NAMPA

City of Nampa

Wastewater Treatment Plant Phase I Upgrades: Group A–Liquid Stream Upgrades

Volume 6—Supplementary Information

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CONTRACT DOCUMENTS

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Strata Geotechnical Engineering Evaluation – September 8, 2014

Geotechnical Engineering Evaluation
Nampa Wastewater Treatment Plant Upgrades
Nampa, Idaho

Prepared For:
City of Nampa
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September 8, 2014

STRATA
A PROFESSIONAL SERVICES CORPORATION
Integrity from the Ground Up

September 8, 2014
File: BO13170D

City of Nampa
c/o Mr. Zach Dobroth, E.I.T.
Brown and Caldwell
950 W. Bannock Street, Suite 250
Boise, Idaho 83702

RE: **Geotechnical Engineering Evaluation**
Nampa Wastewater Treatment Plant Upgrades
Nampa, Idaho

Dear Zach:

Strata, A Professional Services Corporation (STRATA) has performed the authorized geotechnical engineering evaluation for the proposed wastewater treatment plant upgrades located at the existing Nampa Waste Water Treatment Plant (WWTP) in Nampa, Idaho. The purpose of our geotechnical engineering evaluation was to explore the subsurface soil within the proposed construction areas and to provide geotechnical engineering recommendations to assist project planning, design and construction. Our services were performed referencing our proposal dated June 20, 2013, and our supplemental proposal dated June 4, 2014.

This report summarizes the results of our field evaluation, laboratory testing, opinions and geotechnical recommendations. Loose silty sand and soft clay and silt underlie the planned building/construction areas. Soil improvements will be required within these soils to enable the planned construction to proceed. Specific geotechnical opinions and recommendations are included for foundations, floor slabs, and earthwork construction. The geotechnical recommendations presented must be read and implemented in their entirety. Individual portions of the report cannot be relied upon without the supporting text of relevant sections.

The success of the proposed construction will depend, in part, on following the report recommendations and good construction practices. We recommend STRATA be retained to provide geotechnical consultation, observation and earthwork testing services during construction to verify our report recommendations are followed. Our experience has been that maintaining continuity with a single geotechnical consultant reduces errors and contributes to overall project success and economy.

We appreciate the opportunity to work with you on this project. Please do not hesitate to contact us if you have any questions or comments.

PROFESSIONAL ENGINEER
REGISTERED
10830
9/8/14
STATE OF IDAHO
MICHAEL G. WOODWORTH

Sincerely,
STRATA, Inc.

Michael G. Woodworth, P.E.
Engineering Manager

Adrian Mascorro, P.E.
Geotechnical Engineer

MGW/AM/nm

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INTRODUCTION

STRATA is pleased to present our authorized geotechnical engineering evaluation for the proposed Waste Water Treatment Plant (WWTP) upgrades located in Nampa, Idaho. The site is located at the existing and currently operational City of Nampa WWTP, located at 340 West Railroad Street. Indian Creek is located immediately north of the WWTP site. A vicinity map showing the location of the site is presented on Plate 1, *Exploration Location Plan*. The purpose of our geotechnical engineering evaluation was to assess the subsurface soil conditions within the proposed construction areas and provide geotechnical opinions, design and construction recommendations with respect to the proposed construction. Our recommendations are based on our field observations and laboratory test results. To provide this evaluation, we conducted the following:

1. Reviewed the current site plan and discussed the project details with Mr. Andy Zimmerman of the City of Nampa (City) and Mr. Matt Gregg and Mr. Zach Dobroth of Brown and Caldwell.

2. Coordinated site access and utility identification with Mr. Matt Gregg and the City WWTP personnel. We also contacted a utility mark out through Idaho Digline prior to field exploration.

3. Subcontracted a drill rig and operators to observe the advancement of 8 hollow-stem auger exploratory borings at the project site to evaluate the subsurface conditions. The soil encountered in the borings were described and classified referencing the ASTM D 2487 and D 2488 *Unified Soil Classification System* (USCS), and the soil profiles were logged by a professional geologist. Standard Penetration Testing (SPT) was performed while obtaining soil sampling via 2.5- and 3-inch, outside-diameter, split-spoon samplers and transported back to our laboratory for testing.

4. Select soil samples were tested for classification and to establish engineering design parameters.

5. Analyzed the field and laboratory data and accomplished engineering analyses to provide geotechnical opinions and recommendations for the following:

    - Site Preparation/Earthwork
        - Site excavations
        - Construction dewatering
        - Wet weather/wet soil construction
        - Subgrade and site preparation
        - Structural fill
        - Geosynthetics
        - Pipe bedding and compaction criteria

- Foundation Design Recommendations
  - Allowable bearing pressures
  - Settlement criteria
  - Frost depth
  - Sliding coefficient
  - Floor slab construction
  - Lateral earth pressures (static and dynamic)
  - Seismic design parameters

- Evaluation of corrosivity/reactivity
  - Discussion of soil corrosivity based on lab testing
  - Recommended Portland cement type for construction

PROJECT UNDERSTANDING

Existing Site Conditions

Asphalt and gravel access is provided throughout the site. The overall site is presently used as a waste water treatment plant for the City of Nampa. The site topography is relatively flat, but slopes gently from the south to the north. Indian Creek is located north of the site.

Proposed Construction

We understand development plans consist of a new Aeration Basin No. 3, a primary effluent pump station, digester and solids handling facility. The aeration basin and primary effluent pump station are referred to as Project Group A, and are planned in the northern portion of the WWTP site. The aeration basin is planned to be approximately 44,000 square feet in area. The aeration basin will be constructed with concrete walls and a concrete slab planned to be approximately 24 feet below existing grade. The pump station is planned to have an approximate 2,000-square-foot footprint, with approximately 900 square feet planned to be 27 feet below existing grade.

The planned solids handling facility will be located in the southwestern portion of the WWTP site, and is referred to as Project Group B. The solids handling building will have 2 levels above grade with no basement. The structure will have an approximate 7,000-square-foot footprint. The basement is planned to be approximately 15 feet below existing grade. The building will be constructed as a concrete masonry unit (CMU) building with concrete slab-on-grade floors.

The planned Primary Digester no. 4 is also located in the southwestern portion of the site and is referred to as Project Group C. The digester will be constructed utilizing reinforced concrete and will be approximately 75 feet in diameter. The digester base is planned to be approximately 10 to 14 feet below existing grade, with the central sump pit extending up to 20 feet below existing grade.

SUBSURFACE EVALUATION PROCEDURES

A professional geologist observed the drilling of 5 exploratory hollow-stem auger borings on July 25, 2013 and 3 supplementary borings in June, 2014. The approximate locations of borings are shown on Plate 1, *Exploration Location Plan*. Borings were advanced from 26.5 to 51.5 feet below existing grades. We obtained select soil samples for laboratory testing and visually classified and described referencing ASTM D 2487 and D 2488, *Unified Soil Classification System* (USCS). We provide an explanation of the USCS in Appendix A, which should be referenced to identify the terms and conditions used throughout this memorandum and on exploration logs, which are also presented in Appendix A.

We observed drilling of exploratory borings utilizing a truck-mounted CME 75 drill rig, with 8-inch, outside-diameter, hollow-stem augers and mud-rotary drilling techniques. We recovered samples within borings at 2.5 to 5 foot intervals, using a 2-inch, outside-diameter, split-spoon sampler with a 140-lb hammer falling 30 inches. Standard Penetration Test (SPT) N_{60} values were recorded for each sample. N_{60} values were obtained by counting the number of hammer blows required to advance the 18-inch sampler from 6 to 18 inches. The blow counts for each 6-inch segment of the sample are presented on individual boring logs. We did not correct the SPT N-values for overburden pressures or dilation effects of the samplers. SPT N-values can provide an indication of the relative density, or consistency of the soil sampled, and is utilized for soil engineering strength and liquefaction analyses. In addition, relatively undisturbed samples of fine-grain soil were obtained at select locations using a 3-inch outside-diameter California Modified ring sampler. The blow counts for the 3-inch sampler were converted to equivalent SPT N-values for a 2-inch outside-diameter split-spoon sampler as shown on the boring logs.

Boring B-1 was installed as a pumping well with 4-inch-diameter PVC casing. The well was installed to an approximate depth of 25 feet below the existing ground surface and included 10 feet of screened casing. The screened interval included Colorado sand as a sand pack to assist well development. The pumping well was developed for approximately 1.5 hours using a small submersible pump. The upper 15 feet of the well was sealed with granular bentonite in general accordance with Idaho Department of Water Resources (IDWR) requirements. This well was permitted through IDWR and can reportedly be lawfully utilized to assist the dewatering program during construction. A monitoring well was installed in B-2 and B-6 and was constructed with 10 feet of screened, 2-inch-diameter PVC casing and 15 to 40 feet of granular bentonite, considering B-2 and B-6, respectively.

At the conclusion of exploration, the borings were backfilled with bentonite chips below the groundwater elevation, followed with soil cuttings level with the existing ground surface and a labeled stake was placed in the boring locations for future identification.

SUBSURFACE EVALUATION

General Site Conditions and Geology

The generalized project geology, based on our current and past fieldwork, and review of geologic references, consists of alluvial silty sand, sandy silt, clay and sand. Although the borings did not encounter basalt, our exploration database shows basalt bedrock is typically encountered between 40 and 50 feet below the existing ground surface. The alluvial soil encountered during exploration is associated with the depositional environment of Indian Creek, which trends to the northwest. The alluvial creek system has the potential for small-scale soil variability in short horizontal and vertical distances. Ancient buried stream channels and flood deposits are likely within the upper 50 feet of the subsurface profile.

Soil and Groundwater Conditions

Subsurface conditions generally consist of previously placed undocumented fill, silty sand and poorly graded sand alluvium, and silt and clay alluvium. We provide more specific discussion of each soil unit encountered below:

- **Undocumented fill** – We observed surficial silty sand, silty gravel, and poorly graded gravel fill at the ground surface to depths of up to 13 feet below existing grades. Fill soil was described as brown, very loose to dense, and moist to saturated. This fill is associated with previous site construction and demolition of previous structures.
- **Silty sand and poorly graded sand and gravel alluvium** – Below fill soil, we observed native silty sand and poorly-graded sand and gravel alluvium. Silty sand and poorly-graded sand and gravel alluvium is generally very loose to medium dense and moist to saturated. Alluvium generally exhibits increased density with depth.
- **Silt and clay alluvium** – Brown, very soft to very stiff, silt and clay alluvium is present interbedded with silty sand and poorly-graded sand and gravel alluvium. The silt and clay generally exhibits increased stiffness with depth, and extended through the termination depth of exploration in the majority of borings and to a depth of 47 feet in Boring B-6.

- **Basalt bedrock** – Below a depth of 47 feet, we encountered weathered basalt bedrock in Boring B-6. Depth to basalt will vary across the site, but typically is encountered between 40 and 50 feet.
- **Groundwater** – Our explorations encountered groundwater at depths of 4 to 12 feet across the site. We anticipate the depth to groundwater is impacted by the flow of Indian Creek to the north of the project site, as well as dewatering operations for the existing plant operations. Previous investigations at the project site have documented artesian pressure in a lower aquifer at the project site. It is our opinion the effect of the lower aquifer may be observed at any depth below or within the clay or silt layer, which typically extends to approximately 25 to greater than 35 feet below the ground surface. However, during our investigation, we did not encounter artesian groundwater conditions.

Specific soil contacts and descriptions are further described on individual boring logs provided as Appendix A to this deliverable, along with a USCS explanation to assist with boring log information.

Aquifer Field Testing

To gain hydrogeologic data to supplement existing data, STRATA accomplished an aquifer pump test within the upper aquifer, utilizing borings B-1 and B-2. A 36 gallon per minute (gpm) submersible pump was utilized in boring B-1 to discharge water from the well. Solinst™ Levelogger pressure transducers were installed in each well to monitor groundwater drawdown during the aquifer test. An electric water level indicator was also utilized to field-check pressure transducers and for groundwater static level measurements and monitoring. Groundwater was discharged to an approved stormwater discharge location. Discharge quantities were monitored using a 5-gallon bucket with measured intervals using a stopwatch timer.

The test was initiated on September 13, 2013. The 36 gpm pump discharge was throttled to approximately 1.6 gpm and was set at a depth of 24 feet below the existing ground surface. The test was performed for approximately 8 hours. Drawdown was measured in the pumping well, B-1, and monitoring well B-2. The groundwater level in the pumping well experienced drawdown of approximately 9.5 feet, with no measurable drawdown in the monitoring well.

Laboratory Testing

We accomplished laboratory testing referencing ASTM International test procedures. Laboratory tests included the following:

- Grain size analyses (minus No. 200 wash)
- In-situ moisture

- In-situ unit weight
- Atterberg limits
- Consolidation testing
- Chemical reactivity testing (including pH, sulfate and resistivity)

Laboratory test results are presented on the individual exploration logs and are also provided in Appendix B.

HYDROGEOLOGY

We have previously discussed the hydrogeologic conditions at the site in the *Soil and Groundwater Conditions* section. Groundwater from the upper aquifer can generally be encountered from 4 to 12 feet below the existing ground surface.

Aquifer Testing and Analyses

Aquifer test data from the upper aquifer were used to develop time-drawdown curves for the observation well and the pumping well. Well construction, pumping rates, subsurface aquifer geometry, and well spacing were documented to facilitate hydrogeologic analyses. The Cooper-Jacob (1946) method was used to estimate transmissivity of the upper aquifer. The short duration of the aquifer test did not allow for valid estimates of specific yield (storativity). Transmissivity is defined as permeability or soil hydraulic conductivity times the saturated thickness of the aquifer. Transmissivity of unconfined aquifers will vary as groundwater levels are decreased. Based on the transmissivity estimated from aquifer testing and measured saturated thickness, a range of hydraulic conductivity values were back-calculated for each analysis. Hydraulic conductivity is a measure of a soil's ability to permit water flow under a hydraulic gradient. Hydraulic conductivity is a vital parameter in construction dewatering analyses. STRATA also utilized the subsurface geometry and soil conditions to calibrate our model. Known boring locations, pumping rates and knowledge of well construction were utilized to refine estimates of hydraulic conductivity of the upper aquifer.

Our analyses indicate the hydraulic conductivity of the upper aquifer for preliminary design will be 1.5 x 10^{-5} to 8 x 10^{-5} feet per second. The above hydrogeologic parameters should not be solely relied upon by the contractor. The dewatering system designer must evaluate the hydraulic conductivity and dewatering characteristics of both aquifer systems to facilitate a successful dewatering design. STRATA did not provide aquifer test results due to the potential for misinterpretation of the data. The raw data is available for review upon request, contingent upon STRATA's participation in data interpretation.

GEOTECHNICAL OPINIONS AND RECOMMENDATIONS

General

Our geotechnical opinions and recommendations are presented in the following sections to assist project planning, design, and construction. Our recommendations are based on the results of our field evaluation, laboratory testing, and our understanding of the proposed construction. These opinions and recommendations reflect our conversations on information provided to us by Brown and Caldwell and the City. If design plans change, such as loading conditions, or the building configuration, STRATA should be notified to review our report recommendations and make necessary modifications.

The subsurface conditions may vary from what we observed during exploration across the site. These changes in conditions may not be apparent until construction. If the subsurface conditions change from those observed during exploration, the construction schedule, plans, and costs may change.

Subsurface Constraints and Opportunities

- **Shallow Groundwater –** As noted above, we encountered groundwater at depths of 4 to 12 feet during exploration. Based on the planned depth of construction, groundwater dewatering will be required during construction. Additionally, the design of individual structures must account for the buoyant pressure as a result of planned construction extending below groundwater.
- **Reusability of on-site soils** – The silty sand, poorly graded sand and gravel, and silty gravel soil may be reused as structural fill below slab foundations provided it is moisture conditioned and recompacted to structural fill criteria as presented in the *Structural Fill* section below. Additionally, silt soil may be re-used as structural fill, but achieving near optimum moisture conditions for existing moist to saturated silt will be difficult. Clay soil should not be used as structural fill in any case.

Site Preparation/Earthwork

Site Excavations

We anticipate soil within the planned construction areas may be excavated using conventional excavation techniques. We recommend earthwork contractors closely review subsurface conditions presented in this report and select appropriate excavation and shoring methods (if required). Excavations and/or support structures for excavations deeper than 20 feet may be required. The excavation and slope stability design, design calculations and a report should be accomplished by a licensed professional engineer in accordance with OSHA requirements.

Site excavations must be sloped in accordance with the *Occupational Safety and Health Administration* (OSHA) regulations and local codes. The site soil generally consists of loose and medium dense sandy soil and is classified as "C" type soil according to OSHA requirements and therefore, we recommend provisions be made to allow temporary excavations up to 20 feet be sloped back to at least 1.5H:1V. It is our opinion that temporary side slopes constructed at 1.5H:1V or flatter for C soil will be stable from deep soil seated failure, provided the site has been dewatered to a minimum of 2 feet below the desired subgrade, and that the dewatering extends a minimum of 20 feet beyond the crest of the excavation. However, the contractor will ultimately be responsible for excavation stability and site safety as soil and groundwater conditions can vary. Isolated, local flattening of slopes may be required due to localized surficial soil sloughing. Surcharges must not be allowed within a horizontal distance equal to one-half the excavation depth. Construction vibrations can cause excavations to slough or cave. Ultimately, the contractor is solely responsible for site safety and excavation configurations.

Temporary trench excavations less than 5 feet may be constructed with vertical sides, if adequately dewatered. Deeper trenches will require side support in the form of steel trench boxes, steel or timber shoring, and other means of trench wall protection. If trench boxes or other means of temporary support of pipe excavations are utilized, the trench box or shoring should be of sufficient width to be able to install foundations, piping, pipe bedding, and provide safe and productive working conditions. We recommend a licensed engineer design any shoring plans required for excavation.

Minor sloughing of the soil could require OSHA approved maintenance and protection for workers and equipment. Localized perched groundwater, subsequent to dewatering, may cause local flowing soil conditions and excavation instability. Rain and other water sources will exacerbate the potential for caving and sloughing of the soils. Excavation equipment and other construction procedures must be selected to avoid inducing dynamic loading, which could increase soil pore water pressure causing local disturbance, which may lead to both side slope and foundation soil instability.

Deep excavations may utilize temporary shoring or a combination of shoring and an open excavation. Shoring will reduce the excavation size. Shoring alternatives include soldier pile and lagging or sheet piling. Both shoring systems may require soil anchors to maintain sheet pile and soldier pile stability. Design for shoring should use the lateral earth pressure values recommended in this report and should be designed by a licensed engineer qualified, through past project experience, to design such systems. Additionally, shoring design must consider the influence of Indian Creek and the potential for hydrostatic pressure to act upon the shoring system.

Maintaining dewatered conditions at the excavations is imperative for the selected temporary excavation system to perform as designed. This is particularly important for shoring systems if they are designed assuming dewatered lateral earth pressure values. If the shoring design assumes dewatered conditions, we recommend that the contractor have sufficient back-up pumps that can be installed quickly should a pump(s) fail.

Construction Dewatering

We observed groundwater at approximately 4 to 12 feet below existing grades during exploration. However, seasonal groundwater levels will vary. Therefore, dependent on groundwater elevation at time of construction, construction dewatering will be required for foundation soil improvement excavations. Groundwater levels must be maintained a minimum of 2 feet below the proposed construction excavations. Excavations must be carefully planned, allowing for groundwater collection points and utilizing conventional sumps and pumps to remove groundwater seepage, nuisance water seeps, or precipitation.

A specific dewatering plan must be developed by the contractor based on the location and configuration of site improvements and recommendations from the contractor's retained geotechnical design professional. The contractor must evaluate the site conditions, potential dewatering options, and considerations relative to their dewatering design and equipment, and construction approach. The contractor should submit a sealed engineering dewatering plan to the design team prior to initiation of construction. We recommend review of the dewatering plan to verify it meets the intent of the project performance specifications.

It is our opinion site dewatering is possible, assuming a well-planned, practical approach is implemented by the contractor. Specific and detailed recommendations for the dewatering plan and/or specific dewatering characteristics such as pump requirements, contractor capabilities (experience) and other factors are not provided as it is beyond the scope of this deliverable. Therefore, general dewatering considerations presented herein are not provided as specific hydrogeologic recommendations for final construction dewatering planning or design.

We recommend the contractor develop the dewatering plan as part of their work scope. The contractor should be experienced in construction dewatering. STRATA should review the dewatering plan and provide comment as appropriate. Allowing us to review the dewatering plan may reduce the potential for construction delays, additional dewatering costs, or excavation instability associated with an inadequate site dewatering plan and/or misinterpretation of reported data.

The following sections present general concepts, or preliminary options, for site dewatering to assist the contractor in gaining understanding of the hydrogeologic conditions at the site for planning and design of construction dewatering. The contractor's specific dewatering plan should consider the potential for seasonal fluctuation in precipitation, irrigation, infiltration, and the impact of Indian Creek. Variations in subsurface geology, depth of planned construction, precipitation, infiltration, irrigation in the area, and variations in the existing groundwater gradient will affect dewatering results. Finally, we expect the methods implemented to dewater the site will be a dynamic process, based on actual site and hydrogeologic conditions encountered during construction.

Several methods of dewatering are outlined below, based on our understanding of successful dewatering approaches for similar structures in similar conditions. The following discussion of dewatering options is intended to provide information for the dewatering system design professional to aid in their evaluation and design of the contractor's dewatering plan. These options are not to be used by the contractor as an engineered dewatering plan for construction.

Trench Drain Option

One possible method to dewater the upper aquifer within the planned excavation area is a gravity trench drain and sump pit system. The trench drain system must be constructed to the top of the clay layer anticipated at approximately 25 feet below grade. An appropriately sized, perforated pipe could be placed at the base of each trench and sloped to several sump pits, where the groundwater could be pumped to an approved discharge location. We recommend trench drains completely surround the area to be dewatered and be backfilled with drain rock. We recommend the perforated pipes be completely surrounded with free-draining material. The pipe should not be wrapped with geotextile fabric, which may clog with fines and impede pumping rates, but non-woven geotextile fabric should be placed surrounding the drain rock to reduce piping or soil migration into the drain rock.

We anticipate pumping volumes may be approximately 0.7 to 0.9 gallons per minute per foot of trench at steady-state conditions. Depending on trench construction details, we anticipate this will result in dewatering rates of approximately 650 to 950 gallons per minute to achieve dewatering for Aeration Basin No. 3. However, total pumping rates will be influenced by the height of the static water table above this clay layer and interaction between the aquifer and Indian Creek.

Near-vertical excavations constructed below the groundwater table will not remain stable. Therefore, trench boxes or other shoring must be used and trenches must be backfilled with free-draining material to keep the trench stable and allow dewatering.

Well Point Option

Closely spaced well points are another option to help dewater the aquifer to allow construction to occur. However, considering the elevation of the proposed excavations relative to the top of clay soil, we consider well points to be feasible as a dewatering method for the smaller excavations only.

Alternative Options

Other methods of dewatering are possible, including localized dewatering within an enclosed, shored excavation. Appropriately designed sheet pile or soldier pile shoring walls may be effectively dewatered from within the closed shoring system. Dewatering can occur as excavation occurs to attain the subgrade elevation. One benefit to a sheet pile excavation is that sheet piles driven into the clay can effectively cut off the upper aquifer from reaching the working area. Using this option will require a sheet pile system that is designed by a professional engineer in the state of Idaho for hydrostatic conditions. Further, all sheet pile connections must be tight to reduce water infiltration through the joints of the sheet piles.

General Well and Pump Considerations

Establishing a successful dewatering program will be contingent upon individual spacing, pumping rates and well construction. Well construction has the potential to limit pumping rates. Further, water production may be reduced as groundwater is drawn down and transmissivity decreases. It is our opinion that each well may need to be instrumented with water level indicators to shut down the pump as the water level approaches the pump intake. This level generally should be set a few feet above the actual pump intake. It should be possible to maintain relatively constant water levels by setting the pumps to turn on and off as necessary in combination with pumping rate adjustments. Pumps should be active as much as possible to maintain as much drawdown at the well as possible without causing the pump to burn up. Pump cycles should be set accordingly, such that the pumps are pumping for a longer period of time than they are shut down. If the well is shut down for too long, groundwater levels will recover and the regional groundwater below the area to be dewatered will not decrease, only fluctuate. We recommend the contractor establish a groundwater discharge location that does not conduct water to the site groundwater system and meets regulatory agency requirements.

Wet Weather / Wet Soil Construction

We strongly recommend earthwork construction take place during dry weather conditions. Subgrade soil consists of very loose to medium dense silty sand and very soft to stiff clay and silt soil. Silty and clayey soil is susceptible to pumping or rutting from heavy loads such as rubber-tired equipment or vehicles when the soil is above optimum moisture content. Earthwork should not be performed immediately after rainfall or until soil can dry sufficiently to allow construction traffic without disturbing the subgrade. During and after achieving subgrade elevation, the contractor must take precautions to protect the subgrade from becoming disturbed or saturated. We recommend the contractor reduce exposure to precipitation and water within the excavation. The contractor should:

- Grade subgrades to aggressively direct surface water away from construction areas that could be adversely affected by infiltration.
- Remove exposed subgrade soil that becomes soft or begins to pump to firm soil and replace it with structural fill as described in this report for over-excavations.
- Never attempt structural fill placement during or immediately following a significant precipitation event.
- Never allow subgrades to freeze or become saturated prior to fill placement.

The final subgrade conditions and careful construction procedures are critical to the long-term project performance. We recommend earthwork specifications specifically identify the contractor's responsibility to protect and maintain prepared subgrades. It may improve project economy to retain STRATA to observe excavation activities to identify techniques or construction activities that may be attributing to unstable subgrades and contributing to the need for over-excavations.

Subgrade and Site Preparation

Site stripping and excavation can commence and continue to 2 feet above the static groundwater table as dewatering proceeds. Excavation to achieve the subgrade elevations should not extend into non-dewatered soil. Once the subgrade elevation has been achieved, the groundwater must be maintained 2 feet below the subgrade during construction.

Dewatering the deep excavations must occur to render the subsurface soil conditions sufficiently dry to complete the required earthwork to achieve the foundation elevations anticipated to be approximately 15 to 27 feet below existing grades. Disturbing the native soil may result in inconsistent subgrade support for site improvements including the foundations, concrete slabs, piping and structural fill. Soil disturbance at the foundation subgrade elevation will occur if the site is not adequately dewatered prior to and during site earthwork and if construction equipment that is not suitable for working in moist to wet, dewatered soil conditions is utilized.

Excavations can commence and continue to approximately 2 to 3 feet above the static groundwater table as dewatering proceeds. Excavation to achieve the subgrade should not extend into saturated soil. Thus, a minimum of 2 feet of dewatered soil should be maintained above the static groundwater level during dewatering and excavation. Once the subgrade elevation has been achieved, the dewatered condition must be maintained at least 2 feet below the subgrade elevation.

The upper aquifer will cause the silty sand to maintain a near saturated condition as a result of capillary rise. Due to this condition, equipment loads, sand boils and seepage, the potential for the foundation soil at the base of the excavation to pump or rut must be considered. Excavation should be terminated immediately if water-related soil disturbances are observed, and STRATA advised of the condition(s) in order for us to provide the necessary consultation. We anticipate excavation within the silty and clay soil will be necessary to achieve subgrade elevation for the aeration basin and other structures. The potential for water-related foundation soil disturbance is greatest at this point in the excavation and the contractor may need to undertake additional localized dewatering measures and revise their construction approach should this condition occur.

Excavation of soil at subgrade elevation should be achieved using smooth blade, tracked equipment to reduce the potential for soil disturbance. Soil that is disturbed during subgrade preparation should be excavated to firm soil and replaced with granular structural fill or drain rock.

We recommend that the foundation subgrades for all structures be verified by STRATA such that the subgrade is firm and does not yield or pump, have ruts, or have other conditions that could affect performance of structures. Soil that is disturbed by subgrade preparation should be excavated to native undisturbed soil, and the excavation backfilled with drain rock. Again, this can only be accomplished if groundwater and/or seepage are controlled locally. The on-site soil has significant potential to migrate into drain rock. Therefore, a woven geotextile must be placed over the prepared subgrade prior to placement of structural fill.

