-tired construction equipment. Fills which rut, pump, or weave should be considered to possess excess moisture and are not acceptable. These should be removed and replaced with fill material of proper moisture content or dried to the proper moisture content as specified herein.
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<td>Base Rock Layer</td>
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<td>Beneath Pavements</td>
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<td>Below 3 ft. of grade</td>
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Notes:  
1) Where conflicts occur between values, the higher percentage shall govern.  
2) Imported granular backfill should be approved by the engineer prior to delivery.  
3) Use lightweight, manually guided compactors within 3 feet of embedded walls.

General Notes  
The recommendations in this report have been prepared for design of the proposed Child Care Center project in Portland, Oregon as understood at this time and as described in this report. In the event that changes in the design or location of the structure occur, the conclusions and recommendations contained herein should not be considered valid unless verified in writing by CDM Jessberger.

This report was prepared solely for the Owner and Engineer for the design of the project. We encourage its review by bidders and/or the Contractor as it relates to factual data only (logs of borings and laboratory data). The opinions and recommendations contained within the report are not intended to be nor should they be construed to represent a warranty of subsurface conditions but are forwarded to assist in the planning and design process.

If, during construction, unexpected subsurface conditions are encountered, we should be notified at once so that we may review such conditions and revise our recommendations, if
necessary. We request that we be retained to review the applicable portion of the plans and specifications for the project prior to bidding for conformance to our recommendations.

We would be pleased to provide additional input, as necessary, during the design process and to provide on-site observations during construction. Please feel free to contact us for this work as well as for any questions you might have regarding this report.

Very truly yours,

CDM Jessberger, a division of Camp Dresser & McKee Inc.

Lloyd J. Reitz, P.E.
Geotechnical Engineer

Robert J. Strazer, P.E.
Associate Geotechnical Engineer

Attachments: Figures 1 through 9
Appendix A
Appendix B
Appendix C