(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 five percent critical damping ratio in accordance with the IBC, Section 1615. 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.

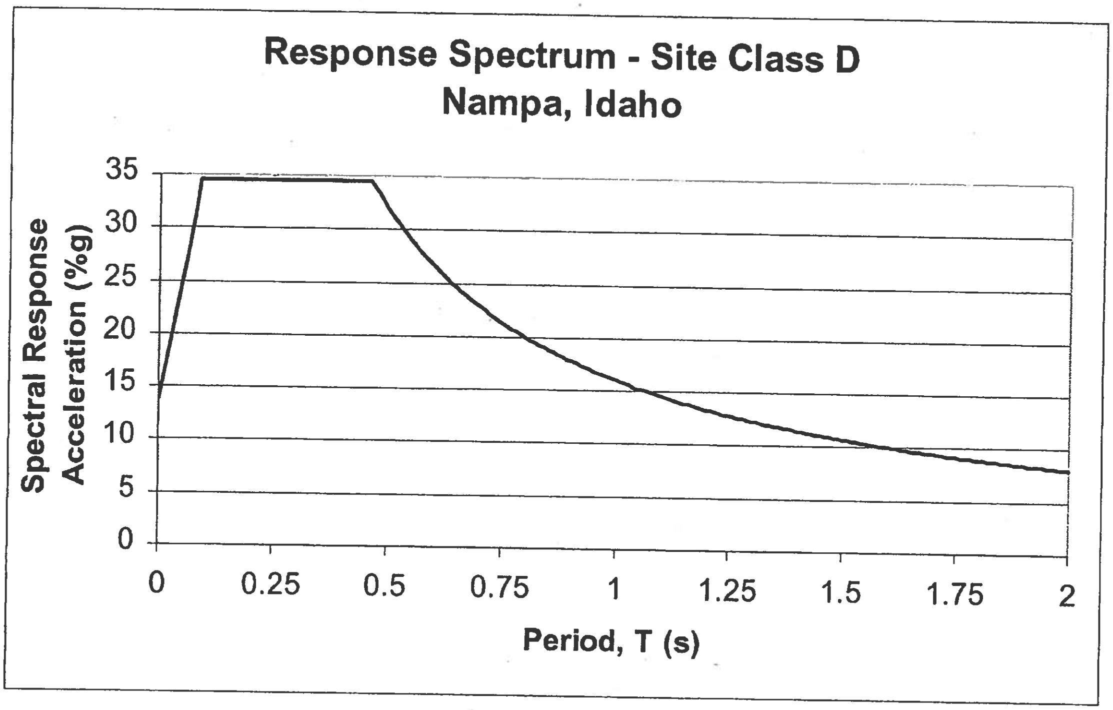


Figure 1. Spectral Response Acceleration

For engineering design, reliability-based accelerations can also be selected according to the "National Seismic Hazard Maps" (Frankel, et al., 1996) published by the U.S. Geological Survey. For Nampa, Idaho (zip code: 83687), these maps recommend the following values for the peak horizontal ground acceleration (PGA) and the spectral accelerations (for 5 percent critical damping ratio) corresponding to three different periods:

Table 2. Spectral Response Accelerations

Nampa, Idaho - Zip Code 83687			
Type of Acceleration	Probability of exceedence in 50 years		
	10 Percent (RP of 500 years)	5 Percent (RP of 1,000 years)	2 Percent (RP of 2,500 years)
Peak Ground Acceleration	0.066g	0.09g	0.14g
Spectral Acceleration at 0.2 seconds	0.15g	0.20g	0.32g
Spectral Acceleration at 0.3 seconds	0.13g	0.18g	0.28g
Spectral Acceleration at 1.0 second	0.047g	0.06g	0.096g

For example, if one uses a PGA value of 0.066g, there is a 10 percent chance that this value may be exceeded during the next 50 years. Alternatively, this value corresponds to a return period (RP) of about 500 years. The above values may be slightly larger at the Nampa WWTP site due to a localized response of the soil profile above bedrock.