Structural Fill

All fill placed for support of foundations, floor slabs, flatwork and pavement areas must be placed as structural fill. Project structural fill products are described in Table 1 below. The on-site fill sandy silt with gravel and silty sand may be reused as structural fill, provided it meets the material specifications below and the on-site contractor is familiar with earthwork construction practices utilizing moisture sensitive silty soils.

Table 1. Structural Fill Specifications and Allowable Use

Structural Fill Product	Allowable Use	Material Specifications
General Structural Fill	General site grading, utilities, slab area over-excavation, and fill placement	• Soil must be classified as silt, sand, or gravel (GP, GM, GW, GC, SP, SM, SW, SC or ML) according to the USCS. • Soil may not contain particles larger than 6-inches in median diameter. • Soil must consist of inert earth materials with less than 3% organics or other deleterious substances (wood, metal, plastic, waste, etc).
Granular Structural Fill (Granular Subbase)	Over-excavations, foundation wall backfill, temporary haul roads, granular subbase, general structural fill	• Soil must be classified as sand or gravel (GP, GW, SP, or SW) according to the USCS. • Less than 10% passing No. 200 sieve. • Soil may not contain particles larger than 6 inches in diameter. • Soil meeting the latest requirements in ISPWC –*Section 801 Uncrushed Aggregate for Granular Subbase.*
Aggregate Base Course	Foundation and slab support, general structural fill	• Soil must meet granular structural fill requirements. • Soil meeting the latest requirements in ISPWC – *Section 802 ¾-inch-minus Crushed Aggregate.*
Drain Rock	Over-excavation Foundation Support	• Drain Rock shall meet requirements stated in the latest edition of the Idaho Standard for Public Works Construction (ISPWC), *Section 801 – Aggregate Subbase.*
Pipe Bedding	Utility trench pipe bedding	• Soil meeting requirements stated in ISPWC specifications for pipe bedding.
Unsatisfactory Soil	No structural applications, landscaping per landscape engineer	• Soil classified as CL, CH, MH, OH, OL or PT may not be used at the project site for structural fill. • Soil not maintaining moisture contents within 3 percent of optimum. • Any soil containing more than 3 percent organics by weight or other deleterious substances (wood, metal, plastic, waste, etc) is unsatisfactory soil.

All structural fill placed below slab areas for soil improvements or beneath foundations must be compacted to a minimum of 95 percent of the maximum dry density of the soil referencing ASTM D 1557 (Modified Proctor). Fill placed outside any building envelope, flatwork or road section can be placed as non-structural fill (i.e. landscape fill) providing there are no structures (flatwork, signs, etc.) planned directly above the landscape fill. We recommend landscape fill be compacted to a minimum of 85 percent of the maximum dry density of the soil according to ASTM D 1557 (Modified Proctor).

Any structural fill products must be moisture conditioned to near-optimum moisture content and placed in maximum 12-inch-thick, loose lifts. The above assumes large compaction equipment with drum energy of at least 10 tons or greater is used to attempt compaction. If smaller or lighter compaction equipment is provided, the lift thickness may have to be reduced to meet the compaction requirements presented herein.

Geosynthetics

We recommend geosynthetic fabrics be used to improve subgrade support when constructing on soft or wet soil such as the native on-site silt encountered at the site. Where required, apply geosynthetics directly on approved subgrade, free of wrinkles, and over-lapped at least 12 inches. Woven geosynthetic fabrics for subgrade stabilization and soil improvements shall have the following minimum properties of 700 pounds (CBR Puncture, ASTM D 6241), 100 pounds (Puncture Strength ASTM D 4833) and 200 pounds (Grab Tensile Strength ASTM D 4632) such as a Contech C200. STRATA must be consulted prior to using geosynthetics for subgrade stabilization. Further, we recommend contractors carefully review subsurface conditions prior to bidding and recommend the design team include a unit price for woven geosynthetics for the earthwork portion of the project.

Pipe Bedding and Compaction Criteria

Pipe bedding should be Type I and should extend from 6 inches below the bottom of the pipe to at least 6 inches above the crown of the pipe. All saturated, loose, or disturbed soil should be removed from the bottom of the trench before placing the bedding. Bedding of the trench and around the pipe should be accomplished in accordance with the latest edition of the *ISPWC Section 305, Pipe Bedding*. Bedding, if sufficiently coarse, may be placed and compacted dry. Alternatively, the bedding with sufficient fines can have water added to produce a uniform near optimum moisture content mixture. Bedding should be placed in maximum 6-inch, loose lifts prior to compaction. Testing of the bedding should occur for every 18 inches or less of bedding materials placed for every 250 or less lineal feet. In areas where loose or soft soil is present at pipe subgrade elevation, compaction or over-excavation and compaction testing of the pipe subgrade should occur prior to placement of bedding. Refer to the *Site and*

Subgrade Preparation section of this report for additional information regarding subgrade compaction and over-excavation.

Avoiding impact tampers or other large compaction equipment directly above the pipe, or preferably not until at least 12 inches of backfill has been placed above the pipe bedding, is recommended to reduce the potential for local deformation and/or pipe damage. Compaction of the pipe bedding along the side of the pipe and below the spring-line should not cause the pipe to lift off of grade, but if uplift movement occurs, adjustments to the type of equipment that is used in the compaction procedures should be reviewed and changed to maintain the pipeline and grade.

It is our opinion that one of the most important aspects of pipe performance is to establish a well-performing, uniformly compacted bedding material between the spring-line and the pipe invert. However, compaction of the soil around this area, and performing compaction tests to verify the compaction, are very difficult. We recommend that a performance compaction criteria are established which includes continuous visual verification that the pipe bedding construction, from pipe spring-line to invert, is being accomplished with the approved backfill compaction equipment.

It is our opinion and recommendation that the verification of the earthwork placement and compaction be undertaken by the owner's representative. The successful bidder has the option of utilizing the results of these observations and test data, but should not rely on these data to fulfill their contractual obligations. Therefore, it should be their responsibility to hire the necessary qualified independent testing group to verify that their contractual quality assurance has been achieved.

Foundations

General

We anticipate the proposed below-grade structures will be constructed on reinforced concrete mat foundations. At-grade portions of the planned construction, such as the solids handling building, are planned to be constructed with conventional shallow foundations. With this understanding, we provide specific foundation design recommendations for each of the planned structures in Table – below.

Structure	Foundation type	Foundation subgrade and design criteria
Solids Handling Facility	Shallow foundations	*Subgrade:* • 12 granular structural fill *Bearing pressure:* • 2,000 pounds per square foot *Settlement:* • Total settlement – 1 inch • Differential settlement – ½ inch
Aeration Basin	Mat foundation	*Subgrade:* • 6 inches aggregate base course • 18 inches drain rock *Bearing pressure:* • 2,000 pounds per square foot *Settlement:* • Total settlement – 1 inch • Differential settlement – ½ inch
Digester No. 4	Mat foundation	*Subgrade:* • 6 inches aggregate base course • 30 inches drain rock • Woven geotextile *Bearing pressure:* • 3,000 pounds per square foot *Settlement:* • 2 Inches center • 1 Inch perimeter

In addition to the above recommendations, we recommend that all foundations or slabs be designed and constructed in accordance with the following general recommendations:

1. All shallow foundations should be extended a minimum of 24 inches below final, exterior grade, or placed on aggregate base course extending 24 inches below final grade to mitigate the effects of frost penetration.

2. All foundations should be designed in accordance with requirements outlined in the 2012 International Building Code (IBC).

3. All loose or frozen soil or water at the base of foundation excavations should be removed, and the subgrade over-excavated with a smooth blade bucket to undisturbed soil. Disturbed native soil at footing subgrade can be recompacted to structural fill criteria.

4. A one-third increase in allowable bearing may be utilized for short-term loading from seismic or wind induced loads.

5. We recommend foundations subjected to uplift loads be designed using a buoyant soil backfill unit weight of 68 pounds per cubic foot (pcf) and a concrete unit weight of 150 pcf. We provide these design values assuming backfill meeting the structural fill requirements outlined above and normal weight structural concrete, respectively.

6. A sliding coefficient of 0.40 may be utilized for cast-in-place foundations bearing on aggregate base course.

7. Mat foundations may be designed using a modulus of subgrade reaction of 200 pounds per cubic inch (pci) for a 1-foot-square mat (k_{s1}). This value assumes mat foundations are underlain by a minimum of 6 inches of aggregate base course structural fill and 18 inches of drain rock as described above.

We recommend STRATA be retained to observe the foundation system installation including reviewing the subgrade and compaction effort prior to placing concrete forms or concrete. Reviewing the subgrade and verifying a consistent, dense subgrade exists below final foundation bearing surfaces helps confirm our allowable bearing pressures and settlement estimates and is an important part of the geotechnical design process.

Concrete Slab-on-Grade Floors

Once subgrade preparation beneath the concrete slabs is accomplished per the *Site Preparation/Earthwork* section of this report, we recommend concrete slab-on-grade floors be underlain by at least 6 inches of ¾-inch-minus, aggregate base course to provide a leveling course and moisture protection for the slab. The base course should be placed over the prepared subgrade and compacted to structural fill requirements. The base course and vapor barriers (if utilized) should be installed after the majority of under-slab plumbing and utilities are completed. Floor slabs should be designed for the anticipated use and equipment or storage loading conditions. Based on correlation to our field and laboratory test results, in conjunction with the placement of recompacted soil improvement layer recommended in floor slab areas, we recommend a modulus of subgrade reaction (k) of 200 pounds per cubic inch (pci) be used for concrete floor slab design. This modulus is based on a silty soil subgrade plus 6 inches of compacted ¾-inch-minus aggregate base course beneath the floor slab.

Interior floor slabs may be susceptible to moisture migration caused by capillary action and vapor pressure. Floor coverings such as tile, vinyl, or other “impervious coatings” may exist within the retail area and a vapor retarder is strongly encouraged in these areas. In shop areas where no floor coverings are expected, a vapor retarder may not be necessary. Where utilized, vapor retarders must consist of a thick, 15-mil, puncture-resistant sheeting consistent with American Concrete Institute (ACI) Section 302.2R-06 specifications. An example of a common vapor retarder is Stego Wrap™, a 15-mil vapor retarder.

The specific location of vapor retarders has been widely discussed in the architectural, structural, construction and geotechnical engineering community, and differing opinions exist. However, current recommendations by the ACI recommend placement of a vapor retarder directly below the concrete slab. However, ultimately, the location of the vapor retarder (if a vapor retarder is specified) should be carefully considered by the owner and architect. Studies have shown that decreased concrete water-cement ratios, higher strength concrete, and good construction finishing practices significantly decrease any negative impacts associated with both of the above options for vapor retarder locations.

Installation of form stakes or other sub-slab penetrations must never be allowed to puncture the vapor retarder. Manufacturer recommendations for proper sealing of slab-to-wall connections, plumbing or other penetrations must be strictly followed. Although these recommendations are used, water vapor migration through the concrete floor slab is still possible. Floor covering must be selected accordingly and manufacturer's recommendations strictly followed.

Below-Grade Walls

As discussed above, we understand the base of the planned construction will vary from approximately 15 to 27 feet below existing grade. As such, below-grade walls must account for lateral earth pressures, any possible equipment surcharges and the surcharge from traffic loading. We recommend lateral earth pressures for temporary shoring be estimated using equivalent fluid pressures (EFP) from the following Table 2 assuming wall drainage will be provided. We have provided estimates for EFP utilizing field information, meeting the requirements in Table 3 below.

Table 3. Static Equivalent Fluid Pressures (dewatered)

Lateral Earth Pressure Case	Equivalent Fluid Pressure (EFP)**
At rest case (no wall movement)	60 pcf* (unsaturated)
Active case (wall movement away from soil mass)	35 pcf* (unsaturated)
Passive case (wall movement toward soil mass)	420 pcf* (unsaturated)

*pcf = pounds per cubic foot

** Does not include buoyant unit weight of the water

We recommend design of the below grade walls subject to hydrostatic conditions use the equivalent fluid pressures from Table 4 below.

Table 4 - Static Equivalent Fluid Pressures (submerged conditions)

Lateral Earth Pressure Case	Equivalent Fluid Pressure (EFP)
At rest case – no movement of structure	90 pcf*
Active case – lateral movement of structure	80 pcf*
Passive case**	280 pcf**

*Based on saturated unit weight

**Has been adjusted for 1/2-inch of lateral deflection

For walls that cannot tolerate movement, we recommend they be designed utilizing at-rest fluid pressures. Lateral surcharge pressures due to vehicle traffic, equipment and storage loads, etc. have not been included in the above lateral earth pressure recommendation. A lateral earth pressure coefficient of 0.45 acting over the entire retaining wall should be used to estimate lateral surcharge loads from equipment storage loads, etc. located behind and above walls. Compaction of backfill within 5 feet of the retaining wall should be performed only with vibratory plates or walk behind smooth-drum vibratory rollers to lessen potential surcharge loading of the walls during compaction.

Dynamic lateral earth pressures are a function of several factors including the presence of groundwater, magnitude of ground shaking, soil strength and soil permeability. Dynamic lateral earth pressures are *additive* to the above static lateral earth pressures, but act as an inverted triangle. *Hydrodynamic* forces also need to be accounted for in wall design and occur in two primary situations; 1) water "sloshing" back and forth between the soil matrix and exerting inertial forces, and 2) water being mobilized with the soil matrix as it is laterally forced against the structure. The former situation occurs in higher permeability soil, while the latter situation occurs in lower permeability soil where the soil has a tendency to experience excess pore water pressures. The degree of excess pore water pressure will impact the degree that water is taken into account for the dynamic lateral earth pressure. If complete excess pore water pressure occurs (i.e. liquefaction) the soil will act as a dense liquid and the EFP will approach the saturated unit weight of the soil during a seismic event. Hydrodynamic forces are discussed below.

The design of below-grade walls should account for dynamic load influences. These dynamic EFPs (excluding hydrodynamic EFPs) should be added to the above static EFPs, but as an inverted triangle distribution. Hydrodynamic EFPs should be added to the hydrostatic forces, acting in the traditional triangular pressure distribution. Table 5 below presents equivalent fluid pressures during dynamic loading (excludes static loads) for the saturated soil.

The seismic component of pressure is assumed to have its resultant acting at 0.6 times the wall height measured from the base of the wall.

Table 5. Mononobe-Okabe Dynamic Equivalent Fluid Pressures (submerged conditions)

Coulomb Lateral Earth Pressure Case	Equivalent Fluid Pressure (EFP)
At rest case (no wall movement)	+16 pcf (submerged[1])
Active case (wall movement away from soil mass)	+7 pcf (submerged[1])
Passive case[2] (wall movement toward soil mass)	-70 pcf (submerged [1,3])
Hydrodynamic EFP[4] ($EFP_{hydrodynamic}$)	+8 pcf[5]

1 – EFP includes the buoyant soil unit weight and excludes the unit weight of water.

2 – Passive resistance has been provided for ½-inch of lateral movement.

3 – Passive resistance should be reduced by 75 pcf acting as an inverted triangle against the wall.

4 – Additive to hydrostatic fluid pressure using traditional triangular pressure distribution.

5 – Hydrodynamic EFP is specific to Nampa, Idaho, soil permeability and other site specific factors.

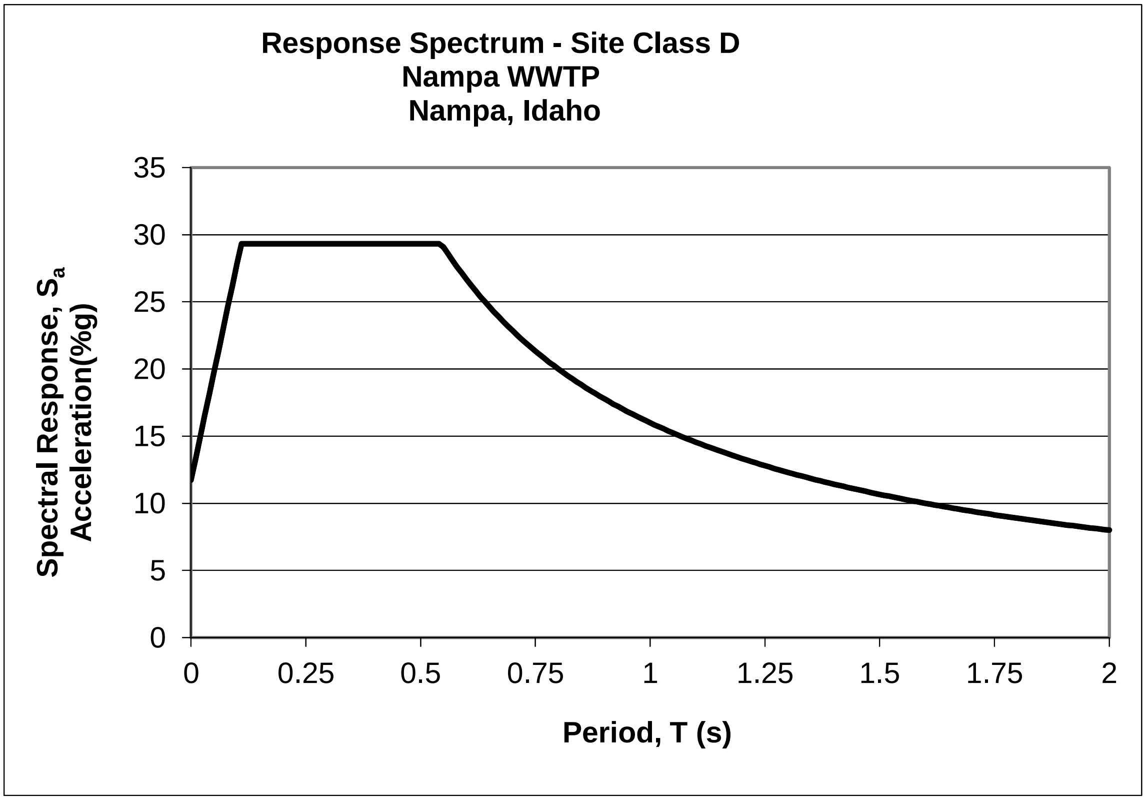
Care must be taken in the use of heavy equipment near the face of walls (in a zone extending 5 feet back from the wall) to avoid creating an undesirable degree of over-compaction or lateral wall loading from the soil immediately along the walls and imposing high stresses on the walls. Below-grade walls should be backfilled as described in the *Structural Fill* section of this report.

Seismic Design Criteria

We understand the 2012 International Building Code (IBC) may be utilized for project structural design. STRATA utilized site soil and geologic data and the project location to establish earthquake loading criteria at the site referencing the 2012 IBC. Based on the results from exploration, and our review of well logs in the area and our interpretation of the IBC, we recommend a Site Class D be utilized as a basis for structural seismic design for the project.

The Maximum Considered Earthquake (MCE) maps from the 2012 IBC were referenced to develop the site response spectrum for Site Class D. The IBC interpreted response spectrum is presented in Figure 1, below.

Figure 1. Design Response Spectrum for the Nampa WWTP.



This response spectrum assumes a 5 percent critical damping ratio in accordance with the IBC. A site-specific study was not performed. Structural design may use the spectral response at period T=0 for peak ground acceleration at the site. The design site-specifics are located in Table 6 below.

Table 6. Seismic Design Parameters

S_s	0.279
S_1	0.100
F_a	1.577
F_v	2.400
S_{DS}	0.293
S_{D1}	0.159

Liquefaction Evaluation

As discussed in this report's *Subsurface Conditions* section, subsurface conditions encountered in the borings generally consist of saturated, loose to medium dense silty sand and poorly graded sand and gravel overlying firm to very stiff silt and lean clay. These soils, particularly the saturated granular alluvium, are susceptible to liquefaction, and we based our liquefaction analyses on engineering parameters of the alluvium.

In order to accomplish our liquefaction triggering analysis, we corrected values obtained during SPT testing according to a procedure outlined in the *1997 National Center for Earthquake Engineering Research (NCEER)* workshop summary report. We also used data from the 2012 IBC to develop anticipated peak ground accelerations. The Cyclic Stress Ratio (CSR), which is defined as a measure of force applied to the soil during seismic loading, can be used to perform the triggering analyses. We calculated the site-specific CSR using the peak ground accelerations as mentioned above, and the procedure developed at the NCEER workshop. The CSR was compared to the Cyclic Resistance Ratio, which is a function of the soil type, density and overburden, to evaluate the potential for liquefaction to be triggered at the site.

We used a peak ground acceleration of 0.117g and an earthquake magnitude of 6.5 for analyses. The analysis indicated the expected factor of safety for resistance to liquefaction is slightly less than 1.0 for the lowest density zones of saturated, coarse grained alluvial soil. Based on this analysis, in our opinion, the likelihood of liquefaction occurring at the project site during a significant seismic event is moderate. However, based on the isolated zones of soil which is susceptible to liquefaction, we anticipate the settlement associated with liquefaction will be limited to approximately 1 inch or less.

Concrete and Corrosivity

STRATA accomplished laboratory soil resistivity, soluble sulfate and pH tests on native and soil encountered during our exploration. Based on resistivity values of approximately 2,564 Ohm-cm, the near surface soil encountered within the upper 15 to 20 feet of the soil profile is classified as moderately corrosive to unprotected steel (Roberge, 2000). Therefore, we recommend all foundations have appropriate corrosion protection and all code minimum steel reinforcement clearances be adhered to.

In addition to soil resistivity and corrosion potential, sulfate concentrations in existing soil units is important in determining cement type for use in the project. Laboratory tests result in water soluble sulfate concentrations of 343 parts per million (ppm). For the soil tested, based on the 2012 International Building Code (IBC) and the sulfate values presented in Appendix B, sulfate exposure to concrete is negligible and we recommend the use of ASTM C 150 Type II cement.

Surface and Subsurface Drainage

Site grading, including all sidewalks and landscaped area grading, should slope a minimum of 5 percent away from the proposed structures within 10 feet to help prevent ponding and to direct surface runoff away from the structure. All runoff from downspouts, roof areas, sidewalk areas, landscaped areas, and other large volumes of stormwater should be directed and maintained away from the structure and not be allowed to infiltrate the soil beneath the building area, sidewalks or footings. We recommend pavement areas slope away from the building to an approved stormwater disposal system.

ADDITIONAL RECOMMENDED SERVICES

Geotechnical Consultation/Review of Plans and Specifications

We understand STRATA will provide geotechnical consultation with the design team during the development of construction documents. STRATA will review earthwork and geotechnical-related portions of the civil and structural plans and specifications prior to construction bidding.

Construction Observation and Testing

We recommend STRATA be retained to observe all site preparation/earthwork, slab and foundation subgrades, and bearing surfaces. Additionally, we recommend that we observe the subgrade preparation to verify site stripping and excavation has been accomplished to the recommended bearing soil, that all soft or unsuitable soil has been removed as described above, and testing of recompacted structural fill. Geotechnical continuity is an important part of the geotechnical design process to assist the design team in identifying potential subsurface condition changes and other unanticipated issues. STRATA can also provide construction material testing and special inspection for reinforced concrete, asphalt, masonry, wood framing and steel. If STRATA is not retained to provide the recommended services, we cannot be responsible for soil engineering-related construction errors or omissions.

EVALUATION LIMITATIONS

This report has been prepared to assist project planning design and construction of the upgrades to be constructed at the existing City of Nampa WWTP. Our geotechnical findings and opinions have been developed based on the authorized subsurface exploration and laboratory testing, as well as our understanding of the project at this time. Our geotechnical design recommendations are specific to the planned design and infrastructure construction and should not be extrapolated to other future site developments without allowing adequate geotechnical consultation by STRATA.

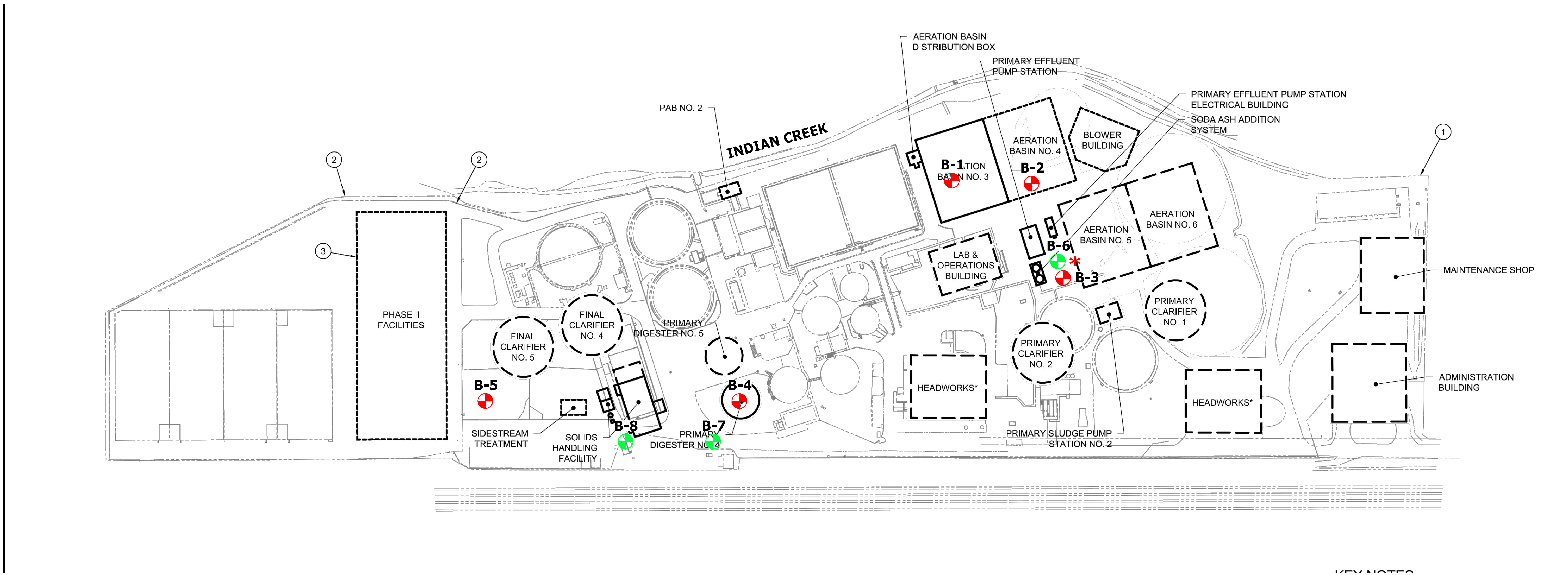
Our services consist of professional opinions and findings made in accordance with generally accepted geotechnical engineering principles and practices in southwest Idaho at the time of this report. The geotechnical recommendations provided herein are based on the premise that appropriate geotechnical consultation during subsequent design phases is implemented and an adequate program of tests and observations will be conducted by STRATA during construction to verify compliance with our recommendations and to confirm conditions between exploration locations. This acknowledgment is in lieu of all warranties either express or implied.

The following plates accompany and complete this report:

Plate 1: Exploration Location Plan

Appendix A: Exploratory Boring Logs and USCS Explanation

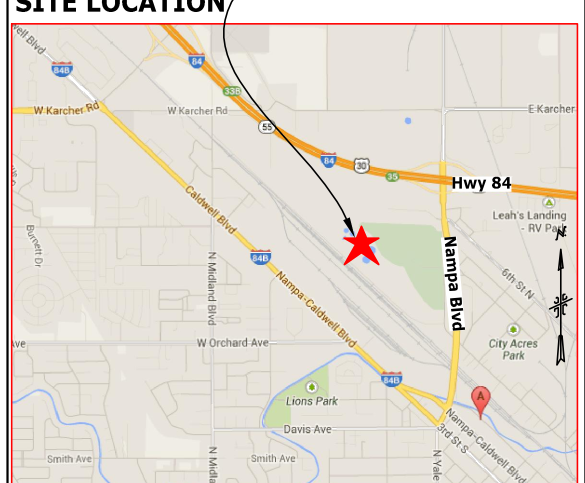
Appendix B: Laboratory Test Results



VICINITY MAP

NOT TO SCALE

APPROXIMATE SITE LOCATION



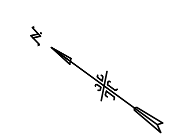
LEGEND

B-1 Approximate boring location observed by STRATA on July 25, 2013.

B-6 Approximate boring location observed by STRATA on June 30, 2014.

* Monitoring well installed in boring.