STRATA performed a liquefaction triggering analysis for the silt/sand encountered during exploration. SPT N_{60} values obtained during exploration were corrected according to a procedure developed by Seed and Idriss (1971, modified). The National Seismic Hazard Maps (Frankel, et al., 1996) published by the USGS were referenced for probabilistic based peak ground accelerations. The Cyclic Stress Ratio (CSR) can be used to perform the triggering analyses and is defined as a measure of the force that is applied to the soil during earthquake loading. The CSR was developed using the peak ground accelerations as mentioned above, using the Seed and Idriss 1971 modified procedure. $N_{1(60)}$ values were obtained using field SPT N_{60} values corrected based on overburden stress, rod length, fines content and boring diameter. The CSR was compared to the $N_{1(60)}$ value using a graphical reference to evaluate the potential for liquefaction to be triggered at the site. The figure used to evaluate liquefaction triggering potential at the site is provided below as Figure 2. Comparison points between the CSR and $N_{1(60)}$ value that plots to the left of the individual curves will experience liquefaction during the design earthquake. Points to the right of the curves presented below will not experience liquefaction. The curves were developed for individual fines percentages based on a 50th percentile probability of exceedence.

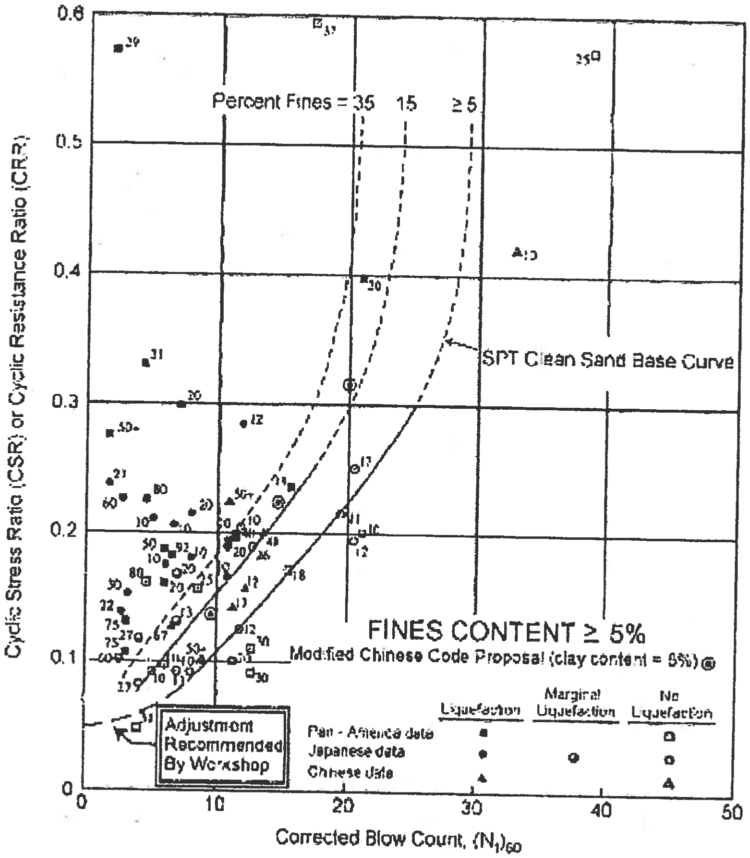


Figure 2. Liquefaction Triggering Analyses Chart (Youd and Idriss, 2001)

Peak ground acceleration for 10 percent, 5 percent and 2 percent probability of exceedence in 50 years were utilized for this analysis. The return periods for the 10 percent, 5 percent, and 2 percent probability of exceedence in 50 years are 500 years, 1,000 years and 2,500 years, respectively. The analyses, using the figure above, indicate that peak ground acceleration based on a 500 year return period will not trigger liquefaction at the site. Liquefaction triggering analyses utilizing the 1,000 year return period ground motion indicate that there will be a 50 percent probability that liquefaction will be triggered (falls on the line). Peak ground accelerations for the 2,500 year return period will likely trigger liquefaction. It is STRATA's opinion that return periods of 1,000 and 2,500 years have a small probability of occurrence for ground motion to occur relative to the operational life of the structure. Further, most SPT $N_{1(60)}$ values obtained at the site are higher than the worst-case SPT value utilized for analyses, which suggests that liquefaction may only occur locally in this soil.

Permanent Dewatering

We understand it will be necessary to periodically empty the clarifier for cleaning or maintenance. Emptying the clarifier will cause an imbalance of water pressure at the base of the slab and the walls due to a high groundwater level outside the structure. A permanent drain system is required 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 clarifier be dewatered to a depth of at least 6 inches below the base of the clarifier slab.