THIS PLAN COMPRISES A PORTION OF STRATA'S REPORT AND THE TEXT OF THE REPORT CONTAINS ESSENTIAL INFORMATION: BEFORE UTILIZING THIS PLAN FOR ANY PURPOSE WHATSOEVER, THE REPORT SHOULD BE READ COMPLETELY. THIS PLAN IS INTENDED TO HELP VISUALIZE THE INFORMATION PROVIDED IN THE REPORT. THESE LOCATIONS AND INFORMATION WERE ADDED TO EXISTING PLANS OF THE SITE PREVIOUSLY PREPARED BY OTHERS AND NO CHECK OF ACCURACY, CURRENCY, APPROPRIATENESS, ETC., OF INFORMATION PROVIDED BY OTHERS WAS PERFORMED, SINCE SUCH CHECKS WERE NOT PART OF STRATA'S SCOPE OF SERVICES.



EXPLORATION LOCATION PLAN
Nampa Wastewater Treatment Plant
Nampa, Idaho

0 100 200 400

SCALE: 1 inch = 200 feet

STRATA
A PROFESSIONAL SERVICES CORPORATION
Integrity from the Ground Up

DRAWING DATE: 7-16-2014	
DRAWING BY: DMS	CHECKED BY: MGW

Client	Project No.	
CITY OF NAMPA	BO13170A	PLATE: 1

REFERENCE: Site plan provided by Brown and Caldwell.

APPENDIX A

USCS Description	DEPTH (ft)	U.S.C.S. CLASS	SYMBOL	Sample Type	SPT Blows Per 6 Inches	SPT $N_{(60)}$	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	REMARKS Note: BGS = Below Ground Surface
SILTY GRAVEL, With Sand (fill), (GM) tan, medium dense to dense, moist	0	GM								Ground surface Elev.=2457
SILTY SAND, (fill), (SM) brown, very loose to loose, moist		SM			4 4 4	8				
	5				1 2 1	3				
					0 2 2	4				
	10				0 0 0	0				Heave noted at 10 feet.
POORLY GRADED SAND, With Silt And Gravel (native), (SP-SM) tan, medium dense, saturated	15	SP-SM			4 10 10	20		12.1		5.1% Passing No. 200 Sieve Resistivity = 2,564 ohm-cm pH = 8.8, sulfates = 343ppm
	20									Blow counts not recorded due to heave
SILTY SAND, (SM) brown, loose, saturated		SM			0 2 2	4				
LEAN CLAY, (CL) brown, soft, saturated	25	CL			0 2 2	4				Well installed to 25 feet BGS. Screened from 15 to 25 feet BGS.
					0 2 2	4	83.1	39.3		64.0% Passing No. 200 Sieve
SILT, (ML) brown, soft to firm, saturated	30	ML			0 2 2	4				Interbedded silty sand at 30 feet BGS
					4 4 4	8				
	35				4 4 4	8				

Borehole Terminated at 36.5 Feet.

Client: CITY OF NAMPA	**Boring Number:** B-1
Project: BO13170A	**Date Drilled:** 07-25-2013
Drill Rig: BK-81	**Borehole Diameter:** 10"
Depth to Groundwater: 9'	**Logged By:** SW

STRATA
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Integrity from the Ground Up

EXPLORATORY BORING LOG

Sheet 1 Of 1

USCS Description	DEPTH (ft)	U.S.C.S. CLASS	SYMBOL	Sample Type	SPT Blows Per 6 Inches	SPT $N_{(60)}$	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	REMARKS Note: BGS = Below Ground Surface
SILTY GRAVEL, With Sand (fill), (GM) tan, loose, moist	0	GM								Ground surface Elev.=2458
					7 5 4	9				
	5									
POORLY GRADED GRAVEL, With Silt And Sand, (GP-GM) tan, medium dense to very dense, moist		GP-GM								
					8 21 5/5.0"	50+				
	10									
					5 8 8	16				
	15									
SILTY SAND, (SM) brown, very loose, saturated		SM								Blow count not recorded due to heave
SILT WITH SAND, (ML) brown, firm to very stiff, saturated	20	ML			0 4 5	6*				*corrected for sampler size
					4 6 12	18				
	25				0 0 7	4*				Well installed to 25 feet BGS. Screened from 15 to 25 feet BGS.

Borehole Terminated at 26.5 Feet.

Client: CITY OF NAMPA	**Boring Number:** B-2
Project: BO13170A	**Date Drilled:** 07-25-2013
Drill Rig: BK-81	**Borehole Diameter:** 10"
Depth to Groundwater: 7.5'	**Logged By:** SW

STRATA
A Professional Services Corporation
Integrity from the Ground Up

EXPLORATORY BORING LOG

Sheet 1 Of 1

USCS Description	DEPTH (ft)	U.S.C.S. CLASS	SYMBOL	Sample Type	SPT Blows Per 6 Inches	SPT $N_{(60)}$	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	REMARKS Note: BGS = Below Ground Surface
POORLY GRADED GRAVEL, With Sand And Silt (fill), (GP) tan, dense, moist	0	GP								Ground surface Elev.=2457
SILTY SAND, (fill), (SM) brown, very loose, moist		SM			5 1 1	2				
	5									
POORLY GRADED SAND, With Silt And Gravel (native), (SP-SM) tan, loose to medium dense, moist to wet		SP-SM			2 2 2	4				
	10									
					8 10 13	23				
	15									
SILT, With Sand, (ML) brown, very soft, saturated		ML			0 0 1	1				
	20				0 1 0	1*				
					0 1 0	1				
POORLY GRADED SAND, With Silt, (SP-SM) tan, medium dense to dense, saturated	25	SP-SM			11 15 17	20*				
					10 14 10	24				
	30				7 13 5	11*				
SILT, (ML) brown, very soft, saturated	35	ML			0 0 1	1				* Blow count corrected for 3 inch sample.

Borehole Terminated at 36.8 Feet.

Client: CITY OF NAMPA	**Boring Number:** B-3	STRATA A Professional Services Corporation Integrity from the Ground Up	**EXPLORATORY BORING LOG**
Project: BO13170A	**Date Drilled:** 07-25-2013		
Drill Rig: BK-81	**Borehole Diameter:** 10"		
Depth to Groundwater: 12'	**Logged By:** SW		Sheet 1 Of 1

USCS Description	DEPTH (ft)	U.S.C.S. CLASS	SPT Blows Per 6 Inches	SPT $N_{(60)}$	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	REMARKS Note: BGS = Below Ground Surface
POORLY GRADED GRAVEL, With Sand (fill), (GP) tan, dense, moist	0	GP	6 14 18	32				Ground surface Elev.=2457
SILT, With Sand (native), (ML) brown, very soft, moist to saturated	5	ML	0 1 1	2				
LEAN CLAY, (CL) brown, very soft, saturated	10	CL	0 1 1	2				
POORLY GRADED SAND, With Silt, (SP-SM) tan, loose to medium dense, saturated	15	SP-SM	16 8 3	7*				
	20		6 12 2	14				
LEAN CLAY, (CL) brown, soft, saturated		CL						
POORLY GRADED SAND, With Silt, (SP-SM) tan, very loose, saturated		SP-SM	1 2 1	2*				
SILT, (ML) brown, soft to very stiff, saturated	25	ML	0 6 14	12*				
			1 1 2	2*				
	30		5 12 19	19*				* Blow count corrected for 3 inch sample.

Borehole Terminated at 31.5 Feet.

Client: CITY OF NAMPA	**Boring Number:** B-4	STRATA A Professional Services Corporation Integrity from the Ground Up	**EXPLORATORY BORING LOG**
Project: BO13170A	**Date Drilled:** 07-25-2013		
Drill Rig: BK-81	**Borehole Diameter:** 10"		
Depth to Groundwater: 8'	**Logged By:** SW		Sheet 1 Of 1

STRATA BH / TP / WELL - STRATA.GPJ - 10/8/13 15:00 - R:\GINT\PROJECTS\2013\BOISE\BO13170A BORING.GPJ

USCS Description	DEPTH (ft)	U.S.C.S. CLASS	SYMBOL	Sample Type	SPT Blows Per 6 Inches	SPT $N_{(60)}$	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	REMARKS Note: BGS = Below Ground Surface
Asphalt (2")	0									
SILT, (ML) light gray, stiff, moist		ML			6 9 13	22				Ground surface Elev.=2457
SILTY SAND, (SM) brown, loose, saturated	5	SM			0 1 3	4				
	10				3 2 2	4		26.5		38.0% Passing No. 200 Sieve
POORLY GRADED SAND, With Silt, (SP-SM) tan, medium dense, saturated	15	SP-SM			3 8 20	17*				*Blow count corrected for sampler size
	20				2 4 8	12				
LEAN CLAY, (CL) brown, soft to firm, saturated		CL			0 1 2	2*	80.2	41.9		ATTERBERG LIMITS LL = 45 PI = 23
	25				2 5 4	9				
POORLY GRADED SAND, With Silt, (SP-SM) brown, loose to medium dense, saturated		SP-SM			7 3 7	6*				
	30				10 11 11	22				

Borehole Terminated at 31.5 Feet.

Client: CITY OF NAMPA	**Boring Number:** B-5	STRATA A Professional Services Corporation Integrity from the Ground Up
Project: BO13170A	**Date Drilled:** 07-25-2013	**EXPLORATORY BORING LOG**
Drill Rig: BK-81	**Borehole Diameter:** 10"	
Depth to Groundwater: 4'	**Logged By:** SW	Sheet 1 Of 1

USCS Description	Depth (ft)	Symbol	Sample Type	SPT Blows Per 6 Inches	SPT $N_{(60)}$	Dry Density (pcf)	Remarks
POORLY GRADED GRAVEL, With Sand And Silt (fill), (GP) tan, dense, moist	0	GP					Note: BGS = Below Ground Surface
SILTY SAND, (fill), (SM) brown, very loose, moist		SM		5 1 1	2		
POORLY GRADED SAND, With Silt And Gravel (native), (SP-SM) tan, loose to medium dense, moist to wet	5	SP-SM		2 2 2	4		
	10			8 10 13	23		
SILT, With Sand, (ML) brown, very soft, saturated	15	ML		0 0 1	1		
	20			0 1 0	1		
				0 1 0	1		
POORLY GRADED SAND, With Silt, (SP-SM) tan, medium dense to dense, saturated	25	SP-SM		11 15 17	20*		
				10 14 10	24		
	30			7 13 5	11*		
LEAN CLAY, (CL) brown, soft, saturated	35	CL				68.4	Started 5 feet sample interval at 35 feet BGS 100, 100, 100, 150 psi
SILT, (ML) brown, very stiff, saturated	40	ML		10 15 16	31		* Blow count corrected for 3 inch sample.
	45			6 4 10	14		2 ring samples
(RX) BASALT, weathered, highly fractured, black		RX					
	50			50/3.5"	50+		

Borehole Terminated at 50.3 Feet.

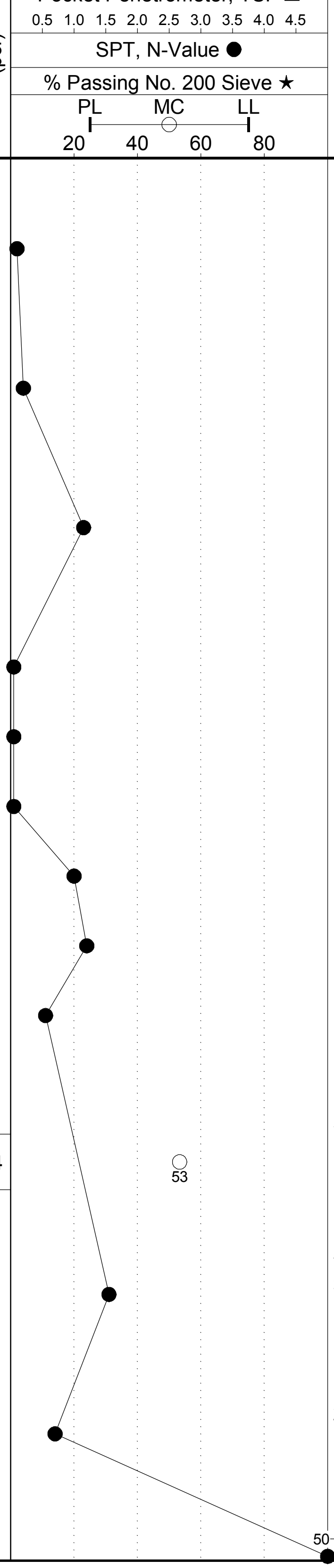
TEST RESULTS

Pocket Penetrometer, TSF ▲

SPT, N-Value ●

% Passing No. 200 Sieve ★

PL MC LL



Client: CITY OF NAMPA	**Boring Number:** B-6
Project: BO13170D	**Date Drilled:** 06-30-2014
Drill Rig: CME 75	**Borehole Diameter:** 8"
Depth to Groundwater: 12'	**Logged By:** SW

STRATA
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Integrity from the Ground Up

EXPLORATORY BORING LOG

Sheet 1 Of 1

USCS Description	Depth (ft)	Symbol	Sample Type	SPT Blows Per 6 Inches	SPT $N_{(60)}$	Dry Density (pcf)	TEST RESULTS	Remarks
SILTY GRAVEL, With Sand (fill), (GM) brown, medium dense, moist	0	GM		18 13 8	21			3 inches asphalt
SILT, (native), (ML) brown, stiff, moist		ML		2 5 7	12			
	5			3 1 1	2			
LEAN CLAY, (CL) brown, stiff to very stiff, moist to saturated		CL	RG	3 4 7	11	95.4	PL 19, LL 28	2 ring samples
	10			4 5 8	13			
				1 3 6	9			
	15			5 6 17	23			100, 100, 50, 50 psi
SILTY SAND, (SM) brown, loose, saturated		SM						
POORLY GRADED SAND, With Silt, (SP-SM) brown, medium dense, saturated	20	SP-SM		11 14 12	26		MC 20	
LEAN CLAY, (CL) brown, soft, saturated	25	CL				84.4	PL 16, LL 33	
				2 2 2	4			
SILTY SAND, (SM) brown, very loose, saturated	30	SM		1 0 1	1			
SILT, (ML) brown, stiff, saturated	35	ML		5 5 10	15			

TEST RESULTS: Pocket Penetrometer, TSF ▲ (0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5); SPT, N-Value ●; % Passing No. 200 Sieve ★; PL MC LL (20 40 60 80)

Remarks — Note: BGS = Below Ground Surface

Borehole Terminated at 36.5 Feet.

Client: CITY OF NAMPA	**Boring Number:** B-7
Project: BO13170D	**Date Drilled:** 06-30-2014
Drill Rig: BK-81	**Borehole Diameter:** 8"
Depth to Groundwater: 12'	**Logged By:** SW

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EXPLORATORY BORING LOG

Sheet 1 Of 1

USCS Description	Depth (ft)	Symbol	SPT Blows Per 6 Inches	SPT $N_{(60)}$	Dry Density (pcf)	Test Results	Remarks
SILTY SAND, With Gravel (fill), (SM) brown, medium dense, moist	0	SM	8 11 11	22			Note: BGS = Below Ground Surface
SILT, (native), (ML) brown, stiff, moist		ML	4 5 5	10			
	5		4 5 6	11			
			5 7 10	17	103.8	MC 25	
CLAY, With Gravel, (CL) brown, firm, saturated	10	CL	2 3 4	7			
POORLY GRADED SAND, With Gravel, (SP) brown, medium dense, saturated		SP	6 10 15	25		MC 17	
	15		6 8 12	20			
LEAN CLAY, (CL) brown, soft to firm, saturated	20	CL	1 1 2	3		PL 20, LL 40	
	25		2 3 5	8			

TEST RESULTS: Pocket Penetrometer, TSF ▲ (0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5); SPT, N-Value ●; % Passing No. 200 Sieve ★; PL, MC, LL (20 40 60 80)

Borehole Terminated at 26.5 Feet.

Client: CITY OF NAMPA	**Boring Number:** B-8
Project: BO13170D	**Date Drilled:** 06-30-2014
Drill Rig: BK-81	**Borehole Diameter:** 8"
Depth to Groundwater: 9'	**Logged By:** SW

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EXPLORATORY BORING LOG

Sheet 1 Of 1

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS	GRAVELS	CLEAN GRAVELS		GW	Well–Graded Gravel, Gravel–Sand Mixtures.
				GP	Poorly–Graded Gravel, Gravel–Sand Mixtures.
		GRAVELS WITH FINES		GM	Silty Gravel, Gravel–Sand–Silt Mixtures.
				GC	Clayey Gravel, Gravel–Sand–Clay Mixtures.
	SANDS	CLEAN SANDS		SW	Well–Graded Sand, Gravelly Sand.
				SP	Poorly–Graded Sand, Gravelly Sand.
		SANDS WITH FINES		SM	Silty Sand, Sand–Silt Mixtures.
				SC	Clayey Sand, Sand–Clay Mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50%			ML	Inorganic Silt, Sandy or Clayey Silt.
				CL	Inorganic Clay of Low to Medium Plasticity, Sandy or Silty Clay.
				OL	Organic Silt and Clay of Low Plasticity.
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%			MH	Inorganic Silt, Mica–ceous Silt, Plastic Silt.
				CH	Inorganic Clay of High Plasticity, Fat Clay.
				OH	Organic Clay of Medium to High Plasticity.
				PT	Peat, Muck and Other Highly Organic Soils.

BORING LOG SYMBOLS	GROUNDWATER SYMBOLS	TEST PIT LOG SYMBOLS
Standard 2–Inch OD Split–Spoon Sample	Groundwater After 24 Hours	BG Baggie Sample
California Modified 3–Inch OD Split–Spoon Sample	(7–3–07) Indicates Date of Reading	BK Bulk Sample
Rock Core	Groundwater at Time of Drilling	RG Ring Sample
Shelby Tube 3–Inch OD Undisturbed Sample		

Shorthand Notation:
BGS = Below Existing Ground Surface
N.E. = None Encountered

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APPENDIX B

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Summary of Test Results

Project: Nampa WWTP
Client: City of Nampa

Project Number: BO13170A
Date: 8/29/2013

Boring	Depth (feet)	Lab Number	Soil Classification (remarks)	Dry Unit Weight, pcf	In Situ Moisture, %	Passing No. 200,%	Resistivity ohm-cm	pH	Sulfates ppm	Atterberg Limits		Fines Class.
										LL	PI	
1	15-16.5	B13L0985A	P.G. Sand with Silt and Gravel		12.1	5.1	2,564	8.8	343			
1	28.5-29	B	Sandy Clay*	83.1	39.3	64						
2	26-26.5	C	Silt with Sand	90.1	35.3							
3	27.5-29	D	P.G. Sand with Silt		14.3	6.9						
4	28.5-29	E	Silt*	73.2	48.2							
5	10-11.5	F	Sandy Silt		26.5	38						
5	23-23.5	G	Lean Clay*	80.2	41.9					45	23	CL

* See Individual Consolidation Graph

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Integrity from the Ground Up

Summary of Test Results

Project: Nampa WWTP Supplemental Hydrogeologic Modeling / Consultation
Client: City of Nampa

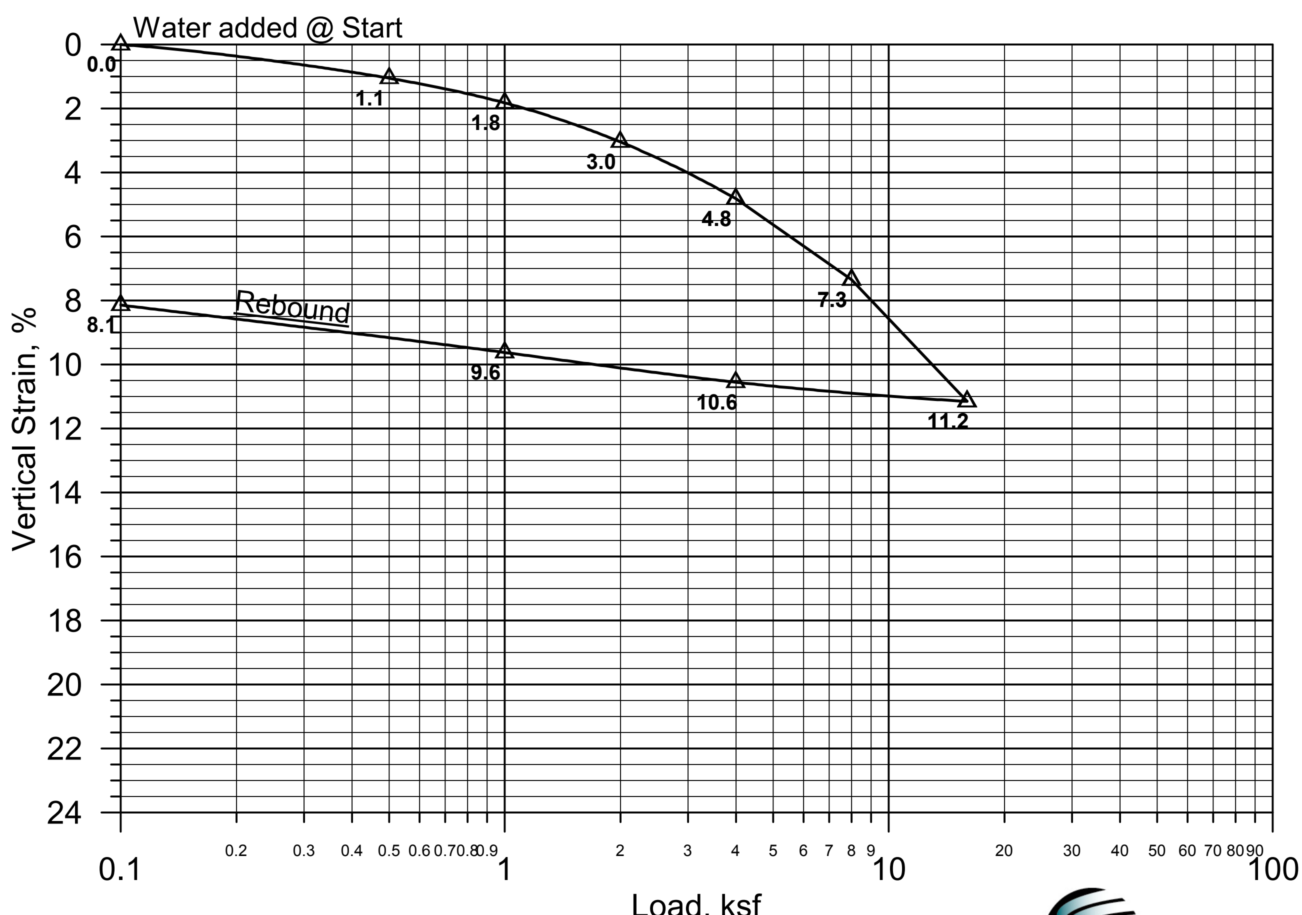
Project Number: BO13170D
Date: 7/30/2014

Boring Number	Depth (feet)	Lab Number	Soil Classification (remarks)	Dry Unit Weight, pcf	In Situ Moisture, %	Passing No. 200,%	Atterberg Limits LL	PI	Fines Class.
6	36 - 37	BO1400746A	Clay*	68.4	53.3				
7	8 - 8.5	B	Lean Clay	95.4			28	9	CL
7	20 - 21.5	C	P.G. Sand with Silt		20.0	9.2			
7	28.5 - 29	D	Sandy Clay	84.4			33	17	CL
8	8 - 8.5	E	Silt*	103.8	24.6				
8	12.5 - 14	F	P.G. Sand with Silt		17.4	7.4			
8	20 - 21.5	G	Lean Clay				40	20	CL

* See Individual Consolidation Graph

CONSOLIDATION TEST RESULTS
ASTM D 2435 (Method A)

Project: Nampa WWTP
Client: City of Nampa
Project Number: BO13170A
Lab Number: B13L0985B
Sample Identification: B-1 @ 28.5 - 29 ft
Sample Classification: Sandy Clay
Sample: In-Situ Tube (Condition: Good)
Date Tested: 8/15-23/13 By: IR
Sample Dry Unit Weight: 83.1 pcf
Moisture Content: 39.3%
Passing No. 200 Screen = 33%



Reviewed By: [signature]

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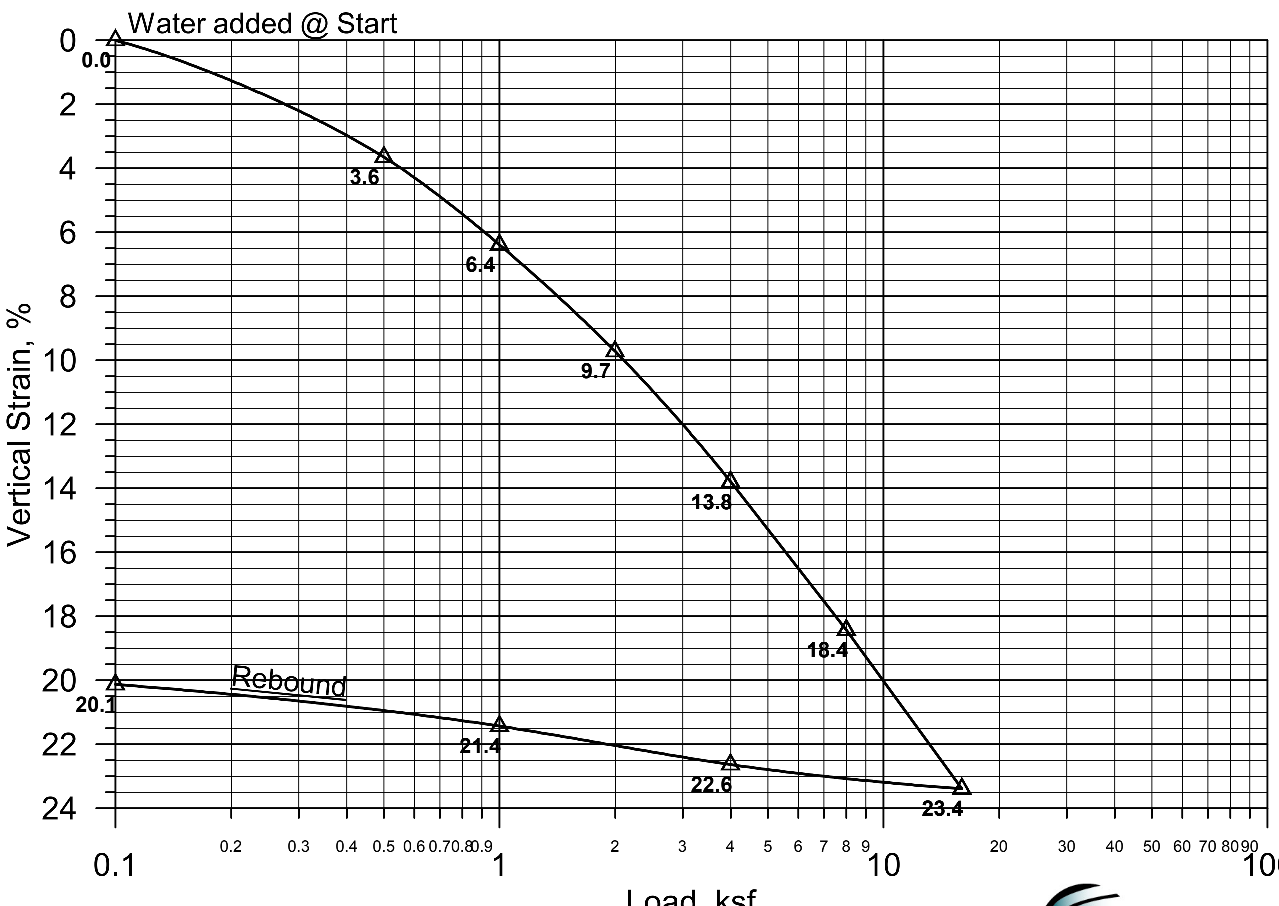
Integrity from the Ground Up

CONSOLIDATION TEST RESULTS
ASTM D 2435 (Method A)

Project: Nampa WWTP
Client: City of Nampa
Project Number: BO13170A
Lab Number: B13L0985E
Sample Identification: B-4 @ 28.5 - 29 ft
Sample Classification: Silt
Sample: In-Situ Tube (Condition: Good)
Date Tested: 8/15-23/13 By: IR
Sample Dry Unit Weight: 73.2 pcf
Moisture Content: 48.2%



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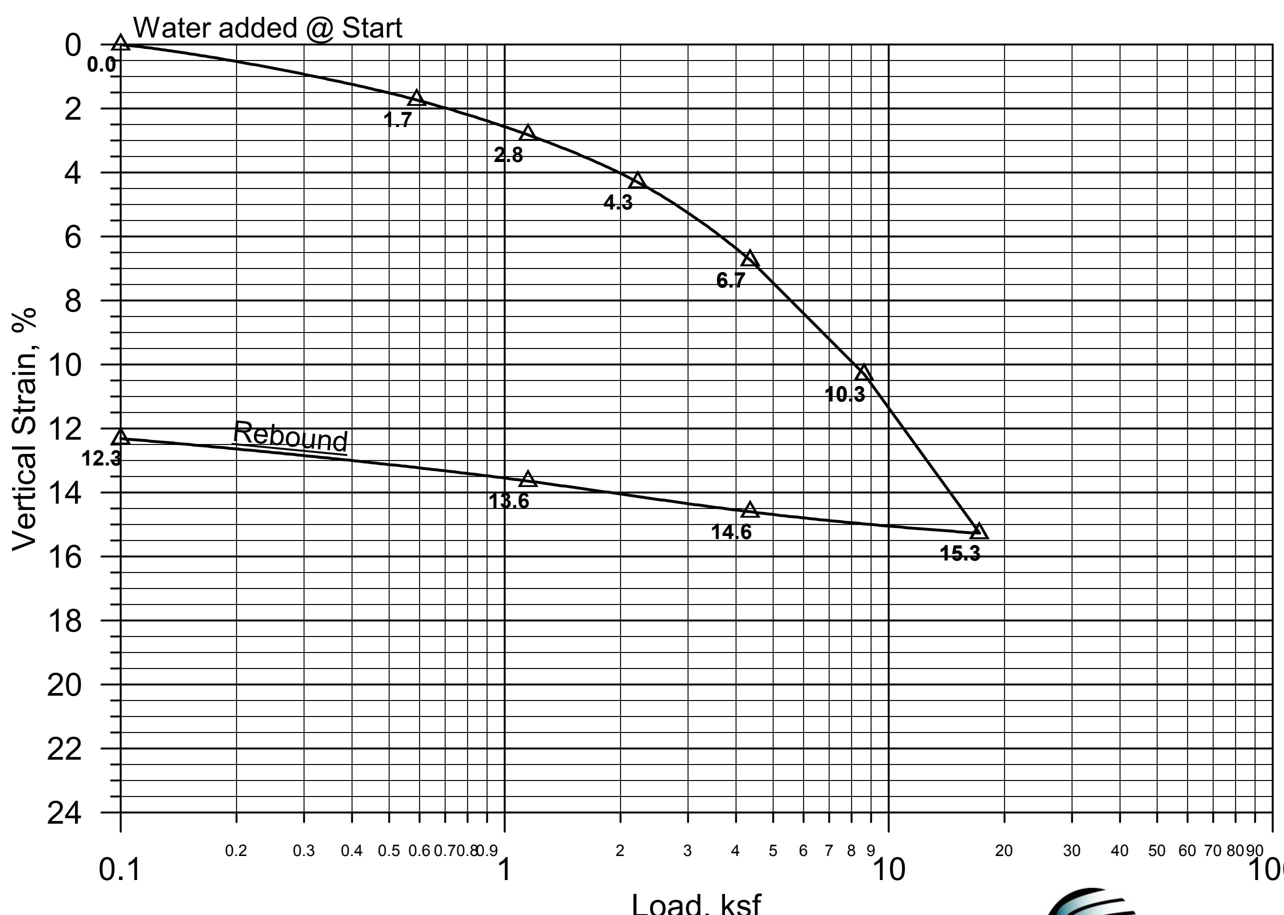
Reviewed By: Adrian Vasconez

CONSOLIDATION TEST RESULTS
ASTM D 2435 (Method A)

Project: Nampa WWTP
Client: City of Nampa
Project Number: BO13170A
Lab Number: B13L0985G
Sample Identification: B-5 @ 23 - 23.5 ft
Sample Classification: Lean Clay
Sample: In-Situ Tube (Condition: Good)
Date Tested: 8/15-23/13 By: IR
Sample Dry Unit Weight: 80.2 pcf
Moisture Content: 41.9%
Atterberg Limits: LL = 45, PI = 23



Reviewed By: Adrian Nascimento

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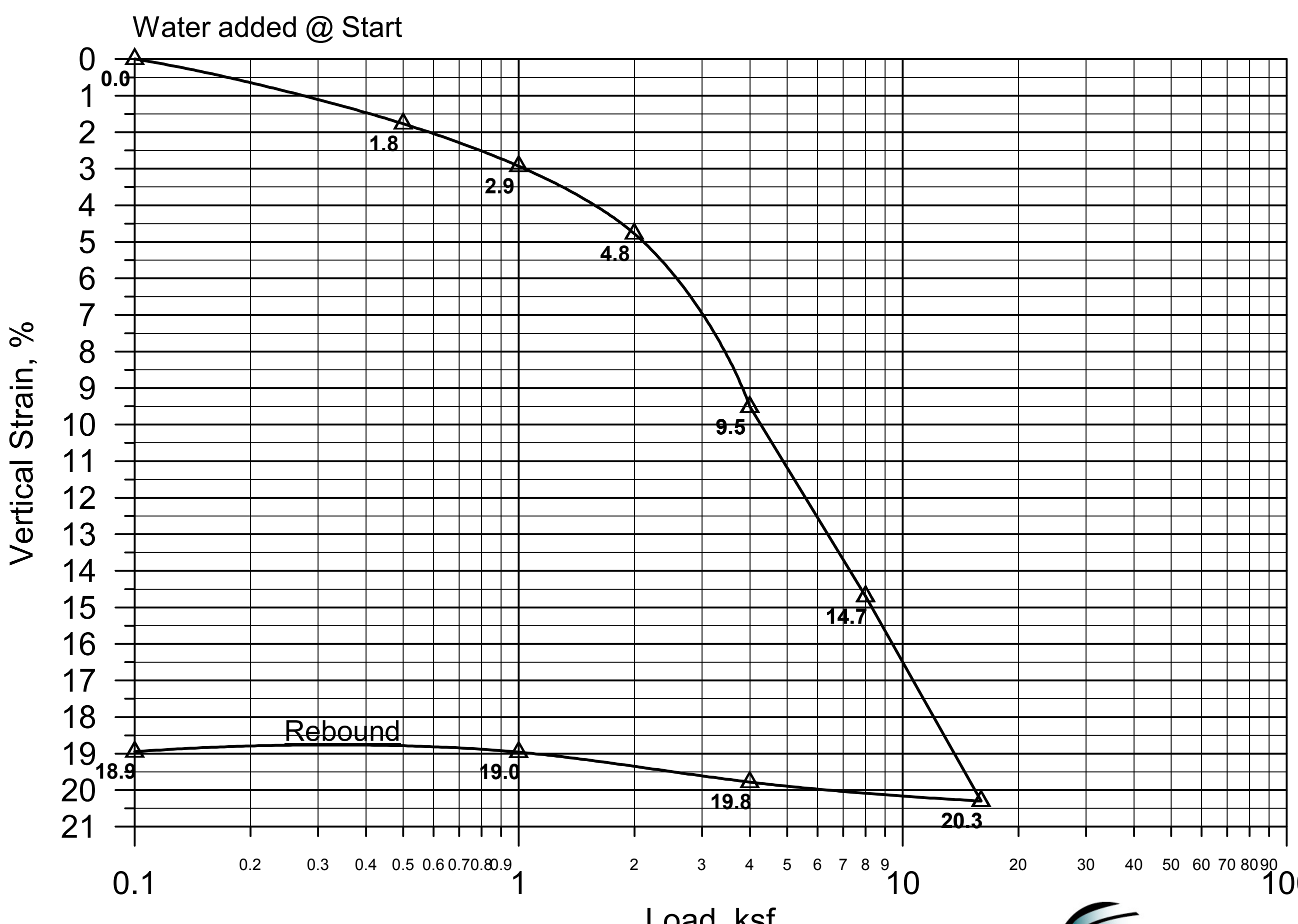
Integrity from the Ground Up

CONSOLIDATION TEST RESULTS
ASTM D 2435 (Method A)

Project: Nampa WWTP Supplemental Hydrogeologic Modeling/Consultation
Client: City Of Nampa
Project Number: BO13170D
Lab Number: BO1400746A
Sample Identification: B6 @ 36 - 37 ft
Sample Classification: Clay
Sample: In-Situ Tube (Condition: Good)
Date Tested: 7/16 - 7/23/14 By: J Sanders
Sample Dry Unit Weight: 68.4 pcf
Moisture Content: 53.3%



Reviewed By:

STRATA
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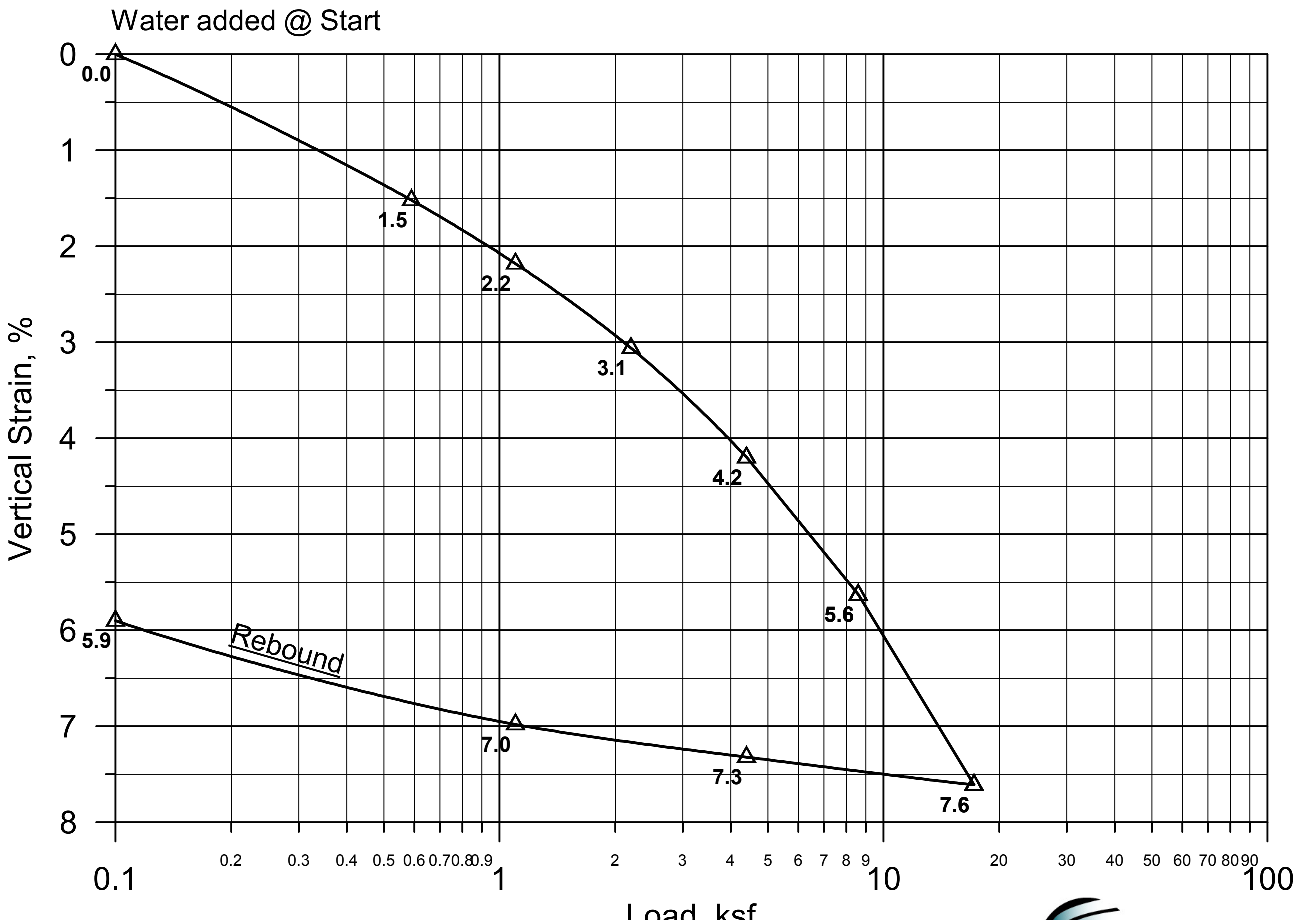
CONSOLIDATION TEST RESULTS
ASTM D 2435 (Method A)

Project: Nampa WWTP Supplemental Hydrogeologic Modeling/Consultation
Client: City Of Nampa
Project Number: BO13170D
Lab Number: BO1400746E
Sample Identification: B8 @ 8 - 8.5 ft
Sample Classification: Silt
Sample: In-Situ Tube (Condition: Good)
Date Tested: 7/16 - 7/23/14 By: J Sanders
Sample Dry Unit Weight: 103.8 pcf
Moisture Content: 24.6%



Reviewed By:

A Professional Services Corporation
Integrity from the Ground Up

Strata Geotechnical Engineering Evaluation – November 21, 2007

1690802/ 6.0

Integrity from the Ground Up

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

STRATA

REPORT

Geotechnical Engineering Evaluation
New Digester
Nampa Wastewater Treatment Facility
Nampa, Idaho

Prepared by
Brent Norris, E.I.T.
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Caldwell, Idaho 83605

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November 21, 2007

STRATA

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

Integrity From the Ground Up

November 28, 2007
File: MONWAT B07179A

Mr. Nick Smith, P.E.
MWH Americas
671 E. Riverpark Ln., Ste. 200
Boise, ID 83706

RE: **REPORT**
Geotechnical Engineering Evaluation
New Digester
Nampa Wastewater Treatment Facility
Nampa, Idaho

Dear Mr. Smith:

STRATA, Inc. has performed the authorized geotechnical engineering evaluation for the proposed digester to be located south of the final clarifiers at the Nampa Waste Water Treatment Plant (WWTP) in Nampa, Idaho. Our services were accomplished referencing our confirming proposal dated August 1, 2007. The accompanying report summarizes the results of our field evaluation, laboratory testing and analyses, and presents our geotechnical engineering opinions and recommendations. Based on our field evaluation and subsequent analyses, it is our opinion the site is suitable from a geotechnical standpoint for the project, provided the recommendations presented herein are implemented for design and construction. Portions or individual sections of this report cannot be relied upon without the supporting text of related sections.

The report presents our geotechnical evaluation and assessment of the hydrogeologic conditions and provides preliminary recommendations or suggestions for approaching site dewatering. Our hydrogeologic assessment provides preliminary estimates for pumping rates and duration for assumed dewatering approaches based on our interpretation of the hydrogeologic conditions. The contractor may review or use these options, but should not rely solely on this assessment in planning for and design of site dewatering and earthwork in wet soil conditions. The contractor should conduct independent site evaluation and additional geotechnical engineering they feel is required for planning and design of their construction dewatering approach.

The success of the proposed construction will, in part, depend on following the report recommendations and utilizing good construction practices. Also, we recommend the City of Nampa retain STRATA to provide geotechnical testing and consultation services during construction to verify our report recommendations are followed, and provide input as site conditions vary. It has been our experience that maintaining continuity with the geotechnical consultant of record helps reduce soil and construction-related errors, and contributes to overall project success and economy

We appreciate the opportunity to continue our relationship with MWH and the City of Nampa. Please contact us if you have any questions or further requirements.

Sincerely,
STRATA, Inc.

PROFESSIONAL ENGINEER
REGISTERED
12744
11/28/07
STATE OF IDAHO
CHRIS M. COMSTOCK

Chris M. Comstock, P.E., P.G.
Project Engineer

H. Robert Howard, P.E.
Senior Engineer

CMC/nl

TABLE OF CONTENTS

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REPORT
Geotechnical Engineering Evaluation
New Digester
Nampa Waste Water Treatment Plant
Nampa, Idaho

INTRODUCTION

This report presents the results of our geotechnical engineering evaluation for the proposed digester to be located at the existing Nampa Waste Water Treatment Plant (WWTP), Nampa, Idaho. The approximate locations of the Nampa WWTP and new digester are shown on Plate 1, Site Plan.

The purpose of our evaluation was to characterize the subsurface soil and hydrogeologic conditions to prepare geotechnical and hydrogeologic opinions and recommendations to be used to assist civil engineering design and preparation of construction drawings and specifications. Specifically, we focused on providing geotechnical recommendations for planning and design, and preliminary dewatering criteria for constructing the proposed digester.

To perform this evaluation, we accomplished the following:

1. Reviewed data from evaluations for existing structures at the WWTP.

2. Reviewed preliminary concepts for the digester project provided by Mr. Nick Smith, P.E. with MWH Global.

3. Coordinated with Mr. Smith and Mr. Greg Pearce with the City of Nampa to identify site access issues to coordinate exploration, and to help avoid site utilities.

4. Subcontracted to drill 1 exploratory boring to a depth of 26.5 feet below the existing ground surface within or near the proposed digester footprint in an area acceptable to MWH and the City of Nampa. We did not install a standpipe piezometer within the boring. We understand the standpipe location would be difficult to access because of normal sludge drying activities and also due to possible contamination of subsurface groundwater from sludge.

5. Coordinated with WWTP staff to accomplish a field test of the existing dewatering system for Final Clarifier No. 2. The intent of this testing was to help characterize the required pumping rates for potential permanent dewatering systems and to assist our estimates for construction dewatering pumping rates. We evaluated the findings from this field evaluation relative to our previous hydrogeologic analyses accomplished for the Primary Clarifier No. 3 and Final Clarifier No. 3 projects.

6. Reviewed our hydrogeologic analyses for Primary Clarifier No. 3, Final Clarifier No. 3, and referenced the field evaluation of the permanent dewatering system for Final Clarifiers No. 1 and No. 2. We also reviewed our notes for dewatering pumping rates from the Final Clarifier No. 3 project and Primary Clarifier No. 3. We referenced the above information to provide hydrogeologic opinions for preliminary dewatering criteria and recommendations for permanent dewatering pump capacity, if a permanent dewatering system is utilized.

7. Performed analyses and prepared geotechnical recommendations for the following:

    - Foundation bearing soil and allowable bearing pressure
    - Estimated digester total and differential settlement
    - Lateral earth pressures (static and dynamic)
    - Temporary excavations
    - Soil subgrade preparation
    - Structural fill and compaction requirements
    - IBC Site Class and Seismicity including short and long period spectral response
    - Flexible pavement design

8. Summarized the findings from our field and laboratory evaluation and our geotechnical analyses and hydrogeologic opinions in a geotechnical engineering evaluation report.

PROPOSED CONSTRUCTION

We understand a new digester will be installed in the area of an existing sludge drying bed south of Final Clarifier No. 1 and Final Clarifier No. 2 at the Nampa WWTP, as shown on Plate 1. The top of the perimeter footings for the digester are currently planned at approximately elevation 2454, which is roughly the existing grade at the sludge dewatering bed. The foundation at the center of the digester is preliminarily planned about 6 to 7 feet below grade. The side water depth of the digester will be approximately 20 feet above the base of the perimeter footing and the digester will allow for gas storage at the upper portion of the digester tank.

Based on the concept drawings provided by Mr. Smith, the digester will consist of an exterior cast-in-place concrete ring wall supported by an approximate 7 to 8-foot-wide footing. The ring wall footing base will be 2 to 3 feet below adjacent outside grade. The digester will have a reinforced concrete slab on-grade, an upper level (suspended) reinforced concrete floor and a reinforced concrete roof, which will be structurally connected to the perimeter footing. Additionally, the suspended floor and roof will be supported by columns. Roof and suspended floor slab column footings may be planned to bear roughly 5 to 6 feet below the slab-on-grade.

Mr. Smith also provided preliminary estimates for digester weight and loading conditions. STRATA reviewed the weight calculations provided by Mr. Smith and we present the following assumptions for the digester loading:

- The total weight of the structure, including sludge, is on the order of 5,200 tons.
- The floor area for the digester is on the order of 4,700 to 5,700 square feet.
- The total resultant pressure from the structure and the sludge is on the order of 1,800 to 2,100 pounds per square foot (psf).
- Structural loads from the perimeter walls and roof, which will transfer to the perimeter footing, may be on the order of 3 to 3.5 kips per lineal foot (klf) (excludes sludge).
- The columns supporting the floor column slab may realize structural loads on the order of 25 to 30 kips per column (excludes sludge).

We assumed the above structural loading conditions and foundation configurations to provide settlement estimates and to develop digester foundation recommendations. We understand MWH is in the process of preliminary design and structural loads have not been fully identified. We request that MWH provide STRATA with structural loading configurations after final design is underway to verify our settlement estimates and foundation recommendations are commensurate with the final structural loads.

We understand the proposed construction will also include a small digester control building located east of the proposed digester. Foundations for the digester control building will consist of conventional, continuous foundations on the order of 18 to 24 inches wide. These foundations will be installed approximately 24 inches below final finished grade, which corresponds to approximately the current drying bed elevation. Structural loads are expected to be light.

At the time of this report, it had not been determined whether a permanent dewatering system would be required for the digester structure. However, we proposed to provide estimates for permanent dewatering pumping rates as discussed in our August 1, 2007, *Revised Confirming Proposal*.

SITE EVALUATION

STRATA subcontracted to drill one boring near the proposed digester on August 3, 2007. The boring location is presented on Plate 1. The boring location was established based on input from Mr. Nick Smith, P.E. with MWH, and our understanding of the proposed digester location. The boring was generally advanced in 5-foot or 2.5-foot intervals using a BK-81 drill rig equipped with 8-inch (outside diameter) hollow-stem augers. The soils encountered in the borings were described and classified in the field by a STRATA engineer referencing the Unified Soil Classification System (USCS). The boring log is provided in Appendix A. A brief explanation of the USCS is also presented in Appendix A and should be used to interpret the terms on the boring log and throughout this report.

Soil samples were generally obtained in the boring using either a 2-inch (outside diameter) split-spoon, 3-inch (outside diameter) ring sampler, or a 3-inch (outside diameter) Shelby Tube sampler. Standard Penetration Test (SPT), N_{60} values were recorded for each sample, with the exception of the Shelby Tube sample which is slowly pushed into the soil using the drill equipment. N_{60} values were obtained by counting the number of hammer blows required to advance the 18-inch-long samplers from 6 to 18 inches. The SPT blow counts for each 6-inch segment of the sampler are presented on the boring logs. SPT values obtained from a 3-inch ring sampler have been corrected for diameter and normalized to a 2-inch, split-spoon sampler.

Subsurface Conditions

Soil conditions encountered during our recent exploration are relatively similar compared to other borings completed at the site. Specific layer contacts and geotechnical data can be referenced on the boring log in Appendix A.

We encountered uncontrolled fill at the ground surface. Fill varied from medium dense silty gravel with sand encountered to a depth of 2.5 feet, to sandy silt and silty sand below 2.5 feet, which were loose or soft and moist to wet. Native alluvium, classified as sandy clay, was encountered below uncontrolled fill at a depth of 6 feet below the existing ground surface. Sandy clay was brown to tan, stiff, and saturated. The sandy clay extended to approximately 13 feet where silty sand was encountered. Silty sand was brown, dense and saturated, and extended to approximately 19.5 feet below the existing ground surface. Sandy clay was encountered below silty sand and was brown, soft, and saturated. Sandy clay was underlain by sandy silt encountered at 24 feet. Sandy silt was brown, soft and saturated, and extended to the termination depth of the boring at 26.5 feet below the existing ground surface.

The static groundwater level measured in the boring was 5.2 feet below the existing ground surface. We measured static groundwater within the boring annulus after letting the groundwater come to equilibrium for approximately 15 minutes. We recommend structural design assume seasonal high groundwater at 2 feet below the boring elevation.

Bedrock was not encountered during exploration. Our review of a Kleinfelder boring log in this area of the WWTP indicates that bedrock is about 45 feet below the existing ground surface. We do not anticipate bedrock will be encountered within planned excavation depths as described in the *Proposed Construction* section above.

Aquifer Field Testing

To gain hydrogeologic information to supplement preliminary dewatering design, and to help evaluate permanent dewatering rates, we measured existing dewatering system discharge rates for Primary Clarifier No. 3 and Final Clarifier No. 1 and No. 2. We understand the dewatering system for Final Clarifier No. 1 and No. 2 are connected to a pump vault between the two clarifiers. Both permanent dewatering systems for the primary and final clarifiers were operating at the time of our site visit. It appeared that Primary Clarifier No. 2 is operational, Primary Clarifier No. 3 was under construction, and Final Clarifiers No. 1 and No. 2 were undergoing renovation. The permanent dewatering system for the primary clarifiers was discharging into a manhole near Primary Clarifier No. 3. The permanent dewatering system for the final clarifiers was discharging to a sludge dewatering bed south of Final Clarifier No. 3.

We measured the groundwater discharge rate for the final clarifier dewatering system using a 5-gallon bucket and stopwatch. We measured an approximate discharge rate of 65 to 75 gallons per minute (gpm) using an average of 5 measurements. Because the dewatering system for the primary clarifiers was discharging into a manhole, we could not obtain discharge measurements. However, by inspection, it appeared the dewatering rate for the primary clarifier system was equal to or slightly higher than as measured for the final clarifiers. Our resulting analyses and hydrogeologic opinions regarding this field testing are provided in subsequent report sections.

Laboratory Testing

We performed laboratory testing referencing ASTM standards. Select soil samples were tested to assess Atterberg limits, in-situ moisture content and grain size distribution. Laboratory test results are presented on the exploratory boring log. A consolidation test performed on the sandy clay sampled at 22 to 23 feet is presented on Plate 2.

We utilized laboratory data from other evaluations performed at the WWTP to assist our interpretation and analyses of the conditions encountered at this site.

HYDROGEOLOGY

The following report sections discuss our approach to develop dewatering options and to help identify and characterize the hydrogeologic conditions at the site. The options and dewatering considerations presented in subsequent sections are not provided as specific hydrogeologic recommendations to be used for final construction dewatering planning. The dewatering options are presented to allow the contractor and the design team to evaluate the characteristics and limitations of several dewatering options. Our opinion is that site dewatering is possible, assuming a well-planned, practical approach is implemented by the contractor.

During project scoping, MWH and STRATA discussed that detailed hydrogeologic exploration field testing and analyses may not be necessary for the digester project for the following reasons:

- STRATA accomplished detailed hydrogeologic analyses for the Final Clarifier No. 3 project located immediately northwest of the proposed digester.

- The proposed digester is planned at shallower depths than the proposed clarifier projects.

- STRATA could reference actual dewatering approaches and pumping rates from dewatering activities for Primary Clarifier No. 3 and Final Clarifier No. 3.

- STRATA would accomplish a limited assessment of the existing permanent dewatering facilities to help estimate dewatering pumping rates adjacent to the proposed new digester.

In our opinion, construction dewatering for Final Clarifier No. 3 was relatively successful. Actual pumping rates were on the order of 100 to 300 gpm for the mass excavation for Final Clarifier No. 3 and the RAS pump station. Dewatering to 2 feet below the clarifier subgrade was accomplished in approximately 3 to 4 months. Given the above discussion, our opinion is that the design team can reference sufficient historical data and hydrogeologic analyses to evaluate construction dewatering options and permanent dewatering pumping rates.

During our February 2004 exploration for Final Clarifier No. 3, STRATA identified a confined, artesian lower aquifer. This aquifer was encountered at approximately 25 feet below the existing ground surface. STRATA accomplished exploration for the new digester project to 26.5 feet below the existing ground surface and did not encounter a confined aquifer. We suspect the confined aquifer exists, but at a greater depth than explored. In addition, our opinion is the suspected confined aquifer below the digester will not impact construction because the base of the digester excavation is anticipated at less than 20 feet below the existing ground surface. Accordingly, we assume the lower aquifer will not influence dewatering activities, groundwater elevations, or permanent dewatering for the new digester project.

Analyses

STRATA accomplished hydrogeologic analyses from our previous evaluations at the Nampa WWTP including:

- Report, *Geotechnical Engineering Evaluation, Proposed Clarifier and RAS Pump Station, Nampa Waste Water Treatment Plant, Nampa, Idaho* by STRATA dated May 18, 2004.
- Report, *Geotechnical Engineering Evaluation, Proposed Primary Clarifier No. 3, Nampa Waste Water Treatment Facility, Nampa, Idaho* by STRATA dated December 6, 2005.

The *Hydrogeology* sections of the aforementioned reports present a detailed discussion of our hydrogeologic analyses and hydraulic conductivity estimates. In addition to referencing the above data, we obtained field information from permanent dewatering systems at the Nampa WWTP site as discussed in the *Aquifer Field Testing* section above. We referenced our understanding of the permanent dewatering system, clarifier configurations, and measured pumping rates to estimate hydraulic conductivity from the recent aquifer field testing. Our analyses and experience indicate the hydraulic conductivity as measured via aquifer field testing may be on the order of 10^{-4} centimeters per second (cm/sec). This estimate is consistent with our previous hydraulic conductivity estimates developed through hydrogeologic analyses and aquifer testing.

Dewatering Issues

Based on our understanding of the proposed construction and the conditions encountered during exploration, we anticipate the majority of the subsurface soil encountered during excavation will consist of sandy clay (which is suspected to contain clayey sand layers at the project site) underlain by silty sand. In general, these soils do not transmit high volumes of water during dewatering activities, but have the potential to flow under saturated conditions. Further, because of a phenomenon known as capillary rise, these soils may remain saturated even if the groundwater table is greater than a few feet below subgrade. Capillary rise occurs as a result of water tension, which draws moisture from the groundwater table into the overlying soil. The estimated capillary rise, based on grain size evaluation, is on the order of 2 to 7 feet. We believe this condition affected the dewatering and subgrade preparation at the time of the Final Clarifier No. 3 project. We suspect that, even if dewatering is successful in the digester excavation, the soil expected at the subgrade may remain saturated and could become easily disturbed.

While dewatering will be required within the silty sand, our geotechnical recommendations presented in subsequent sections account for capillary rise and difficulties that may be experienced to achieve a stable subgrade.

Our opinion is Indian Creek will not influence dewatering. We base this opinion referencing our analyses, which indicate dewatering activities do not draw groundwater down more than a few feet from the excavation. As an example, dewatering activities were ongoing for Final Clarifiers No. 1 and No. 2 at the time of exploration for Digester No. 3. We suspect groundwater was drawn down at these clarifiers to a depth up to 20 feet below the existing ground surface. However, the groundwater level measured in the boring (approximately 75 feet from Final Clarifier No. 2) was measured at 5.2 feet. Because dewatering activities at the clarifiers do not appear to influence groundwater at a reasonable distance from excavations, similarly, Indian Creek is not expected to influence the dewatering activities for the digester because of its relatively large distance from the proposed new digester.

Dewatering Options

Several dewatering approaches have been identified to allow construction at the site. We understand it will be the contractor's responsibility to develop a specific dewatering approach that reflects their capabilities, equipment, schedule, and construction approach. We recommend the contractor's specific dewatering plan be submitted to the City for review and comment. The following section presents general concepts or preliminary options for site dewatering to assist the contractor in gaining understanding of the hydrogeologic conditions at the site for planning and design of site dewatering. This section does not present a specific dewatering design that can be relied upon or specifically used during construction. The specific dewatering plan should consider the potential for seasonal fluctuation in precipitation, irrigation, infiltration, and infrastructure additions to the project site. Further, specific aspects of the site will affect dewatering outcomes including, variations in subsurface geology, ongoing dewatering activities during construction, and an aquifer groundwater gradient sloping to the north.

The following text discusses potential dewatering options, schedule, and other considerations. This is a partial list of dewatering options, and we anticipate the selected approach will be a combination of several alternatives. Further, we expect the methods implemented to dewater the site will be a dynamic process, based on actual site and hydrogeologic conditions encountered during construction. The following methods below were developed, in part, based on our understanding of successful dewatering approaches for the existing clarifiers and aeration basin as indicated in discussions with MWH and the City of Nampa.

Well Point Option

Closely spaced well points are one option to help dewater the upper aquifer to allow construction to occur. Well points could be drilled or driven into the soil to the full depth required for dewatering or in an iterative manner using small benches or terraces. It should be noted that the nature of the hydraulic conductivity of the site soil within the upper aquifer will cause relatively steep drawdown curves and the groundwater level between well point locations will be significantly higher than at the well. Well losses are expected to be significant for wells constructed in the soil comprising the upper aquifer (i.e. the soil above the clay layer at 20 feet). Due to well losses, complete dewatering of the upper aquifer to the clay layer may not be possible. Total pumping rates for a configuration of well points can be expected on the order of 50 to 200 gpm, depending upon the contractor's selection of well diameters, well depths, and the number of wells. We understand the contractor's approach to dewatering for the Primary Clarifier No. 3 and Final Clarifier No. 3 was a system of well points in conjunction with intercepting groundwater from the sidewalls of excavations and routing that groundwater to a pump vault. It should be noted that dewatering pumping rates for both aforementioned dewatering activities varied between 100 to 300 gpm. The existing dewatering pumping rate for the permanent dewatering systems are on the order of 65 to 75 gpm.

It may be more economical and efficient to utilize centrifugal pumps at the ground surface with a solid pipe installed within the screened well points. Centrifugal pumps at the ground surface can only draw water from depths up to approximately 17 to 18 feet before the pumps break suction. We anticipate that a header or manifold system could be used in conjunction with high volume centrifugal pumps for each terrace or bench of well points.

Large Diameter Excavation Dewatering

A large diameter excavation dewatering system refers to large diameter well points or "glory hole" style of dewatering. This method of dewatering is typically employed by excavating slightly below the groundwater surface, locally dewatering the excavation and installing large-diameter, perforated casing to the base of the excavation. Data collected at the site suggests the zone of influence of the dewatered area will be small. It will likely require several of these large-diameter glory holes or large-diameter well points to achieve the necessary surface area to allow dewatering and construction of the digester.

Using this option, once the groundwater has been drawn down, and the digester excavation has been completed, localized trenches, sump pits or groundwater collection galleries may be required at the base of the excavation to collect perched groundwater or runoff. Excavations extending below the water table will be unstable and will experience flowing soil conditions. The contractor should prevent flowing soil at the sides and at the base of the excavations. This dewatering option may require a long time period of pumping to draw groundwater levels down sufficiently to allow construction. From experience and analyses, pumping rates are expected to vary between 100 and 200 gpm for this method of dewatering.

Perimeter Excavation Drainage

This method of dewatering refers to the approach that was utilized by the contractor for the Final Clarifier No. 3 and RAS Pump Station dewatering approach. The excavation contractor utilized a combination of several methods to dewater the aquifer. A few large excavations were accomplished around the two structures using trench boxes or "glory holes." Some depression of the groundwater surface was accomplished and then the contractor began mass excavating for the two structures. The mass excavation was dewatered locally along the perimeter by using drain rock and 4-inch, outside-diameter perforated pipe sloped to a sump pit. Some flowing soil and excavation instability was observed along the perimeter of the excavation.

The RAS Pump Station was unable to be dewatered within 30 days, and the contractor and the City of Nampa opted to overexcavate the saturated sand at the subgrade into the underlying clay soil and replaced the excavation with granular pit run.

The granular pit run soil intercepted groundwater flow from the sides of the excavation. The contractor installed a 4-inch (outside-diameter) perforated pipe around the perimeter of the excavation to remove water from the pit run soil below the RAS foundation.

The clarifier excavation was dewatered with a perimeter drainage system and large glory hole at the center of the excavation. The contractor communicated that the glory hole may have caused heave of the lower aquifer and a significant portion of pumped water at the center could be attributed to the lower aquifer, which is discussed in more detail in the *Site and Subgrade Preparation* and *Excavation Characteristics* section on this report. Several seeps and isolated lenses of granular soil within the slope of the excavation allowed groundwater to flow over the clarifier subgrade. Dewatering was accomplished after about 60 to 90 days using an iterative dewatering and excavation approach, which allowed underdrain construction to begin. Underdrains for the clarifier floor dewatering system were also used to assist dewatering of the excavation.

It is our current opinion this dewatering approach may be best suited to accomplish dewatering for the new digester. If utilized, the perimeter drain should completely surround the digester excavation and drain positively to a sump pit. It may be necessary to install one or more isolated well-points or glory holes at the center of the digester excavation to accelerate the dewatering schedule.

One drawback to this iterative dewatering approach is that the perimeter drain must be excavated below the groundwater table. This will result in flowing soil, sand boils, and other excavation instability, as was observed in isolated locations for the Final Clarifier No. 3 excavation. Total pumping rates were estimated from the Final Clarifier No. 3 excavation at between 20 and 60 gpm, which exclude pumping rates from exterior well points, the center "glory hole" and trench box dewatering locations. As previously discussed, the total pumping rates for the entire system of dewatering is on the order of 60 to 200 gpm.

Alternative Options

Other methods of dewatering are possible including localized dewatering within an enclosed sheet pile excavation or installing the underdrain system prior to initiating digester construction. If the site is dewatered and the underdrain system is functional, it could be used to supplement and possibly maintain dewatering of the upper aquifer while digester construction occurs. We understand contractors at the Nampa WWTP have successfully employed use of a completed permanent dewatering system to supplement dewatering activities to allow concrete placement and formwork to occur.

If the contractor elects to utilize sheet piles to assist the dewatering effort, they must be designed by a licensed engineer and account for perched water behind the sheet pile, potential flowing soil, and artesian conditions that may be encountered during construction. If an enclosed sheet pile excavation is designed appropriately, localized dewatering can be completed within the enclosure. It should be noted that flowing soil and sand boils are still possible above the clay layer if sheet pile construction is initiated.

General Well and Pump Considerations

Establishing a successful dewatering program will be contingent on individual spacing, pumping rates and well construction. Well construction has the potential to limit pumping rates. Further, water production may be reduced as groundwater is drawn down, and transmissivity decreases. Actual pumping rates will be controlled by the saturated thickness near the well and variations in hydraulic conductivity expected in the soil, which will be reflected by the actual number of wells needed and the pumping requirements. Our opinion is that each well may need to be instrumented with water level indicators to shut down the pump as the water level approaches the pump intake. This level generally should be set a few feet above the actual pump intake, which may be several feet below the top of the clay layer. It should be possible to maintain relatively constant water levels by setting the pumps to turn on and off as necessary in combination with pumping rate adjustments. Pumps should be active as much as possible to maintain as much drawdown at the well as possible without causing the pump to burn up. Pump cycles should be set accordingly, such that the pumps are pumping for a longer period of time than they are shut down. If the well is shut down for too long, groundwater levels will not decrease, only fluctuate. We recommend the contractor establish a groundwater discharge location that does not conduct water to the site groundwater system, and meets regulatory agency requirements.

Dewatering Schedule and Drawdown Verification

STRATA previously evaluated drawdown schedules from our previous hydrogeologic evaluations at the Nampa WWTP site. Our analyses, assumptions, and resulting findings were presented in our previous hydrogeologic evaluations. These evaluations reflected estimates to dewater the project site between 20 to 60 days. The contractor's selected dewatering approaches for the Final Clarifier No. 3 and Primary Clarifier No. 3 achieved drawdown in 30 to 90 days. Consequently, the dewatering time depends on the dewatering system implemented, depth, size and location of the structure. We strongly recommend the contractor accomplish dewatering in advance of initiating excavation to minimize slope failures and achieve a stable soil foundation subgrade.

Monitoring of groundwater levels will be required prior to initiating excavation. This may be accomplished using any existing wells and piezometers that may be adjacent to the digester site or by installing piezometers specifically to verify the required drawdown has been achieved in both aquifers. Monitoring of drawdown is critical to construction timing and to help plan against excavation instability, including flowing soil and sand boils. Reusable hand driven piezometers are available from several manufacturers. These piezometers can be driven with standard T-post drivers and are available with continuous electronic monitoring systems. We recommend the contractor's dewatering plan outline the methods they will use to verify groundwater levels prior to initiating excavation.

Construction Dewatering Criteria

We recommend the City of Nampa and MWH incorporate the following dewatering criteria into the plans and specifications for the project.

1. The contractor must submit a dewatering plan for review by the City. Approval of the plan by the City shall not alleviate the contractor from assuming full responsibility for their plan.

2. Dewatering the upper aquifer shall be initiated a minimum of 30 days prior to initiating earthwork construction at the project.

3. The contractor shall coordinate and plan a groundwater discharge location that meets all regulatory agency requirements and does not allow discharged groundwater to infiltrate the aquifer.

4. Well-points or dewatering locations within the aquifer shall be spaced no further than 20 feet on center. In the event an excavation is used to collect groundwater from the sidewalls, the collection facility (pipe or ditch) must be continuous around the perimeter of the excavation.

5. The sides of the excavation for the clarifier shall be completely dewatered at all times, to increase worker safety and reduce the potential for slope failure due to hydrostatic pressures from groundwater.

6. Standpipe piezometers shall be installed within the upper aquifer to verify drawdown has occurred for the clarifier excavation prior to initiating excavation. Excavation can commence and continue no closer than 2 feet above the static groundwater table as dewatering commences.

7. STRATA, MWH, or the City of Nampa must verify dewatering has occurred prior to excavation and prior to placing structural fill or concrete at the foundation bearing elevation.

Dewatering Plan Aspects

We recommend the contractor provide, at a minimum, the following criteria as part of their dewatering plan. The contractor should submit the dewatering plan to the design team at least four weeks prior to initiation of the excavation. The dewatering plan should include:

1. A summary of the contractor's approach to dewatering.

2. Location and depth of all planned well-points or dewatering facilities for the upper aquifer.

3. Well construction or dewatering facility construction details.

4. Method to verify drawdown has occurred to at least 2 feet below subgrade.

5. Groundwater discharge locations and appropriate permits from regulatory agencies (if applicable).

6. Timeline of dewatering schedule versus excavation and construction.

7. Operations and maintenance plan for dewatering systems to show that dewatering systems can be effectively operated during all aspects of digester construction.

8. Show that all wells have been designed, constructed, and permitted in accordance with IDWR regulations.

9. Name and contact information of the contractor's representative responsible for maintaining and operating the dewatering system during all aspects of digester construction.

Hydrogeologic Summary

In summary, a specific dewatering plan must be developed by the contractor based on the location and configuration of the proposed construction. We consider perimeter excavation drainage and large diameter excavation dewatering the most viable alternatives for dewatering the aquifer, but not the only alternative. However, the selected contractor must evaluate the site conditions, potential dewatering options and considerations relative to their dewatering design and construction approach.

GEOTECHNICAL OPINIONS AND RECOMMENDATIONS

Based on our understanding of the proposed construction, interpreted site geologic and hydrogeologic conditions, and results from our analyses, it is our opinion the site is suitable for the proposed construction. However, it will be necessary to carefully plan and stage construction to allow dewatering, excavation, and backfill to be accomplished as proposed.

The recommendations contained in this report reflect our understanding of the location and configuration of the proposed construction, hydrogeologic conditions and subsurface conditions. If design plans change or subsurface conditions encountered during construction vary significantly from what was observed during our subsurface evaluation, we should be notified to review the report recommendations and make any necessary revisions. Understanding and implementation of these recommendations will require our involvement with the contractor, design team, and owner to verify correct report interpretation.

The report recommendations reflect our interpretation of the subsurface conditions at boring locations. However, the subsurface conditions could vary at the proposed site. The variation in subsurface conditions will not be known until construction, and may affect the scope of the construction effort.

Design Assumptions and Conditions Affecting Recommendations

We assume the contractor will accomplish construction by open-excavating the digester area following dewatering. Connecting utilities and piping to the digester will likely be constructed using a trench excavation and portable shoring or trench boxes. For trench stability and earthwork construction, we have provided recommendations for a dewatered condition so hydrostatic pressures do not occur within the excavation. Excavation equipment and other construction procedures should not induce dynamic loading which could increase soil pore water pressure causing local liquefaction, which may lead to both side slope and foundation soil instability. Further, our settlement estimates and geotechnical recommendations are contingent upon following report recommendations for compaction, site preparation and dewatering.

Three primary loading conditions have been identified and analyzed according to the following assumptions to prepare the report recommendations:

- The ring wall has up to 3.5 kips per lineal foot distributed over a 7 to 8-foot-wide footing, resulting in a contact pressure of approximately 500 pounds per square foot (psf).
- The four interior columns support up to 30 kips by at least a 7-foot-square footing resulting in a contact pressure of approximately 500 psf.
- The average *net contact pressure* the digester slab-on-grade exerts on the foundation soil is approximately 1,600 psf.
- Basalt bedrock occurs at an approximate depth of 45 feet.

Earthwork

Topsoil or soil containing significant vegetation and organics was not encountered during exploration. However, we encountered dewatered sludge at the ground surface and uncontrolled fill extending to approximately 5 feet below the existing ground surface. The fill is not suitable for use as structural fill for this project and should be removed from the area or stockpiled for later use as landscaping material. The uncontrolled fill encountered to 5 feet below the ground surface should also be removed below the digester footprint and where other improvements, such as vehicle access/egress are planned.

We recommend the sandy clay (encountered from 6 to 13 feet below grade) be removed to 15 feet outside the perimeter ring wall footing of the digester. The sandy clay should be removed to the underlying silty sand (approximately elevation 2440) anticipated at approximately 13 feet below the existing sludge drying bed elevation. STRATA should be retained to verify this overexcavation and that the excavation has been accomplished to the required distance outside the edge of footings and to the required depth. The excavation should be accomplished in dewatered soil and be completed with smooth blade equipment to reduce disturbance to the exposed foundation soil. Construction of the excavation may utilize other than smooth blade equipment to within 1 foot of the required subgrade and could then remove the last foot to achieve the required foundation subgrade using smooth blade equipment. If the underlying sandy clay is disturbed from construction activity, it should be removed to undisturbed, stiff or medium dense subgrade soil using smooth blade equipment and the overexcavation replaced with approved structural fill according to this report.

In addition to the overexcavation requirements for the digester, the uncontrolled fill encountered from the ground surface extending to 6 feet must be removed within the digester control building footprint and extending to 5 feet outside the edge of footing. The uncontrolled fill should be completely removed to the underlying sandy clay. The excavation must also be completed with smooth blade equipment as discussed in detail above. Complete removal of the sandy clay (as required for the digester) is not required below the control building. After the overexcavation is complete, structural fill may commence to the desired digester control building foundation subgrade.

Uncontrolled fill removal, sludge bed demolition, and digester excavation can commence and continue to 1 foot above the static groundwater table as dewatering proceeds. Excavation to achieve the digester subgrade should not extend into saturated soil. Thus, a minimum of 1 foot of dewatered soil must be maintained above the static groundwater level during dewatering and excavation. Once the digester subgrade has been achieved, the groundwater must be maintained 2 feet below the subgrade during digester construction. These recommendations may require revision if the subgrade is pumping, rutting, weaving or exhibiting other disturbance for the selected equipment being used to accomplish the earthwork.

We have recommended uncontrolled fill be removed below proposed asphalt pavement or below other improvements that could be adversely affected by foundation soil settlement. However, to provide project economy, the City of Nampa may elect to leave the uncontrolled fill in place below proposed new asphalt pavement, etc. In our opinion, and referencing our understanding of the proposed construction, new asphalt pavement at the project site may be constructed at the same grade as the existing asphalt pavement adjacent to the sludge drying beds. The sludge drying bed will be demolished and structural fill will be placed in the resulting excavation to elevate the site to the pavement subgrade adjacent to the digester. Accordingly, the uncontrolled fill is not expected to realize an increased surcharge from new pavements. Because the uncontrolled fill has existed at the project site for many years and has likely come to equilibrium for the historic loading conditions, construction of asphalt pavement may not induce additional settlement, provided pavement loading conditions do not increase. Because structural fill will likely be placed adjacent to the digester or in the demolished sludge drying bed area, the majority of the pavement section is expected to exist over structural backfill. Our opinion is this scenario will produce surcharge loading in excess of the historic load, and additional fill settlement may be induced. Ultimately, the City of Nampa should realize that there is some risk in allowing uncontrolled fill to remain in place below pavements, which would be realized as asphalt cracking and distress and/or general settlement of the pavement relative to other existing structures. We recommend STRATA be consulted before a final decision is made relative to final design and construction of improvements exterior of the digester.

The on-site soil has the potential to infiltrate drain rock that may be installed as part of the proposed underdrain system and for digester wall backfill. Therefore, we recommend placing a woven or non-woven geotextile fabric at the base of the subgrade to control fines migration into the drain rock. We recommend geotextile fabric utilized for the project be Amoco™ 1199, Amoco™ 4552 or have the following properties.

Mullen Burst Strength (ASTM D3786)	– 250 psi (minimum)
Grab Tensile Strength (ASTM D4632)	– 180 lbs (minimum)
Apparent Opening Size (ASTM D4751)	– 70 to 120 sieve

Flow Rate (ASTM D4491) – 4 gal/min/ft^2 (minimum)

STRATA should be contacted to observe excavation and subgrade preparations immediately prior to geotextile placement and granular structural fill placement. Due to the disturbance susceptibility of the native soil, it will be necessary to rapidly achieve subgrades, place geotextile fabric, and place granular structural fill; the contractor should schedule construction accordingly. We recommend the geotextile fabric be placed everywhere drain rock is placed adjacent to native soil.

Structural Fill

Fill placed to develop the site should consist of structural fill and granular structural fill. Structural fill may be used as pipe and structure backfill but only granular structural fill or drain rock may be used to support structures. Structural fill should be free from vegetation and organic matter and consist of GW, GP, GM, SW, SM, SP, or ML soil as designated by the USCS. Granular structural fill should consist of sand or gravel classified as SP, SW, GW or GP by the USCS and contain less than 10 percent passing the No. 200 sieve. Structural fill should not contain particles greater than 6 inches in diameter. Granular drain rock should have particles no larger than 3 inches and should be a washed product capable of free drainage. The on-site silty sand and sandy clay may be reused as structural fill providing it is moisture conditioned sufficiently to allow the contractor to achieve compaction requirements. The on-site silty sand and sandy clay are not suitable for reuse below structures, but are suitable for reuse as trench backfill, in landscape areas, or as backfill for the structure. On-site soil containing vegetation, organics, dewatered sludge, or other debris may not be used as structural fill. The contractor should expect significant moisture conditioning efforts when utilizing any of the native, on-site soil and difficulty in achieving uniform moisture conditions to obtain the required compaction.

Backfilling should be accomplished in accordance with MWH project specifications We recommend structural fill be placed in maximum 12-inch-thick, loose lifts at near-optimum moisture content. Structural fill placed at the site should be compacted to at least 95 percent of the maximum dry density of the soil as determined by ASTM D 698 (Standard Proctor), or to 65 percent relative density based on ASTM D

4253 and D 4254 if the soil contains more than 30 percent particles passing the ¾ inch sieve (i.e. oversize particles).

The native soil, if wet or saturated, has the potential for disturbance and/or construction-induced liquefaction due to vibratory compaction equipment. If vibratory equipment is used, care should be taken to avoid excessive vibratory compactive effort on structural fill placed directly over wet, native soil. If the soil is disturbed, which includes weaving, pumping, rutting or visual contamination of gravel placed over native soil, it will be necessary to completely remove the disturbed area to undisturbed native soil and replace the excavated area with approved granular structural fill.

These compaction requirements assume large (5 ton drum weight or larger) compaction equipment such as sheeps-foot rollers or smooth-drum rollers will be utilized. The lift thickness must be reduced when using light compaction equipment with less than 5 ton drum weight. If earthwork and structural fill placement is completed under wet conditions, we recommend the contractor have contingencies for replacing soft, wet soil with granular structural fill or drain rock placed over geotextile fabric. Structural fill should never be placed over disturbed or frozen subgrades. We recommend STRATA be retained to evaluate the condition of on-site soil for reuse as structural fill and to monitor compaction during structural fill placement.

Compaction of backfill within 5 feet of walls should be performed only with small vibratory plates or walk-behind, smooth drum, vibratory rollers to reduce surcharge loading of the walls. Walls designed for little or no wall movement should be monitored during the backfilling process through survey and string line methods. Below-grade digester walls should be backfilled as described in the *Permanent Dewatering* section of this report or as specified by MWH if the permanent dewatering system will not be necessary.

Wet Weather/Wet Soil Construction

The on-site silty sand and sandy clay encountered within the upper 26 feet of the soil profile will likely maintain significant moisture content even after dewatering has occurred. Earthwork construction practices should acknowledge the potential for soft soil subgrades and high disturbance potential.

Site construction could occur during winter or fall months, which typically exhibit inclement weather and generally poor construction conditions. If site construction is undertaken during wet weather periods, wet soil for structural fill or any soil exposed at the digester subgrade will be susceptible to pumping or rutting from heavy loads such as rubber-tired equipment or vehicles. Earthwork should not be performed immediately after rainfall or until soil can dry sufficiently to allow construction traffic without disturbance. If construction commences before soil can dry after dewatering or precipitation, or during wet periods of the year, earthwork should be performed by low pressure, track-mounted equipment that spread the vehicle load. All soft and disturbed soil should be removed to undisturbed firm or dense native soil as outlined in the *Earthwork* section of this report. If native soil or structural fill is wet and soft but not disturbed, the following lift of structural fill placed over the subgrade should be a minimum depth of 12 inches. Structural fill placement and compaction should be such as to prevent pumping and disturbance of the underlying soft soil. During construction, runoff from precipitation or additional moisture seepage from excavation sidewalls should be intercepted and diverted to prevent ponding of water within the project excavation.

We recommend STRATA be periodically present during excavation and subgrade preparations to verify no soft or pumping areas exist prior to placing structural fill or concrete. We expect wet to saturated conditions may be encountered during digester foundation excavations and subgrade preparation. The contractor should expect these conditions and be equipped to replace wet or disturbed soil with granular structural fill or geotextile fabric and drain rock. If significant soft soil conditions are encountered, the use of a geotextile fabric within overexcavated areas may be necessary. STRATA should be consulted before placing either any geotextile fabric within overexcavated areas in and/or to the fabric already planned for the underdrain and digester subgrades.

Once final subgrades are achieved, it will be the contractor's responsibility to protect the soil from degrading under construction traffic and/or wet weather. Concrete or structural fill placement directly over the subgrade should not be attempted following a significant precipitation event and the subgrade should never be allowed to freeze.

The condition of the subgrade and careful construction procedures are critical to foundation and slab stability and long-term performance of structures.

Slope Stability for Temporary Excavation and Cuts

We expect the contractor will achieve the digester excavation by open-excavating to achieve the desired subgrade and stable side slopes. The large excavation for the digester will likely be constructed concurrently with site dewatering. Trench excavations are expected for pipe utilities connecting to the digester. The following discussion provides general guidelines for open and trench excavations and temporary slope stability *providing a dewatered condition* has been achieved.

We recommend all excavations, including trench construction and earthwork, be constructed according the OSHA excavation regulations, *Document 29, CFR Part 1926, Occupation Safety and Health Standards – Excavations; Final Rule*. In general, the subsurface conditions have been classified as B and C soils according to the OSHA criteria. Class B and C soil typically cannot be sloped steeper than 1H:1V (horizontal to vertical) and 1.5H:1V, respectively for excavations up to 20 feet deep. Steeper side slopes will require trench boxes or some other type of lateral support and protection. According to our interpretation of the OSHA criteria, design of excavations and/or excavation support structures for excavations deeper than 20 feet require design calculations and a report by a licensed qualified engineer submitted to OSHA. Although trench excavations can be constructed with terracing according to the OSHA criteria, it is our preliminary opinion trench excavations made at or near vertical, using available shoring technology for excavations deeper than 5 feet, will be expedient and require less construction space. If trench boxes or other means of temporary support of pipe excavations is utilized, the trench box or shoring should be of sufficient width to be able to install the pipe, pipe bedding, and provide safe and productive working conditions.

Minor sloughing of the soil represented in this report could occur for excavation side slopes, requiring appropriate maintenance and protection for workers and equipment. Localized perched groundwater subsequent to dewatering may cause local flowing soil conditions and excavation instability. If near vertical excavation for trenches is selected using sheet piling, trench boxes or other methods for temporary side slope support, caving will likely occur. The caving will cause trench boxes to become lodged, requiring additional time to remove soil debris confining the box and to move the box to a new location. Rain and other water sources will exacerbate the potential for caving and sloughing of the soils.

Foundations

Bearing Soil

STRATA previously provided recommendations to overexcavate the sandy clay soil within the entire digester footprint and extending 15 feet outside the footing. After overexcavation and replacement with granular structural fill, we expect the foundation and slab subgrade for the digester will consist entirely of granular structural fill. The foundation subgrade for the digester control building will also consist of structural fill.

Settlement Estimates

STRATA accomplished digester settlement estimates based on SPT N_{60} values, consolidation characteristics of the on-site soil, and our understanding of the critical load conditions causing settlement for the loading conditions described in the *Proposed Construction and Design Assumptions* and *Conditions Affecting Settlement* sections. Our settlement estimates assume complete removal of the sandy clay layer from 6 to 13 feet below the existing ground surface. Our analyses account for placement of granular structural fill, which has a higher unit weight than the sandy clay, and will increase the surcharge to the underlying soil. Based on the preliminary loads and foundation bearing depths and configurations, we present the following settlement estimates:

- For ring wall foundation constructed over the recommended structural fill, we estimate foundation settlement will be less than 1/4-inch between the footing and adjacent slab.
- For the center column footing bearing on structural fill, we estimate foundation settlement will be less than 1/4-inch between the foundation and adjacent slab.
- For a maximum contact pressure of 2,100 psf, settlement of the perimeter and center of the structure is estimated to be less than 1 and 2 inches, respectively.

Our foundation settlement estimates for the digester control building were less than one inch total and ½-inch differential settlement.

General

We recommend footings bear on granular structural fill placed over silty sand as described in previous report sections. If the above recommendations are followed, foundations may be designed utilizing an allowable bearing pressure of 2,500 psf. The allowable bearing pressure could be increased by 30 percent to account for transitory live loads such as wind or seismic forces. A submerged vertical modulus of subgrade reaction of 200 pounds per cubic inch can be utilized for design of slabs and pipe bedding placed in accordance with the *Structural Fill* and *Earthwork* sections of this report.

Foundations should bear a minimum of 24 inches below the finished exterior grade to reduce the potential for frost action. All foundation walls should be backfilled with drain rock and granular structural fill as shown on Plate 2, Perimeter Drain Detail and as discussed in the *Permanent Dewatering* section of this report.

Lateral Earth Pressures

All retaining and foundation wall systems should be designed to resist lateral earth pressures from the retained soil behind the structure and surcharge from equipment, slopes, or vehicles adjacent to the walls. We recommend an unfactored coefficient of friction of 0.55 be used for footing and wall design for concrete cast directly on granular structural fill.

We recommend lateral earth pressures for conventional wall systems be estimated using the following equivalent fluid pressures from Table 1.

Table 1. Coulomb Equivalent Fluid Pressures (submerged conditions)

Coulomb Lateral Earth Pressure Case	Equivalent Fluid Pressure (EFP)[1]
At rest case (no wall movement)	90 pcf[1]
Active case (wall movement away from soil mass)	75 pcf[1]
Passive case (wall movement toward soil mass)	250 pcf[2]

1-Includes soil buoyant unit weight and the unit weight of water.
2-Has been corrected for 1/2-inch of lateral deflection.

Lateral surcharge pressures due to equipment, slopes, storage loads, etc. have not been included in the above lateral earth pressure recommendations. The lateral earth pressure coefficient of 0.5, acting over the entire wall height could be used to estimate the lateral earth pressure induced on walls due to adjacent surcharge loads from equipment and the slope behind the structure. Digester walls will be subject to load influences from adjacent equipment structures and foundations.

Dynamic lateral earth pressures are a function of several factors including the presence of groundwater, magnitude of ground shaking, soil strength and soil permeability. Dynamic lateral earth pressures are additive to the above static lateral earth pressures, but act as an inverted triangle. Hydrodynamic forces also need to be considered for below-grade wall design and occur in two primary situations; 1) water "sloshing" back and forth between the soil matrix and exerting inertial forces, and 2) water being mobilized with the soil matrix as it is laterally forced against the structure. The former situation occurs in higher permeability soil, while the latter situation occurs in lower permeability soil where the soil has a tendency to experience excess pore water pressures. The degree of excess pore water pressure will impact the degree that water is taken into account for the dynamic lateral earth pressure. If complete excess pore water pressure occurs (i.e. liquefaction) the soil will act as a dense liquid and the EFP will approach the saturated unit weight of the soil during a seismic event. *Hydrodynamic* EFPs should be added to the hydrostatic forces, acting in the traditional triangular pressure distribution. Tables 2 below presents equivalent fluid pressures during dynamic loading (excludes static loads) for the saturated backfill soil, respectively. The seismic component of lateral earth pressure is assumed to have its resultant acting at 0.6 times the wall height measured from the base of the wall.

Table 2. Mononobe-Okabe Dynamic Equivalent Fluid Pressures (submerged conditions)

Coulomb Lateral Earth Pressure Case	Equivalent Fluid Pressure (EFP)
At rest case (no wall movement)	+9 pcf (submerged[1])
Active case (wall movement away from soil mass)	+6 pcf (submerged[1])
Passive case[2] (wall movement toward soil mass)	-85 pcf (submerged [1,3])
Hydrodynamic EFP[4] ($EFP_{hydrodynamic}$)	+16 pcf[5]

1 – EFP includes the buoyant soil unit weight and excludes the unit weight of water.
2 – Passive resistance has been provided for ½-inch of lateral movement.
3 – Passive resistance should be reduced by 85 pcf acting as an inverted triangle against the wall.
4 – Additive to hydrostatic fluid pressure using traditional triangular pressure distribution.
5 – Hydrodynamic EFP is specific to Nampa, Idaho, soil permeability and other site specific factors.

Care must be taken in the use of heavy equipment near the face of walls (in a zone extending 5 feet back from the wall) to avoid creating an undesirable degree of overcompaction or lateral wall loading from the soil immediately along the walls and imposing high stresses on the walls. Below-grade walls should be backfilled as described in the *Structural Fill* and *Permanent Dewatering* sections of this report.

Seismicity

We understand the 2003 International Building Code (IBC) will be utilized for structural design. Section 1615.1 of the 2003 IBC outlines the procedure for evaluating site ground motions and design spectral response accelerations. STRATA utilized site soil and geologic data and the project location to establish earthquake loading criteria at the site referencing Section 1615.1 of the 2003 IBC. Based on our field exploration and knowledge of the upper 100 feet of the soil profile, we recommend a Site Class of "D" be utilized as a basis for structural seismic design. The Maximum Considered Earthquake (MCE) maps from the 2003 IBC were referenced to develop the MCE Response Spectrum for Site Class D. The response spectrum is presented as Figure 1 below. This response spectrum assumes a 5 percent critical damping ratio in accordance with the IBC, Section 1615.1. A site-specific study was not performed. Structural design may use the spectral response at period T=0 for peak ground acceleration at the site.

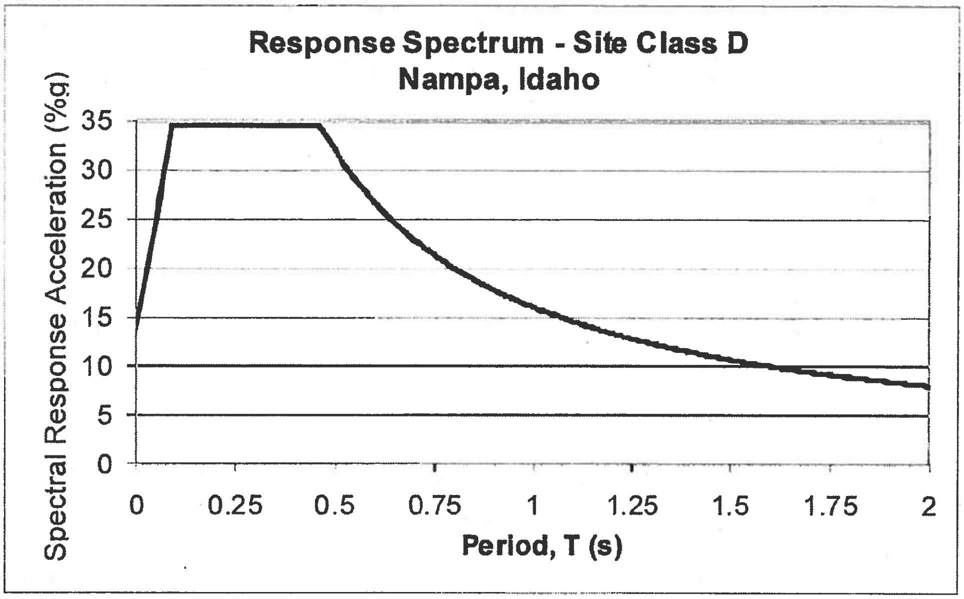


Figure 1. Spectral Response Acceleration

Permanent Dewatering

We understand it may be necessary to periodically empty the digester for cleaning or maintenance. Emptying the digester will cause an imbalance of water pressure at the base of the slab and walls due to a possible high groundwater level outside the structure. A permanent drain system may be planned to relieve this imbalance of hydrostatic pressure on the walls and floor. Based on our site exploration, hydrologic aquifer testing, and analyses, we recommend the soil around and beneath the digester be dewatered to a depth of at least 6 inches below the base of the digester slab during maintenance. Alternatively, structural design could account for uplift pressures from hydrostatic lift. If the structure will be designed to resist uplift from buoyancy, we recommend the designer assume groundwater could reach the existing ground surface.

We recommend underdrains be constructed in concentric circles starting at approximately 10 feet radiating from the center pivot and at an approximate 20 to 25-foot radial spacing. A perimeter underdrain should be located outside ring wall footings as outlined on Plate 3, Perimeter Drain Detail. The wall backfill should include a minimum of 1 foot of drain rock placed to within 4 feet of the final finished ground surface and connecting to the perimeter underdrain trench as shown on Plate 2. The drain rock must be separated from the native soil and backfill using the recommended geotextile fabric. The underdrain pipes in each trench line should be at least 6-inch-diameter perforated PVC, with perforations not exceeding ½-inch in size. The inverts of the pipes should be set at a minimum depth of 18 inches beneath the proposed base of the digester slab and be sloped a minimum of 1 percent to connect to the manifold or collection discharge pipe to remove groundwater that infiltrates the drain rock. We anticipate the perimeter underdrain and interior underdrains may be installed deeper below the digester slab to meet grades and pipe invert requirements near the center of the structure.

A minimum of 6 inches of drain rock should be placed beneath the digester slab. The top and base of the drain rock should be protected by geotextile fabric having properties discussed in the *Earthwork* section of this report. The geotextile is recommended to provide separation from both the native silty sand and the digester slab concrete to help prevent contamination and clogging of the system.

We estimate a dewatering system flow volume at the pump station of up to 100 gpm. A backup pump should also be considered for the system. Monitoring wells should also be provided such that groundwater levels can be measured to verify the static groundwater table has been drawn down before emptying the digester.

Pavement Subgrade Preparation and Section Design

We estimate traffic volumes will be less than 100,000 Equivalent Single Axle Loads (ESAL) in driveway areas (equivalent to a Traffic Index of 6.8). The pavement subgrade is anticipated to consist of either native or granular structural fill or uncontrolled fill consisting of silty gravel with sand. Correlations based on the results of our laboratory testing and our experience indicate the pavement subgrade for the proposed asphalt areas will have an estimated minimum R-value of up to 20. Factors used to design this pavement section were based on empirical data obtained through field and laboratory testing, traffic volumes reported by MWH for the proposed pavement areas near Final Clarifier No. 3 and our understanding of the use for the pavement. Our pavement design and subgrade preparation recommendations reflect these anticipated loading applications and *no construction traffic*. If subgrade conditions appear significantly different during construction, if traffic loading conditions change or traffic volumes increase, STRATA should be notified to amend our recommendations accordingly.

We recommend the pavement subgrade be compacted to structural fill requirements presented herein to a minimum depth of 12 inches. The following pavement sections are applicable to either a silty sand, clayey sand or granular structural fill subgrade. The pavement section should consist of a minimum of 3 inches of asphalt concrete underlain by at least 4 inches of ¾-inch-minus crushed sand and gravel base course placed over 8 inches of granular subbase. Alternatively, this granular support section could consist of 2 inches of ¾-inch-minus crushed gravel base course placed over 10 inches of granular subbase. We recommend STRATA verify the pavement subgrade and pavement section thickness and materials to verify our design assumptions.

We recommend the asphalt concrete should be compacted to 95 percent of the maximum density for the mix design (Marshall 50 blow) or 92 percent of Hveem mix design. Asphalt concrete should meet *Idaho Standards for Public Works Construction* (ISPWC) Class I or II asphalt design requirements. Asphalt mix designs and all appropriate aggregate source certificates should be submitted to the engineer for review at least 21 days prior to initiating asphalt paving. All pavement section aggregate and asphalt properties should conform to ISPWC requirements.

We recommend crack maintenance be accomplished on all pavement areas every three to five years to reduce the potential for surface water infiltration into the underlying pavement subgrade. Surface and subgrade drainage are extremely important to the performance of the pavement section. Therefore, we recommend the subgrade, base and asphalt surfaces slope at no less than 2 percent to an appropriate stormwater disposal system or other appropriate location that does not impact adjacent structures. The life of the pavement will be dependent on achieving adequate drainage throughout the section, especially at the subgrade, since water that ponds at the subgrade surface can induce heaving during freeze-thaw processes.

REVIEW OF PLANS AND SPECIFICATIONS

We recommend STRATA be retained to review final plans and specifications for the proposed project and assist the design team with construction submittals. STRATA will provide plan and specification review on a time and expense basis.

CONSTRUCTION OBSERVATION AND TESTING

Our opinion is the success of the proposed construction will be dependent on following the report recommendations, good construction practices and providing the necessary geotechnical construction observation, testing and consultation to verify the work has been completed as recommended. We recommend STRATA be retained on behalf of the City of Nampa to provide geotechnical observation, testing and consultation services, to verify our report recommendations and related project specifications are being followed. If we are not retained to perform the recommended services, we cannot be responsible for geotechnical-related construction errors or omissions. The recommended services are not included in this evaluation and would be performed on a time and expense basis as retained by the owner.

EVALUATION LIMITATIONS

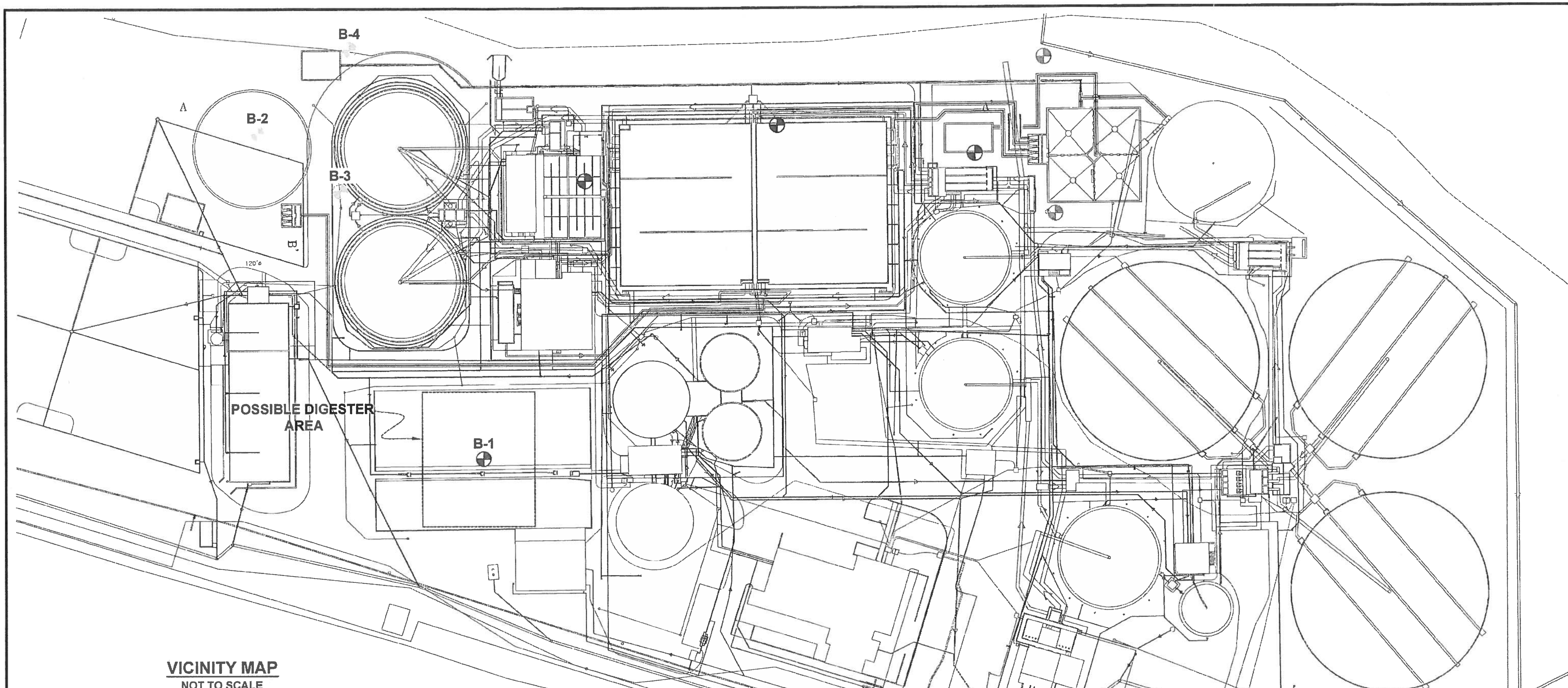
The opinions and recommendations contained herein are based on findings and observations made at the time of our subsurface evaluation. If conditions are exposed which appear to be different from those observed during our field evaluation and as described in this report, STRATA should be notified to consider the possible need for modifications to the geotechnical recommendations presented herein.

This document has been prepared to provide geotechnical information to the engineering design team. It should be understood that this report is not a document that should be used for construction planning by the contractor, but should only be used as a reference by the contractor. We recommend contractors verify the soil and hydrogeologic conditions that have been represented in this report by performing the necessary evaluation and design to obtain the data they feel are necessary to complete construction design and planning. This report shall not be used as a stand-alone tool to facilitate bids, project submittals and construction planning.

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices. This acknowledgement is in lieu of all express implied warranties.

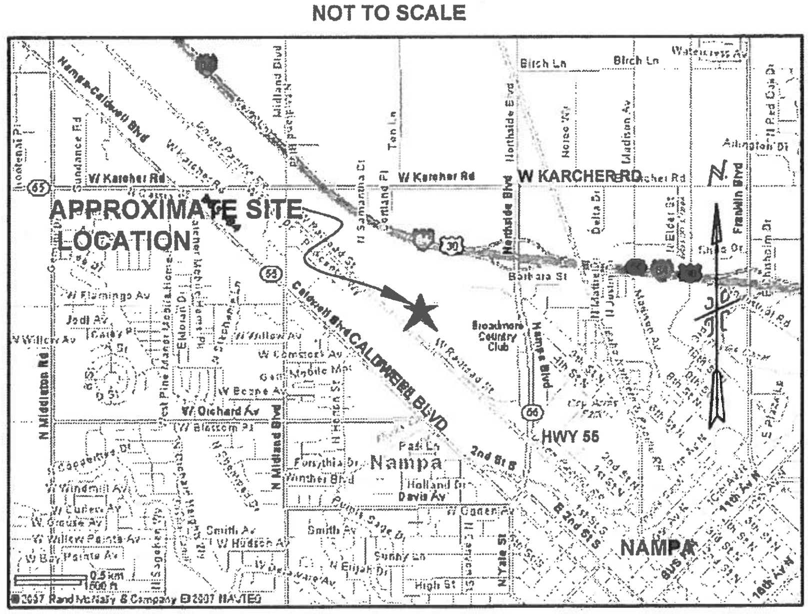
The following plates accompany and complete this report:

Plate 1:	Site Plan
Plate 2:	Consolidation Test Results
Plate 3:	Perimeter Drain Detail
Appendix A:	Exploratory Boring Log and USCS explanation



VICINITY MAP

NOT TO SCALE



LEGEND

B-1 Approximate Location of Boring by STRATA on August 3, 2007.

B-1 Approximate Location of Boring by STRATA on February 17, 2004.

B-1 Approximate Location of Boring by STRATA on October 10, 2005.

Previous Geotechnical Borings performed at the Site by Kleinfelder, 2000.

Previous Geotechnical Borings performed at the site by CH2MHill, 1979.

0 50 100 200

SCALE: 1 inch = 100 ft

N

SITE PLAN
New Digester
Nampa WWTP
Nampa, Idaho

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING
Integrity from the Ground Up

MONWAT B07179A	PLATE: 1

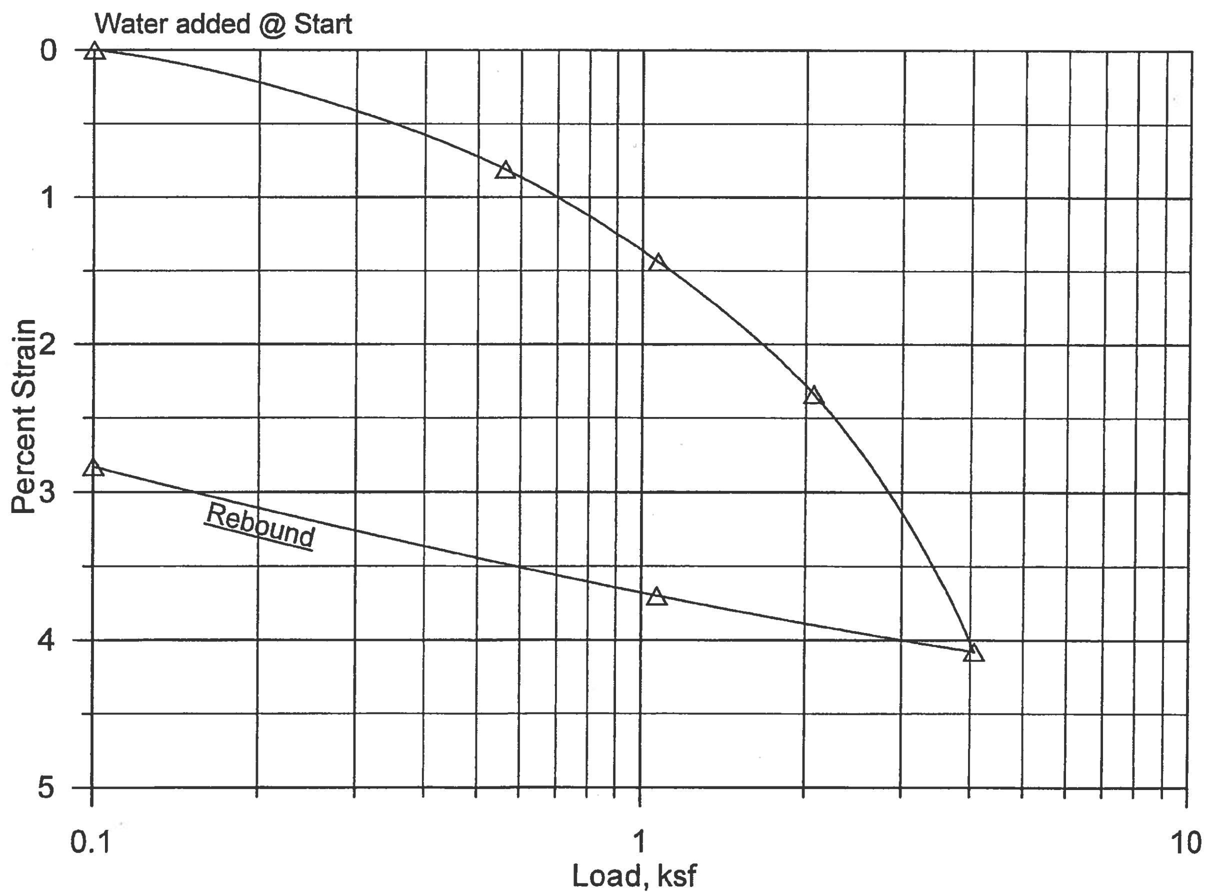
THIS PLAN COMPRISES A PORTION OF STRATA'S GEOTECHNICAL REPORT AND THE TEXT OF THE REPORT CONTAINS ESSENTIAL INFORMATION: BEFORE UTILIZING THIS PLAN FOR ANY PURPOSE WHATSOEVER, THE REPORT SHOULD BE READ COMPLETELY. THIS PLAN IS INTENDED TO INDICATE APPROXIMATE LOCATIONS OF GEOTECHNICAL EXPLORATIONS, TESTS, AND OTHER GEOTECHNICAL ISSUES (REFER TO TEXT FOR INFORMATION ON METHODS, RESULTS, AND SUBSEQUENT CONCLUSIONS AND RECOMMENDATIONS). THESE LOCATIONS AND INFORMATION WERE ADDED TO EXISTING PLANS OF THE SITE PREVIOUSLY PREPARED BY OTHERS AND NO CHECK OF ACCURACY, CURRENCY, APPROPRIATENESS, ETC., OF INFORMATION PROVIDED BY OTHERS WAS PERFORMED, SINCE SUCH CHECKS WERE NOT PART OF STRATA'S WORK SCOPE.

Reference: Site Plan Taken From File by MWH Global, "fig101A.pdf" Named "Site Plan and Topography".

CONSOLIDATION TEST RESULTS

ASTM D-2435

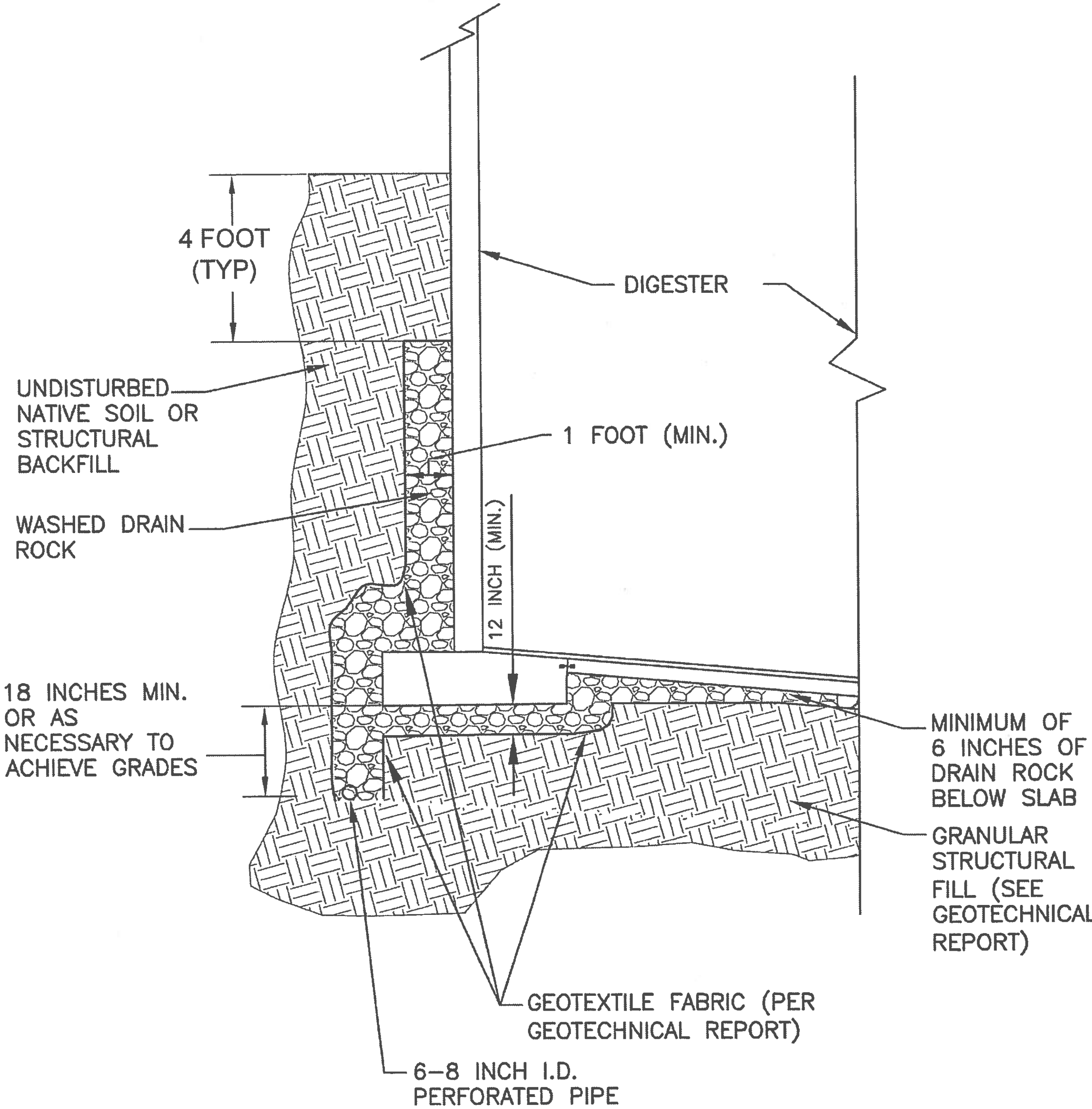
Project: New Digester
Client: MWH Global
File Name: MONWAT B07179A
Date Tested: 9/6/07 By: AM/TC
Sample Number: B7L1957E
Sample Location: B-1 @ 22 - 23'
Sample Description: Clay
Initial test conditions: In-Situ (Ring)
Moisture Content: 38.5%
Dry Unit Weight: 81.4 pcf



Reviewed By

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING
Integrity from the Ground Up

Plate 2



PERIMETER DRAIN DETAIL

STRATA

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

Integrity From the Ground Up

MONWAT B07179A	PLATE: 3

APPENDIX A

USCS Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE Type	SPT Blows Per 6 Inches	SPT N(60)	% Passing No. 200	Moisture Content (%)	POCKET Penetro-meter (tsf)	REMARKS Note: BGS = Below Ground Surface
Silty GRAVEL with Sand (Uncontrolled Fill) – brown, medium dense, moist.	1 2	GM			9 7 5	7*				No vegetation or organics observed at ground surface.
Sandy SILT (Uncontrolled Fill) – gray, soft to stiff, moist to wet.	3	ML			1 1 1	2			2.0 1.5 0.5	Slight organic smell detected.
Silty SAND (Uncontrolled Fill) – brown, loose, wet.	4	SM								
Sandy SILT (Uncontrolled Fill) – gray, soft, moist to wet.	5	ML			1 1 2	3			0.5	
Sandy CLAY (Native) – brown to tan, stiff, saturated.	6 7 8 9 10 11 12	CL			3 4 7	11	66	21.4	2.0	Atterberg Limits: PI=11, LL=32
Silty SAND – brown, dense, saturated	13 14 15 16 17 18 19	SM			10 13 16	29	15	13.9		*Corrected to 2" O.D. SPT split spoon.
Sandy CLAY – brown, soft, saturated.	20	CL								

Client: MONWAT	Boring Number: B–1	**EXPLORATORY BORING LOG**
Project: B07179A	Date Drilled: 8/03/07	STRATA
Drill Rig: BK 81	Boring Diameter: 8" AUGER	Integrity from the Ground Up
Depth to Groundwater: 5.2	Logged By: CMC	**Sheet 1 of 2**

USCS Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE Type	SPT Blows Per 6 Inches	SPT N(60)	% Passing No. 200	Moisture Content (%)	POCKET Penetro-meter (tsf)	REMARKS Note: BGS = Below Ground Surface
Sandy CLAY – brown, soft, saturated.	21	CL			0 1 2	3	93	25	0 0	Atterberg Limits: PI=17, LL=38 No sample recovery
	22 23				50psi				0.5	
Sandy Silt – brown, soft, saturated.	24 25 26	ML			3 4 4	8	70	30	0.5	Atterberg Limits: PI=3, LL=30
Boring terminated at 26.5 feet BGS.	27 28 29 30 31 32 33 34 35 36 37 38 39 40									

Client: MONWAT	Boring Number: B-1	**EXPLORATORY BORING LOG**
Project: B07179A	Date Drilled: 8/03/07	
Drill Rig: BK 81	Boring Diameter: 8" AUGER	
Depth to Groundwater: 5.2	Logged By: CMC	**Sheet 2 of 2**

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING
Integrity from the Ground Up

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UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS	GRAVELS	CLEAN GRAVELS		GW	Well-Graded Gravel, Gravel-Sand Mixtures.
				GP	Poorly-Graded Gravel, Gravel-Sand Mixtures.
		GRAVELS WITH FINES		GM	Silty Gravel, Gravel-Sand-Silt Mixtures.
				GC	Clayey Gravel, Gravel-Sand-Clay Mixtures.
	SANDS	CLEAN SANDS		SW	Well-Graded Sand, Gravelly Sand.
				SP	Poorly-Graded Sand, Gravelly Sand.
		SANDS WITH FINES		SM	Silty Sand, Sand-Silt Mixtures.
				SC	Clayey Sand, Sand-Clay Mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50%			ML	Inorganic Silt, Sandy or Clayey Silt.
				CL	Inorganic Clay of Low to Medium Plasticity, Sandy or Silty Clay.
				OL	Organic Silt and Clay of Low Plasticity.
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%			MH	Inorganic Silt, Micaceous Silt, Plastic Silt.
				CH	Inorganic Clay of High Plasticity, Fat Clay.
				OH	Organic Clay of Medium to High Plasticity.
				PT	Peat, Muck and Other Highly Organic Soils.

BORING LOG SYMBOLS	GROUNDWATER SYMBOLS	TEST PIT LOG SYMBOLS
Standard 2-Inch OD Split-Spoon Sample	▼ Groundwater After 24 Hours	BG Baggie Sample
California Modified 3-Inch OD Split-Spoon Sample	(7-3-07) Indicates Date of Reading	BK Bulk Sample
Rock Core	▽ Groundwater at Time of Drilling	RG Ring Sample
Shelby Tube 3-Inch OD Undisturbed Sample		

Shorthand Notation:
BGS = Below Existing Ground Surface
N.E. = None Encountered

STRATA
Integrity from the Ground Up

MONWAT B07179A

Strata Geotechnical Engineering Evaluation – December 6, 2005

Integrity from the Ground Up

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

STRATA

REPORT
Geotechnical Engineering Evaluation
Proposed Primary Clarifier No. 3
Nampa Waste Water Treatment Facility
Nampa, Idaho

Prepared by
Chris M. Comstock, E.I.T., P.G.
H. Robert Howard, P.E.

Prepared for
Mr. Lawrence Bennett, P.E.
MWH Global
671 E. Riverpark Lane, Ste 200
Boise, ID 83706

STRATA, Inc.
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December 6, 2005

IDAHO MONTANA NEVADA OREGON UTAH WASHINGTON WYOMING
www.stratageotech.com

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING
Integrity From the Ground Up

December 6, 2005
File: MONWAT B05202A

Mr. Lawrence Bennett, P.E.
MWH Global
671 E. Riverpark Lane, Ste 200
Boise, ID 83706

RE: **REPORT**
Geotechnical Engineering Evaluation
Proposed Primary Clarifier No. 3
Nampa Waste Water Treatment Facility
Nampa, Idaho

Dear Mr. Bennett:

STRATA, Inc. is providing the authorized geotechnical engineering evaluation for the proposed Primary Clarifier No. 3 at the Nampa Waste Water Treatment Facility in Nampa, Idaho. Our work was accomplished referencing our proposal dated September 23, 2005. The accompanying report summarizes the results of our field evaluation, laboratory testing and analyses, and presents our geotechnical and hydrogeologic engineering opinions and recommendations. Based on our field evaluation and subsequent analyses, it is our opinion the site is suitable for the project from a geotechnical standpoint, provided the recommendations presented herein are implemented for design and construction.

It is our opinion that six key geotechnical and construction issues exist at the site. We have discussed and addressed these issues within the attached report. The key issues include:

- Two distinct aquifer systems exist at the site;
- Dewatering in advance of excavation construction must occur;
- Localized dewatering as excavation for the clarifier occurs will be required;
- Design for long-term dewatering and structural design of the clarifier station floor should account for the anticipated differences in water elevation in the clarifier and the groundwater elevation;
- The selected dewatering contractor must be experienced in similar dewatering applications;
- Clarifier foundation soil disturbance, due to water issues or inappropriate equipment use, will affect construction and increase the potential for differential foundation performance and for the need to remediate the disturbed soil.

This report presents our evaluation and assessment of the hydrogeologic conditions and provides recommendations or suggestions for approaching site dewatering. Also, the opinions and recommendations reflect our experience for the Final Clarifier No. 3, which is currently under construction. Our presentation provides preliminary estimates for pumping rates and duration for assumed dewatering approaches based on our interpretation of the hydrogeologic conditions. The contractor may review or use these options, but should not rely on the information in planning for and design of site dewatering and earthwork in wet soil conditions. We recommend the contractor conduct independent site evaluations and any additional hydrogeologic analyses they feel are required for final planning and design of their construction, including the dewatering approach.

The success of the proposed construction will, in part, depend on following the report recommendations and utilizing good construction practices. Also, we respectfully recommend that the City of Nampa retain STRATA to provide geotechnical testing and consultation services during construction to verify our report recommendations are followed, and provide input as site conditions vary. It has been our experience that maintaining continuity with the geotechnical consultant of record helps reduce soil and construction related errors and contributes to overall project success and economy.

We appreciate the opportunity to continue our relationship with MWH and the City of Nampa. Please contact us if you have any questions or further requirements.

Sincerely,
STRATA, Inc.

Chris M. Comstock, E.I.T., P.G.
Project Engineer & Professional Geologist

REGISTERED PROFESSIONAL ENGINEER
3339
STATE OF IDAHO
H. ROBERT HOWARD

H. Robert Howard, P.E.
Senior Engineer

CMC/HRH/nl

REPORT
Geotechnical Engineering Evaluation
Proposed Primary Clarifier No. 3
Nampa Waste Water Treatment Facility
Nampa, Idaho

PREPARED FOR:
Mr. Lawrence Bennett, P.E.
MWH Global
671 E. Riverpark Lane, Ste 200
Boise, Idaho 83706

PREPARED BY:
STRATA, Inc.
8653 West Hackamore Drive
Boise, Idaho 83709
(208) 376-8200

December 6, 2005

TABLE OF CONTENTS

REPORT
Geotechnical Engineering Evaluation
Proposed Primary Clarifier No. 3
Nampa Waste Water Treatment Plant
Nampa, Idaho

INTRODUCTION

This report presents the results of our geotechnical engineering evaluation for the proposed Primary Clarifier No. 3 to be located at the existing Nampa Waste Water Treatment Plant (WWTP), in Nampa, Idaho. The approximate location of the Nampa WWTP is shown on Plate 1, Vicinity Map.

The purpose of our evaluation was to characterize the subsurface soil and hydrogeologic conditions in order to prepare geotechnical and hydrogeologic opinions and recommendations to be used for civil engineering design and preparation of construction drawings and specifications to assist final planning, design and preliminary dewatering criteria for constructing the proposed Primary Clarifier No. 3. We anticipate the report could be used by bidding contractors to help plan, schedule and estimate project costs, but should not be relied upon by the contractor to complete their final dewatering and excavation design and planning for construction.

To accomplish this evaluation, we performed the following services:

1. Reviewed data from evaluations for existing structures at the WWTP and reviewed preliminary drawings for the current project.

2. Met with Montgomery Watson Harza (MWH) staff and performed WWTP site visits to gain additional familiarity with the project.

3. Coordinated with Digline and WWTP personnel to avoid existing utilities at the site. We also coordinated with WWTP personnel to delineate cleanup expectations, site access and safety requirements at the plant.

4. Completed one auger boring as a pumping well, one boring as an observation well, and conducted a groundwater aquifer test. Aquifer test data were analyzed to evaluate the characteristics of the drawdown curves and to estimate aquifer transmissivity.

5. Reduced and analyzed aquifer test data to evaluate hydraulic conductivity, potential boundary conditions and estimated hydrogeologic properties of the

aquifer. These coefficients were utilized to evaluate potential dewatering options and to provide construction recommendations.

6. Completed engineering and hydrogeologic analyses, using computer software systems, including AQTESOLV and WinFlow, and hand performed calculations to help evaluate preliminary dewatering methods and configurations.

7. Performed analyses and prepared geotechnical recommendations for foundation bearing soil, allowable bearing pressure, lateral earth pressures, excavation characteristics, temporary excavations, structural fill and earthwork, seismicity, pavement design and specific construction criteria associated with the above civil and geotechnical items.

8. Provided this draft report.

PROPOSED CONSTRUCTION

We understand a new clarifier is planned east of the headworks, near the main entrance of the plant. The clarifier is planned to be approximately 120 feet in diameter and will extend to about 17 feet below the existing ground surface at the perimeter, and 22 feet below the ground surface at the center. The sludge hopper and piping to and from the clarifier will extend to a depth of about 27 to 28 feet below the existing ground surface.

SITE EVALUATION

STRATA subcontracted the installation of two borings at the proposed clarifier on October 10, 2005. Exploration locations are presented on Plate 2, Site Plan and were documented by taping and pacing from existing site features, with input from Mr. Curtis Deveny with the City of Nampa. One boring was advanced using a CME-75 drill rig equipped with 6-inch, outside-diameter (2-inch monitoring well) and 8-inch, outside-diameter (4-inch pumping well) hollow-stem augers. The soils encountered in the borings were evaluated and logged in the field by a hydrogeologist referencing the Unified Soil Classification System (USCS). A brief explanation of the USCS is presented on Plate 3. The USCS should be used to interpret the terms on the boring logs and throughout this report. Boring logs are presented in Appendix A of this report.

Soil samples were obtained in the 8-inch-diameter boring at 5-foot intervals using either a 2-inch (outside-diameter) split-spoon or a 3-inch (outside-diameter) ring sampler. Standard Penetration Test (SPT), N_{60} values were recorded for each sample. N_{60} values were obtained by counting the number of hammer blows required to advance the 18-inch-long samplers from 6 to 18 inches. The SPT blow counts for each 6-inch segment of the sampler are presented on the boring logs. SPT blow counts have been corrected below depths of 10 feet for effective overburden pressure resulting in an $N_{1(60)}$ value. SPT values obtained from a 3-inch ring sampler have been corrected for diameter and normalized to a 2 inch, split-spoon sampler and the perspective $N_{1(60)}$ values presented. SPT values can provide an indication of the relative density or consistency of the soil sampled, and are utilized for soil engineering strength and liquefaction analyses.

Boring B-1 was installed as a pumping well with 4-inch-diameter PVC casing. The well was installed to an approximate depth of 26.5 feet below the existing ground surface and included about 16.5 feet of screened casing. The screened interval included Colorado sand as a sand pack to assist well development. The pumping well was developed for approximately 1.5 hours using a small submersible DC pump. The upper 10 feet of the well was sealed with granular bentonite in general accordance with Idaho Department of Water Resources (IDWR) requirements. This well was permitted through IDWR and can reportedly be lawfully utilized to assist the dewatering program during construction.

A monitoring well was installed in B-2 and was constructed with about 10 feet of screened, 2-inch-diameter PVC casing and 5 feet of granular bentonite.

General Site Conditions and Geology

The site is located east of the WWTP headworks at an existing grass-covered area. The generalized project geology, based on our current and past fieldwork, and review of geologic references, is alluvial silty sand, sandy silt, clay and sand. Although the borings did not encounter basalt, our exploration database shows basalt bedrock is typically encountered between 40 and 50 feet below the existing ground surface. The alluvial soil encountered during exploration is associated with the depositional environment of Indian Creek, which trends to the northwest. The alluvial creek system has the potential for small-scale soil variability in short horizontal and vertical distances. Ancient buried stream channels and flood deposits are likely within the upper 50 feet of the subsurface profile.

Based on the compiled subsurface and historic data (presented in the *Discussion* section), it is our opinion the thickness and lateral extent of the alluvial soil layers are relatively consistent beneath the plant. Isolated sand and gravel lenses exist that will influence the hydrogeologic characteristics of this site. The specific soil types encountered during this evaluation appear to vary slightly across the site with general depositional trends being similar as mentioned above.

Subsurface Conditions

The soil conditions observed in the borings at the proposed clarifier were similar to other borings completed at the site. However, the elevation of the contact between layers could vary across the site. The following discussion delineates the soil profile in borings B-1 and B-2. Specific layer contacts and geotechnical data can be referenced to the boring logs in Appendix A.

Borings B-1 and B-2

The borings encountered grass sod underlain by 7 to 8 feet of silt, which we have evaluated as native soil. The silt was tan, firm to stiff and moist to saturated. Silt with sand was encountered below the silt. Silt with sand was brown, medium dense and saturated. The silt with sand extended to approximately 11 feet in both borings, where brown, medium dense and saturated silty sand was encountered. This soil layer extended to 20 feet in B-1 and to the termination depth of 15 feet in B-2. At 20 feet, B-1 encountered lean clay with sand that was brown, soft to stiff, saturated, and extended to approximately 26 feet below the existing ground surface. Tan, medium dense and saturated poorly-graded sand with silt was encountered below the lean clay with sand. The poorly-graded sand with silt extended to at least the termination depth of exploration at 26.5 feet.

Groundwater Conditions

STRATA encountered two aquifers in B-1. An upper aquifer exists above the clay layer and a lower aquifer exists below the clay layer. The WWTP project has a history of artesian pressure, as we presented in our 2004 report for the current Final Clarifier No. 3

and RAS Pump Station, and as encountered during construction for this project. It is our opinion the effect of the lower aquifer may be observed at any depth below or within the clay layer, which typically extends to about 26 feet below the ground surface.

The upper static groundwater level above the clay layer was observed about 6.5 feet below the existing ground surface. The upper aquifer is assumed to extend to the top of the clay layer at about 20 feet below the ground surface. The lower aquifer was encountered at about 26 feet and the groundwater level raised to about 3 feet below the ground surface, as measured in the boring prior to constructing a monitoring well in the boring. After the rise in groundwater was recorded, a bentonite hole plug was used to plug the lower aquifer from 26.5 feet to 20 feet below the ground surface. A monitoring well was then constructed in the remainder of the boring.

Aquifer Field Testing

To gain hydrogeologic data to supplement existing data and perform preliminary dewatering design, an aquifer pump test was accomplished within the upper aquifer, utilizing borings B-1 and B-2. A 36 gallon per minute (gpm) submersible pump was utilized in boring B-1 to discharge water from the well. Solinst™ Levelogger pressure transducers were installed in each well to monitor groundwater drawdown during the aquifer test. An electric water level indicator was also utilized to field-check pressure transducers and for groundwater static level measurements and monitoring. Groundwater was discharged to an approved stormwater discharge location. Discharge quantities were monitored using a 5-gallon bucket with measured intervals using a stopwatch timer.

The test was initiated on October 12, 2005. The 36 gpm pump discharge was throttled to about 0.5 gpm and was set at a depth of 20 feet below the existing ground surface. The test was performed for approximately one and a half days. Drawdown was measured in the pumping well, B-1, and monitoring well B-2. The groundwater level in the pumping well and monitoring well experienced drawdown of approximately 5 and 0.58 feet, respectively.

Laboratory Testing

Select soil samples were tested to assess Atterberg limits, in situ dry density and moisture content, and grain-size distribution. Laboratory testing was accomplished referencing ASTM standards. Laboratory test results are presented on the exploratory boring logs.

Additionally, we referenced the laboratory and N-value data from the 2004 report, since our interpretation of the engineering and physical properties of the soil at the two sites is that they are similar. The 2004 laboratory pH and resistivity testing indicate the soil has a moderate corrosion potential.

HYDROGEOLOGY

The following report sections discuss our approach to develop dewatering options and to help identify and characterize the hydrogeologic conditions at the site. The dewatering options presented in subsequent sections are not provided as specific hydrogeologic recommendations to be used for final construction dewatering planning. The dewatering options are presented to allow the contractor and the design team to evaluate the characteristics and limitations of several dewatering options. Site dewatering should reflect a well-planned, practical approach implemented by the contractor.

We have previously discussed the hydrogeologic conditions at the site in the *Groundwater Conditions* section. Groundwater from the upper aquifer can generally be encountered from 6.5 to 20 feet below the existing ground surface. The clay layer acts as a confining layer, and the lower aquifer was encountered approximately 26 feet below the existing ground surface.

Aquifer Testing and Analyses

Aquifer test data from the upper aquifer were used to develop time-drawdown curves for the observation well and the pumping well. Well construction, pumping rates, subsurface aquifer geometry, and well spacing were documented to facilitate hydrogeologic analyses. Aquifer test data were input into the aquifer testing software AQTESOLV for analysis. The Cooper-Jacob (1946) method was used to estimate transmissivity of the upper aquifer. The short duration of the aquifer test did not allow for valid estimates of specific yield (storativity). Transmissivity is defined as permeability or soil hydraulic conductivity times the saturated thickness of the aquifer. Transmissivity of unconfined aquifers will vary as groundwater levels are decreased. Based on the transmissivity estimated from aquifer testing and measured saturated thickness, a range of hydraulic conductivity values were back-calculated for each analysis. Hydraulic conductivity is a measure of a soils ability to permit water flow under a hydraulic gradient. Hydraulic conductivity is a vital parameter in construction dewatering analyses. STRATA also utilized the subsurface geometry and soil conditions to calibrate our model. Known boring locations, pumping rates and knowledge of well construction were utilized to refine estimates of hydraulic conductivity of the upper aquifer.

Our preliminary analyses indicate the hydraulic conductivity of the upper aquifer for preliminary design will be 6.5×10^{-5} to 1.6×10^{-4} feet per second (2×10^{-3} to 5×10^{-3} centimeters per second). The above hydrogeologic parameters should not be solely relied upon by the contractor. The dewatering system designer must evaluate the hydraulic conductivity and dewatering characteristics of both aquifer systems to facilitate a successful dewatering design. STRATA did not provide aquifer test results due to the potential for misinterpretation of the data. The raw data is available for review upon request, contingent upon STRATA's participation in data interpretation.

Several assumptions and analytical methods were employed to help simplify the complex system, so preliminary dewatering options could be evaluated. Complex analytical or numerical modeling is not appropriate without additional hydrogeologic data. However, gathering additional hydrogeologic data for the site for this preliminary evaluation would be time consuming and costly.

Dewatering Issues

Due to the nature of hydraulic conductivity of the soil, and based on our experience with the current project at the WWTP site, dewatering will require a significant amount of time. Further, based on subsurface geology and the proposed construction depths, the entire upper aquifer above the clay layer must be dewatered to allow construction to occur. The silty sand encountered at 11 feet in B-1 will have capillary rise as the water table drops, due to dewatering of this soil. The estimated capillary rise will be on the order of 2 to 7 feet. We believe this condition affected the dewatering and subgrade preparation time at the current clarifier project. As such, the silty sand will maintain a saturated condition, making the clarifier foundation preparation difficult to prepare at the foundation depth (14 to 17 feet) of the clarifier.

As discussed above, the capillary rise in the silty sand may not allow for a dewatered subgrade to be achieved. The options discussed below are not presented as dewatering options; rather, they are two reasonable options to achieve a stable subgrade. Dewatering options are discussed in subsequent report sections. The first option limits overexcavation beneath the clarifier to a depth necessary to create a stable subgrade. The second option assumes the native soil is too wet (saturated) to permit a stable subgrade in the silt/sand. Two options to achieve a stable subgrade are as follows:

- Accomplish dewatering to a minimum depth of 2 feet below the clarifier subgrade (bottom of blanket drain). Due to capillary rise, the subgrade will likely remain in a saturated condition. Based on our experience on the current WWTP project, the silty sand at the subgrade elevation may sufficiently dry out, due to warm weather and evaporation, to permit foundation construction. The exposed soil surface must be open for a sufficient time to allow the subgrade to stabilize by forming a surface "crust" due to drying of the soil. However, isolated subgrade disturbance from construction equipment should be expected. It is our opinion this option may require a significant amount of time to achieve a stable subgrade to allow construction to proceed. Using this approach, at least 1 foot of granular structural fill should be placed over the exposed, dry native, undisturbed silt/sand. The structural fill is intended to provide a stable construction surface and to protect the more easily disturbed silt/sand.

- Accomplish dewatering to the top of the clay layer encountered at 20 feet. The silty sand will still be saturated to near saturated due to capillary rise. To establish a stable subgrade, the saturated silty sand can be overexcavated to undisturbed clay encountered at 20 feet and replaced with granular structural fill to the subgrade elevation. A perimeter drain could be installed to intercept lateral flow from the adjacent native soil to facilitate excavation and fill construction.

The options discussed above are presented as a method to achieve a stable subgrade and are not dewatering options. The considerations for aquifer dewatering presented in subsequent sections are based on recent aquifer field testing as well as our experience with dewatering for the Final Clarifier No. 3 and RAS Pump Station, which is ongoing approximately 1,000 feet to the west of the proposed project. Our May 18, 2004 geotechnical evaluation for the Final Clarifier No. 3 and RAS Pump Station identified the potential for artesian influence from the lower aquifer to influence dewatering of the upper aquifer. Excavation heave was identified as a possible concern for the large excavation. However, to our knowledge, significant heave during excavation for the Final Clarifier No. 3 and RAS Pump Station did not occur. Some influence from the lower aquifer was realized as an increased flow and pumping rate in the clarifier excavation, according to the site contractor. Since the proposed Primary Clarifier No. 3 will be shallower than the Final Clarifier No. 3, the dewatering considerations provided in this report assume excavation heave will not occur, and foundation disturbance due to seepage and construction equipment can be adequately mitigated.

It is our opinion a well planned, practical dewatering approach is the most critical part of the proposed project. A specific dewatering design will be required that incorporates the construction schedule, groundwater levels during construction, the method(s) of dewatering and other construction specific considerations. The dewatering criteria presented in subsequent sections may be referenced but should not be relied upon by the contractor to develop a specific dewatering plan.

Assumptions

To provide the preliminary dewatering options, it was necessary to anticipate the construction approach, schedule, possible dewatering methods and anticipated hydrogeologic conditions at the time of construction. The assumptions may not be valid for the contractor's specific dewatering approach or schedule. We have assumed an unconfined, homogeneous, isotropic upper aquifer of infinite area with an impermeable clay layer at the upper aquifer base (separating the upper and lower aquifer), resulting in about 13.5 feet of saturated thickness in the upper aquifer. The aquifer test analyses assume fully penetrating wells within the upper aquifer, pumping at a constant rate with no well losses. Additional assumptions were made associated with the analytical methods used and, in our opinion, are appropriate given that some aquifer characteristics could not be verified. The costs associated with more complicated hydrogeologic analyses are not justified for the preliminary dewatering analysis presented herein.

Dewatering Options

It will be the contractor's responsibility to develop a specific dewatering approach that reflects their capabilities, equipment, schedule and construction approach. We recommend the contractor's specific dewatering plan be submitted to the City for review and comment. Further, we recommend project specifications reflect the requirement for the contractor to submit a dewatering plan to MWH and the City for approval. The dewatering plan should account for initiating dewatering well in advance of construction. Initiation of dewatering should be incorporated into the construction schedule. The excavation portion of the construction schedule could be accelerated if the contractor proves groundwater has been drawn down to the necessary levels prior to initiating excavation.

Two dewatering approaches have been identified that, in our opinion, are suitable for this project: the first is a system of well-points around the perimeter of the structure; and the second is a perimeter, open excavation drain that completely surrounds the clarifier. The following sections present these options to assist the contractor in developing a specific plan and design for site dewatering. The contractor's specific dewatering plan should consider the potential for seasonal fluctuation in precipitation, irrigation, infiltration and infrastructure additions to the project site. Further, site conditions will affect dewatering, including variations in subsurface geology, the artesian pressure at the site resulting from disturbance at the base of excavations and an upper aquifer groundwater gradient sloping toward Indian Creek.

Well-point Option

Closely spaced well-points are one option to dewater the upper aquifer. Well-points could be drilled or driven into the soil to the top of the clay layer encountered at 20 feet. Wells should not extend more than 1 foot into the clay layer. If wells completely penetrate the clay layer, the lower aquifer will infiltrate the upper aquifer, and pumps installed within the wells will not dewater the upper aquifer.

During our aquifer test the pumping well was limited to a yield of less than 1 gpm. It is our opinion well-points may not yield more than this rate, and thus, must be closely spaced to accomplish site dewatering in a reasonable amount of time.

Establishing a successful well-point dewatering program will be contingent on well spacing and construction pumping rates. Poor well construction has the potential to limit pumping rates. Further, water production may be reduced as groundwater is drawn down, and transmissivity decreases. Actual pumping rates will be controlled by the saturated thickness near the well and variations in hydraulic conductivity expected in the alluvium, which will be reflected by the actual number of wells needed, and the pumping requirements.

Total dewatering rates for the upper and lower aquifer have the potential to vary significantly. Total excavation dewatering for the upper aquifer may range between 15 and 50 gpm, assuming no influence from the lower aquifer. The estimated low pumping rates, particularly in the upper aquifer, are related to the soil transmissivity and our experience on the Final Clarifier No. 3 project. The contractor should consider installing two to four wells into the lower aquifer to reduce the impact of artesian pressure to the clarifier excavation. Reducing the lower aquifer's influence may allow faster dewatering of the upper aquifer and affect capillary rise, lower pumping rates and reduce seeps and springs in the excavation.

The schedule for dewatering the upper aquifer using well-points is fully contingent upon the contractor's selected well spacing, well pump volumes, well construction, and other construction approaches to dewatering. For example, STRATA preliminarily modeled 12 well-points around the outside of the clarifier, each extending to the top of the clay layer. The well-point pumping rates were modeled at approximately 1.5 gpm each and achieved approximately 90 percent of the required drawdown in 20 days, assuming no artesian influence and that the dewatering soil will have capillary rise.

Perimeter Excavation Drainage

This method of dewatering refers to the approach that was utilized by the contractor for the Final Clarifier No. 3 and RAS Pump Station dewatering approach. The excavation contractor utilized a combination of several methods to dewater the aquifer. A few large excavations were accomplished around the two structures using trench boxes or "glory holes." Some depression of the groundwater surface was accomplished and then the contractor began mass excavating for the two structures. The mass excavation was dewatered locally along the perimeter by using drain rock and 4-inch, outside-diameter perforated pipe sloped to a sump pit. Some flowing soil and excavation instability was observed along the perimeter of the excavation.

The RAS Pump Station was unable to be dewatered within 30 days, and the contractor and the City of Nampa opted to overexcavate the saturated sand at the subgrade into the underlying clay soil and replaced the excavation with granular pit run. The granular pit run intercepted groundwater flow from the sides of the excavation. A 4-inch, outside-diameter, perforated pipe was installed around the perimeter of the excavation to remove water from the pit run below the RAS foundation.

The clarifier excavation was dewatered with a perimeter drainage system and large glory hole at the center of the excavation. The contractor communicated that the glory hole may have caused heave of the lower aquifer and a significant portion of pumped water at the center could be attributed to the lower aquifer, which is discussed in more detail in the *Site and Subgrade Preparation* and *Excavation Characteristics* section on this report. Several seeps and isolated lenses of granular material within the slope of the excavation allowed groundwater to flow over the clarifier subgrade. Dewatering was accomplished after 60 to 90 days of an iterative dewatering and excavation approach, which allowed underdrain construction to begin. Underdrains for the clarifier floor dewatering system were also used to assist dewatering of the excavation.

It is our opinion this dewatering approach may be suitable to accomplish dewatering for the new Primary Clarifier No. 3. We recommend the perimeter drain extend to the top of the clay layer to intercept lateral seepage. The perimeter drain should completely surround the clarifier excavation and drain positively to a sump pit. It may be necessary to install one or more isolated well-points or glory holes at the center of the clarifier excavation to accelerate the dewatering schedule.

One drawback to this iterative dewatering approach is that the perimeter drain must be excavated below the groundwater table. This will result in flowing soil, sand boils and other excavation instability, as was observed in isolated locations for the Final Clarifier No. 3 excavation. Total steady state pumping rates were estimated from the Final Clarifier No.

3 excavation at between 1,000 and 1,500 gpm. It is our opinion the Primary Clarifier No. 3 excavation will produce similar pumping volumes.

Drawdown Verification

Monitoring of groundwater levels will be required prior to initiating excavation. This may be accomplished using existing wells and piezometers or by installing piezometers specifically to verify the required drawdown has been achieved. Monitoring of drawdown is critical to construction timing and to help plan against excavation instability, including flowing soil and sand boils. Reusable hand driven piezometers are available from several manufacturers. These piezometers can be driven with standard T-post drivers and are available with continuous electronic monitoring systems. We recommend the contractor's dewatering plan outline the methods they will use to verify groundwater levels prior to initiating excavation. We recommend the contractor establish a groundwater discharge location that does not conduct water to the site groundwater system, and meets regulatory agency requirements.

Impact to Adjacent Structures

STRATA evaluated the potential impact to adjacent structures that could occur due to site dewatering. As groundwater is drawn down, buoyancy effects on the soil will be eliminated and effective stress will increase in the foundation soil. Assuming up to 14 feet of drawdown, the maximum effective stress increase to the soil will be approximately 875 pounds per square foot (psf). Assuming a stress increase of 875 psf to the entire soil profile (worst case condition), and using the SPT data obtained during exploration, we estimate additional foundation settlement of any structure adjacent to the new clarifier construction at any depth will be less than ¼ inch.

Construction Dewatering Criteria

We recommend the City of Nampa and MWH incorporate the following dewatering criteria into the plans and specifications for the project.

1. The contractor must submit a dewatering plan for review by the City. Approval of the plan by the City will not alleviate the contractor from assuming full responsibility for their plan.

2. Dewatering the upper aquifer should be initiated a minimum of 60 days prior to initiating earthwork construction at the project.

3. The contractor must coordinate and plan a groundwater discharge location that meets all regulatory agency requirements and does not allow discharged groundwater to infiltrate the upper aquifer.

4. Well-points or dewatering locations within the upper aquifer must be spaced no further than 20 feet on center.

5. The sides of the excavation for the clarifier must be completely dewatered at all times, to increase worker safety and reduce the potential for slope failure due to hydrostatic pressures from groundwater.

6. Standpipe piezometers must be installed within the upper aquifer to verify drawdown has occurred for the clarifier excavation prior to initiating excavation. Excavation can commence and continue no closer than 2 feet above the static groundwater table as dewatering commences.

7. STRATA should be retained to verify dewatering has occurred prior to excavation and prior to placing structural fill or concrete at the foundation bearing elevation.

Dewatering Plan Aspects

We recommend the contractor provide, at a minimum, the following criteria as part of their dewatering plan. The dewatering plan should include:

1. Location and depth of all planned well-points or dewatering facilities for the upper aquifer.

2. Well construction or dewatering facility construction details.

3. Method to verify drawdown has occurred to at least the top of the clay layer.

4. Groundwater discharge locations and appropriate permits from regulatory agencies.

5. Timeline of dewatering schedule versus excavation and construction.

6. Operations and maintenance plan for dewatering systems to show that dewatering systems can be effectively operated during all aspects of clarifier construction.

7. Show that all wells or dewatering systems have been designed and constructed in accordance with IDWR regulations.

8. Name and contact information of the contractor's representative responsible for maintaining and operating the dewatering system during all aspects of lift station construction.

Hydrogeologic Summary

In summary, a specific dewatering plan must be developed by the contractor based on the location and configuration of site improvements. Further, we recommend a minimum of 2 feet of the saturated silty sand be overexcavated and replaced with at least 1 foot of granular structural fill overlain by at least 1 foot of drain rock. This will help to reduce the effect of capillary rise from the silty sand and to accelerate the construction schedule with respect to dewatering. We consider trench drains and closely spaced well-points the most viable alternatives for dewatering the upper aquifer, but not the only alternative. We consider a large-diameter well as the most viable alternative for dewatering of the lower aquifer. However, the selected contractor must evaluate the site conditions, potential dewatering options and considerations relative to their dewatering design and construction approach. The contractor should submit the dewatering plan to the design team at least four weeks prior to initiation of the excavation.

DISCUSSION

Anticipated Use of Report Recommendations

The report findings and the recommendations have been prepared to assist planning and civil design of the proposed project. Specifically, preliminary dewatering options outlined above are contingent upon detailed hydrogeologic and construction assumptions stated in this report. Consequently, the bidding contractors should only use the report information as a reference. We recommend they complete a more thorough evaluation of the subsurface and hydrogeologic conditions that they determine necessary to bid the project.

Research

Prior evaluations were performed by STRATA as follows:

- *Geotechnical Engineering Evaluation, Nampa Wastewater Treatment Plant, Clarifier No. 3 Reconstruction, Nampa, Idaho, June 24, 1998;*

- *Geotechnical Engineering Evaluation, Proposed Clarifier and RAS Pump Station, Nampa Wastewater Treatment Facility, Nampa, Idaho, May 18, 2004.*

The design and construction files for the above projects were reviewed and data and calculations were utilized for this project. Other references utilized to assist our report preparation include:

- **Boring Location Plan:** *WWTP City of Nampa, Idaho Contract 5; January 19, 1979, by CH_2M-Hill.*
- **Project Memorandum:** *Existing Dewatering System for the Proposed New Clarifier, Nampa WWTP, July 19, 2001, by Mr. Larry West*
- **Specification Section 02200-Earthwork:** *October 30, 2000, by HDR Engineering.*
- **Draft Report:** *Groundwater Dewatering Model, Nampa WWTP, October 5, 2000, by Kleinfelder, Inc.*
- **Discussion:** *Information provided by MWH regarding construction for the aeration basin and clarifiers for the 1981 to 1982 construction.*

Key Design and Construction Issues

The soil and hydrogeologic conditions at the site will require specific dewatering and construction procedures to facilitate a successful project. The following items, in our opinion, should be addressed or discussed as part of project planning, design, and construction:

- **Two distinct aquifer systems exist at the site.** We have discussed the conditions encountered during exploration and reiterate the need for dewatering applications to consider both aquifer systems. The systems act independently of each other, but will have increased interaction as excavation toward the clay layer occurs. Excavation heave is not considered likely, but seepage from the lower aquifer is anticipated and may cause sand boils, excavation sidewall instability and possible flooding of the excavation area if the clay layer is penetrated.
- **Dewatering in advance of excavation construction is critical.** The hydraulic conductivity measured at the site and preliminary hydrogeologic analyses indicate dewatering of the upper and lower aquifer will require a significant quantity of time. It is our opinion that the upper groundwater system must be drawn down to sufficient levels as discussed in this report to facilitate excavation. Dewatering of the lower system is not considered to be necessary to achieve dewatered conditions, but the lower system will influence dewatering.

Dewatering the site should occur prior to initiating excavation to reduce the potential for excavation instability and foundation disturbance. The contractor has the option of excavating as the groundwater is being actively drawn down or achieving dewatering of the upper aquifer prior to initiating excavation.

- **Localized dewatering within excavations will be required.** Due to historic isolated boils at the base of excavations and variable aquifer conditions, we anticipate collection galleries or pits may have to be constructed at the base of the clarifier excavation to collect localized perched groundwater and artesian infiltration (effect of lower aquifer) to the excavation.

- **The dewatering contractor must be experienced in similar dewatering applications.** We recommend the contractor have experience that reflects their ability to dewater the site to allow construction. Bidding contractors should demonstrate their ability to plan, design and implement a sufficient dewatering program based on similar project conditions, and provide documentation of similar project experience.

- **Foundation soil disturbance below the clarifier due to water issues or inappropriate equipment use, will affect earthwork construction and the potential for differential foundation performance.** Soil that has been disturbed due to excavation instability or construction procedures is not suitable for support of foundations. Careful construction procedures are required to achieve a stable foundation and slab subgrade for the proposed clarifier.

GEOTECHNICAL OPINIONS AND RECOMMENDATIONS

We have prepared this report based on our understanding of the proposed construction, interpreted site geologic and hydrogeologic conditions, verification of the site conditions versus our recommendations for the clarifier and RAS 2004 report, and results from our preliminary analyses. It will be necessary to carefully plan and stage construction to allow dewatering, excavation, and backfill for Primary Clarifier No. 3 to be accomplished and reduce the potential for project schedule and cost overruns. We consider planning for construction dewatering in advance of excavation construction, and foundation and excavation stability to be the most important from a geotechnical construction perspective. If design plans change, or subsurface conditions encountered during construction vary significantly from what was observed during our subsurface evaluation, we should be notified to review the report recommendations and make any necessary revisions.

Understanding and implementation of these recommendations will require our involvement with the contractor, design team and owner to verify correct report interpretation.

The report recommendations reflect our interpretation of the subsurface conditions at the test boring location. Subsurface conditions may vary at the proposed site, and variation of the soil conditions may affect construction cost and schedule.

Design Assumptions

We have assumed the contractor will accomplish construction by open-excavating the clarifier area following dewatering. Connecting utilities and piping between to the clarifier will likely be constructed using a trench excavation and portable shoring or trench boxes. For trench stability and earthwork construction, we have provided recommendations for a dewatered condition such that no hydrostatic pressures are realized within the excavation. We have assumed excavation equipment and other construction procedures will not induce dynamic loading which could increase soil pore water pressure causing local liquefaction, which may lead to both side slope and foundation soil instability.

Site and Subgrade Preparation and Excavation Characteristics

Native silt and sand underlayed "sod" topsoil and extended to about 8 feet below the ground surface. The topsoil is not suitable for use as structural fill for this project and should be removed from the area or stockpiled for later use as landscaping material. The underlying silt and sand may be used as structural backfill.

Site stripping and clarifier excavation can commence and continue to 2 feet above the static groundwater table as dewatering proceeds. Excavation to achieve the clarifier subgrade should not extend into non-dewatered soil. Once the clarifier subgrade has been achieved, the groundwater must be maintained 2 feet below the subgrade during clarifier construction.

The upper and lower aquifer will cause the silty sand to maintain a near saturated condition as a result of capillary rise. Due to this condition, equipment loads, sand boils and seepage, the potential for the foundation soil at the base of the excavation to pump or rut must be considered. Excavation should be terminated immediately if water-related soil disturbances are observed, and STRATA advised of the condition(s) in order for us to provide the necessary consultation. We anticipate excavation within the clay soil will be necessary to achieve the piping invert elevations beneath the clarifier. The potential for water-related foundation soil disturbance is greatest at this point in the excavation and the contractor may need to undertake additional localized dewatering measures and revise their construction approach should this condition occur.

The foundation soil elevation should be achieved using smooth blade, tracked equipment to reduce the potential for soil disturbance. Soil that is disturbed during subgrade preparation should be excavated to firm soil and replaced with granular structural fill.

The on-site soil has the potential to infiltrate the drain rock planned as part of the proposed underdrain system and the wall backfill for the clarifier. Therefore, we recommend placing a woven or non-woven geotextile fabric at the subgrade elevation to help prevent fines migration into the drain rock, and to facilitate granular structural fill placement and compaction. We recommend geotextile fabric utilized for the project be Amoco™ 1199, Amoco™ 4552 or have the following properties:

Mullen Burst Strength (ASTM D 3786)	– 250 psi (minimum)
Grab Tensile Strength (ASTM D 4632)	– 180 lbs (minimum)
Apparent Opening Size (ASTM D 4751)	– 70 to 120 sieve
Flow Rate (ASTM D 4491)	– 4 gal/min/ft^2 (minimum)

We recommend STRATA observe excavation and subgrade preparations immediately prior to geotextile placement and granular structural fill placement. Due to the potential for disturbance of the native soil, it will be necessary to rapidly achieve the excavation subgrade, place geotextile fabric, and place granular structural fill; the contractor should plan construction accordingly.

The excavation for piping beneath the clarifier is anticipated to extend up to about 28 feet below the existing ground surface. Based on the data from boring B-1, the excavation will penetrate through the clay and into the underlying granular soil. The lower groundwater aquifer will be encountered, which has a hydraulic head of about 23 feet, as measured in the monitoring well at the time we performed our field work. Construction dewatering of the lower aquifer will be necessary where the excavation for the piping (and possibly the central portion of the clarifier floor area that is closest to the aquifer) is close to and penetrates through the clay layer. The lower aquifer can be locally lowered prior to initiating this work using similar dewatering methods discussed in this report, or as excavation occurs. The latter option is presented based on our understanding that this approach was used for dewatering for the 2004 Final Clarifier No. 3 project, per discussion with the contractor. For the 2004 Final Clarifier No. 3 project, we understand the excavation contractor, once the lower aquifer was breached, was pumping using a localized multiple pump system, up to an additional 3,000 gpm (production only from the lower aquifer). Our visual estimate of the upper aquifer production, during limited field observations prior to the contractor penetrating the lower aquifer, was that the lower aquifer was producing at steady state flow less than 100 gpm.

Wet Weather/Wet Soil Construction

The on-site silt/sand and sandy clay encountered within the upper 30 feet of the soil profile is loose or soft and will likely maintain significant moisture content even after dewatering has occurred. Earthwork construction should reflect the potential for soft soil subgrade and the high disturbance potential.

Site dewatering could occur during low groundwater, winter months. Winter months typically exhibit inclement weather and generally poor construction conditions. If site construction is undertaken during wet weather periods or using wet soil for structural fill, the soil will be susceptible to pumping or rutting from heavy loads such as rubber-tired equipment or vehicles. Work should not be performed immediately after rainfall or until soil can dry. If construction commences before soil can dry after dewatering or precipitation or during wet periods of the year, earthwork should be performed by low pressure, track-mounted equipment that spread the vehicle load. All soft and disturbed soil should be removed as outlined in the *Site Preparation* section of this report. If native soil or structural fill is wet and soft but not disturbed, the following lift of structural fill placed over the subgrade should be a minimum thickness of 12 inches. Material placement and compaction should be such as to prevent pumping and disturbance of the underlying soft soil. During construction, runoff from precipitation or additional moisture seepage from excavation sidewalls should be intersected and diverted to prevent ponding of water within the project excavation.

STRATA should be periodically present at the time of excavation and subgrade preparation to verify that no soft or pumping areas exist prior to placing structural fill or concrete. We expect wet to saturated conditions may be encountered during clarifier foundation excavation and subgrade preparation. The contractor should expect these conditions and be equipped to replace wet or disturbed soil with granular structural fill or drain rock. If significant soft soil conditions are encountered, the use of a woven geotextile fabric within overexcavated areas may be necessary. STRATA should be consulted before placing any geotextile fabric within overexcavated areas in addition to the fabric already planned for the underdrain and clarifier floor subgrades.

Once final subgrades are achieved, it will be the contractor's responsibility to protect the soil from degrading from seepage, construction traffic and/or wet weather. Initial footing or underdrain excavations should not be initiated within 24 hours before expected precipitation. Concrete or structural fill placement directly over the subgrade should not be attempted following a significant precipitation event and the subgrade should never be allowed to freeze. The condition of the subgrade and careful construction procedures are critical to foundation and slab stability and long-term performance of structures.

Slope Stability for Temporary Excavation and Cuts

We expect the contractor will achieve the clarifier excavation by open-excavating to achieve the desired subgrade and stable side slopes. The large excavation for the clarifier will likely be constructed concurrently with site dewatering. Trench excavations are expected for pipe utilities connecting to the clarifier. The following discussion provides general guidelines for open and trench excavations and temporary slope stability providing a dewatered condition has been achieved.

All excavations, including trench construction and earthwork, should be constructed according to the OSHA excavation regulations, *Document 29, CFR Part 1926, Occupation Safety and Health Standards – Excavations; Final Rule*. In general, the subsurface conditions have been classified as B soils according to the OSHA criteria. Class B soil typically cannot be sloped steeper than 1:1 (horizontal to vertical) for excavations up to 20 feet deep, or if the side slopes of excavations are steeper, will require trench boxes or some other type of lateral support and protection (designed by a licensed engineer). Design of excavations and/or excavation support structures for excavations deeper than 20 feet may require design calculations and a report by a licensed, qualified engineer submitted to OSHA. Although trench excavations can be constructed with terracing according to the OSHA criteria, it is our preliminary opinion trench excavations made at or near vertical using available shoring technology will be expedient and require less construction space.

Notwithstanding the above OSHA criteria above, STRATA performed slope stability calculations for temporary excavation side slopes constructed at 1:1 (H:V) and 1.5:1 (H:V) for a dewatered condition. It is our opinion that slopes constructed at no steeper than 1.5:1 (H:V) will be stable, provided the site has been dewatered as discussed. Given the relatively loose condition of the silt/sand encountered during exploration, the OSHA criteria may not be adequate to maintain worker safety. Dewatered excavations up to 4 feet could be constructed vertically, depending on specific soil conditions.

Temporary trench excavation supported in the form of steel trench boxes, steel or timber shoring, and other means of trench wall protection can be used but should be designed by a professional engineer licensed in the state of Idaho. If trench boxes or other means of temporary support of pipe excavations are utilized, the trench box or shoring should be of sufficient width to be able to install the pipe, pipe bedding, and provide safe and productive working conditions.

Minor sloughing of the soil represented in this report could occur for excavation side slopes at 1.5:1(H:V), requiring appropriate maintenance and protection for workers and equipment. Localized perched groundwater and saturated soil due to capillary rise subsequent to dewatering may cause local flowing soil conditions and excavation instability. If near vertical excavation for trenches is selected using sheet piling, trench boxes or other methods for temporary side slope support, caving will likely occur. The caving will cause trench boxes to become lodged, requiring additional time to remove soil debris adjacent to, and confining the box. Rain and other water sources will exacerbate the potential for caving and sloughing of the soils.

Structural Fill

We recommend structural fill and backfill be used, where required, to support the clarifier, for pipe trench backfill and backfill around the clarifier, and where the performance of other structures could be affected by settlement. Structural fill and backfill should be free from vegetation and organic matter and consist of GW, GP, GM, SW, SM, SP, or ML soil as designated by the Unified Soil Classification System, Plate 3. Only granular structural fill (GW or GP) containing less than 10 percent passing the No. 200 sieve or drain rock may be used to support the clarifier. Structural fill and backfill should consist of particles no larger than 6 inches in diameter. Drain rock should have particles no larger than 3 inches and should be a washed product capable of free drainage. The on-site silt/sand may be reused as structural fill (except for fill beneath clarifier) and backfill, provided it is moisture conditioned sufficiently to allow the contractor to achieve compaction requirements. The contractor should expect significant moisture conditioning efforts when utilizing any of the native, on-site soil.

Fill and backfill construction should be accomplished in accordance with MWH project specifications. We recommend structural fill and backfill be placed in maximum 12-inch-thick, loose lifts at near-optimum moisture content. Structural fill and backfill placed at the site should be compacted to at least 95 percent of the maximum dry density of the soil as determined by ASTM D 698 (Standard Proctor), or to 65 percent relative density based on ASTM D 4253 and D 4254 if the material contains more than 30 percent material passing the ¾-inch sieve. If material utilized for structural fill and backfill does not have the gradation for relative compaction or relative density testing, a minimum of five complete passes should be applied to the material using a large (5-ton drum weight) roller. STRATA should provide construction observation to help establish a roller pattern and to verify that project compaction requirements have been met.

The native soil, if wet or saturated, has the potential for disturbance and/or construction induced liquefaction due to vibratory compaction equipment. If vibratory equipment is used, care should be taken to avoid excessive vibratory compactive effort on structural fill placed directly over wet, native soil. If the soil is disturbed, as evidenced by pumping, rutting or visual contamination of gravel placed over native soil, it will be necessary to remove the disturbed area to firm soil and replace it with approved granular structural fill.

These compaction requirements assume large (5-ton drum weight or larger) compaction equipment such as sheeps-foot rollers or smooth-drum, rollers will be utilized. The lift thickness must be reduced when using light compaction equipment with less than 5-ton drum weight. If earthwork occurs during or in wet conditions, we recommend the contractor have contingencies for replacing soft, wet soil with granular structural fill or drain rock. Structural fill should never be placed over disturbed or frozen subgrades.

We recommend STRATA evaluate the condition and suitability of on-site soil for reuse as structural fill and backfill and to monitor compaction during the fill placement. Where the subgrade is very soft, and drainage is not required, lean mix concrete may be utilized. Lean mix concrete can reportedly be constructed below shallow standing water if appropriately batched and placed, and should have a minimum compressive strength of 300 psi.

Compaction of backfill within 5 feet of walls should be performed only with small vibratory plates or walk-behind, smooth-drum, vibratory rollers to reduce surcharge loading of the walls. Walls designed for little or no wall movement should be monitored during the backfilling process through survey and string line methods. Below-grade clarifier walls should be backfilled as described in the *Permanent Dewatering* section of this report.

Foundations

We anticipate the underdrain subgrade and bearing soil for the clarifier foundations will consist of silty sand encountered in borings B-1 and B-2. Isolated layers of well-graded sand or sandy clay may be encountered at the footing bearing elevation. The subgrade for piping beneath the clarifier and other deep piping is also expected to consist of the silty sand and sandy clay. We recommend the clarifier subgrade comprise a minimum of 12 inches of granular structural fill (i.e. ≥12 inches of drain rock to be placed over structural fill) placed over undisturbed native soil. If the native soil is disturbed through construction activity, it will be necessary to remove disturbed areas and replace the soil with additional granular structural fill in accordance with structural fill requirements. Footings bearing on granular structural fill over undisturbed native soil as described above may be designed utilizing a maximum allowable bearing pressure of 1,500 psf. The allowable bearing pressure may be increased by 30 percent to account for transitory live loads such as wind or seismic forces. A submerged vertical modulus of subgrade reaction of 200 pounds per cubic inch may be utilized for design of slabs and pipe bedding placed in accordance with the *Structural Fill and Site and Subgrade Preparation* and *Excavation Characteristics* sections of this report.

If the above recommendations are followed, we estimate total and differential settlement (from the center to the edge of the clarifier) will be less than 0.5 inch. Soil disturbance as a result of construction activity has the potential to cause additional foundation settlement. Therefore, it will be critical for native soil below the foundation remain undisturbed prior to and during placing granular structural fill.

We recommend the clarifier foundation walls have a wall drainage system as shown on Plate 4, Perimeter Drain Detail and as discussed in the *Permanent Dewatering* section of this report.

We recommend a coefficient of friction of 0.35 be used for footing or foundation design for concrete cast directly on the silt/sand or sandy clay. Concrete cast directly on granular structural fill may use a coefficient of friction of 0.50 for design.

It is our opinion the maximum groundwater level for uplift design should be 2 feet below the existing ground surface, which reflects the artesian influence from the lower aquifer due to infiltration.

Lateral Earth Pressure and Coefficient of Friction

We recommend the clarifier foundation wall be designed to resist lateral earth pressure and surcharge from equipment or vehicles adjacent to the walls. Values for soil related equivalent fluid pressures are presented in Table 1.

Table 1. Rankine Lateral Earth Pressures

Rankine Lateral Earth Pressure Case	Equivalent Fluid Pressure (EFP)
At rest case (no wall movement)	90 pcf*
Active case (wall movement away from soil mass)	75 pcf*
Passive case (wall movement toward soil mass)	250 pcf*

*Includes soil buoyant unit weight and the unit weight of water.

Clarifier walls will be subject to load influences (surcharge) from adjacent equipment, vehicles, storage of materials, etc. The effect of surcharge has not been included in the above lateral earth pressure recommendations. For these conditions, a lateral earth pressure coefficient of 0.5, acting over the entire wall height should be used for design. Depending on actual static or dynamic loads, surcharge loads greater than 15 feet away from the wall will have a negligible effect on the lateral earth pressure to the foundation wall.

The design of below-grade walls should account for seismic load influences using an equivalent dynamic lateral fluid pressure equal to 10 pcf. The dynamic pressure should be added to the design static equivalent fluid pressure. The seismic pressure acts as an inverted triangle with its resultant acting 0.6 times the wall height measured from the base of the wall. The estimated passive equivalent fluid pressure will be reduced to 235 pcf during earthquake loading conditions.

Seismicity and Liquefaction

We understand the 2003 International Building Code (IBC) will be utilized for structural design. Section 1615 of the 2003 IBC outlines the procedure for evaluating site ground motions and design spectral response accelerations. STRATA utilized site soil and geologic data and the project location to establish earthquake loading criteria at the site referencing Section 1615 of the 2003 IBC. Based on our field exploration and knowledge of the upper 100 feet of the soil profile, we recommend a Site Class of "D" be utilized as a basis for structural seismic design. The Maximum Considered Earthquake