We recommend underdrains be constructed in concentric circles starting 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 4, Perimeter Drain Detail. The wall backfill should include a minimum of 1 foot of drain rock placed to within 6 feet of the final finished ground surface and connecting to the perimeter underdrain trench as shown on Plate 4. 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 clarifier slab and be sloped a minimum of 1 percent to connect to the manifold or collection discharge pipe to remove water that infiltrates the drain rock. We anticipate the perimeter underdrain and interior underdrains may be installed deeper below the clarifier slab to meet grades and pipe invert requirements near the center of the structure.

A minimum of 12 inches of drain rock should be placed beneath the clarifier slab (i.e. underlain by at least 12 inches of structural fill). The top and base of the drain rock should be protected by geotextile fabric having properties discussed in the *Site and Subgrade Preparation and Excavation Characteristics* section of this report. The geotextile is recommended to provide separation from both the native silt/sand and the clarifier slab concrete to help prevent contamination and clogging of the system.

The underdrains should connect to the 12-inch-thick drainage layer, as shown on the Proposed Clarifier Underdrain System Typical Section, Plate 5. We estimate a dewatering system flow volume at the pump station conservatively of up to 500 gpm, assuming that the lower aquifer will limited to no influence on dewatering of the upper aquifer (i.e. the lower aquifer is sealed from the upper aquifer at the time of construction). 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 water table has been drawn down before emptying the clarifier. Further, permanent dewatering design should account for an increased duration of dewatering to account for artesian pressure influence from the lower aquifer. Artesian pressure influences cannot be predicted at this time and may affect permanent dewatering system rates. We recommend the City and design team consider contingency plans for higher pumping rates than as mentioned above as a result of artesian pressure influence. Alternatively, several large, high volume dewatering wells could be constructed to the lower aquifer to relieve artesian pressure prior to initiating clarifier maintenance. The protocol for accomplishing the dewatering should be carefully outlined in the operations manual for the clarifier and the WWTP staff trained accordingly. Once design and long-term operation plans are finalized, these preliminary dewatering design recommendations should be finalized.

Pavement Subgrade Preparation and Section Design

This section is based on our assumption that the traffic volumes will be less than 200,000 Equivalent Single Axle Loads (ESAL) in driveway areas. The pavement subgrade is anticipated to consist of silt and silty sand. Correlations based on the results of our laboratory testing indicate the subgrade for the proposed asphalt areas will have an estimated R-value of 30. 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.

Clarifier construction traffic may cause failure of the near surface silt subgrade. We suggest the silt subgrade be protected by covering the construction traffic and staging areas with at least 12 inches of pit run sand and gravel. The surface should be observed as construction proceeds for evidence of subgrade failure, which includes pumping and rutting of the granular surface cover. If failure is observed, additional pit run may be placed. Alternatively, the silt subgrade may be covered with a reinforcing geofabric, typically and woven geotextile and 12 inches of pit run placed over the fabric to protect the subgrade from failure. We recommend this option be used if a pavement section is planned. The geofabric should have the following minimum properties:

Mullen Burst Strength (ASTM D 3786)	– 550 psi (minimum)
Grab Tensile Strength (ASTM D 4632)	– 280 lbs (minimum)
Puncture Resistance (ASTM D 4833)	– 120 lbs (minimum)
Trapezoidal Tear Strength (ASTM D 4491)	– 120 lbs (minimum)

The final pavement section may incorporate the construction section provided the subgrade has not failed and the contaminated pit is stripped to relatively non-contaminated pit run. Pavement subgrade specifications should reflect the need to expose the non-contaminated pit run gravel prior to placing granular structural fill to achieve subgrade for the asphalt section. Once the non-contaminated pit run gravel is exposed, the subgrade should be compacted to at least 95 percent of the maximum dry density of the soil as determined by ASTM D 698 (Standard Proctor). If the exposed subgrade will not consist of existing pit run gravel fill, STRATA should be contacted to provide recommendations.

Pavement support sections should include at least 6 inches of ¾-inch-minus crushed sand and gravel base course placed over 12 inches of granular (compacted pit run) subbase. Alternatively, this granular support section could consist of 4 inches of ¾-inch-minus crushed gravel base course placed over at least 8 inches of geofabric reinforced granular subbase. All ¾-inch-minus base and subbase should be compacted to structural fill requirements prior to placing asphalt.

The asphalt surface should consist of a minimum of 3 inches of asphalt concrete. 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 Transportation Department (ITD) Class A or B asphalt design requirements. Asphalt construction and final surface smoothness, joints and density should meet ITD specifications.

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

It is our opinion 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

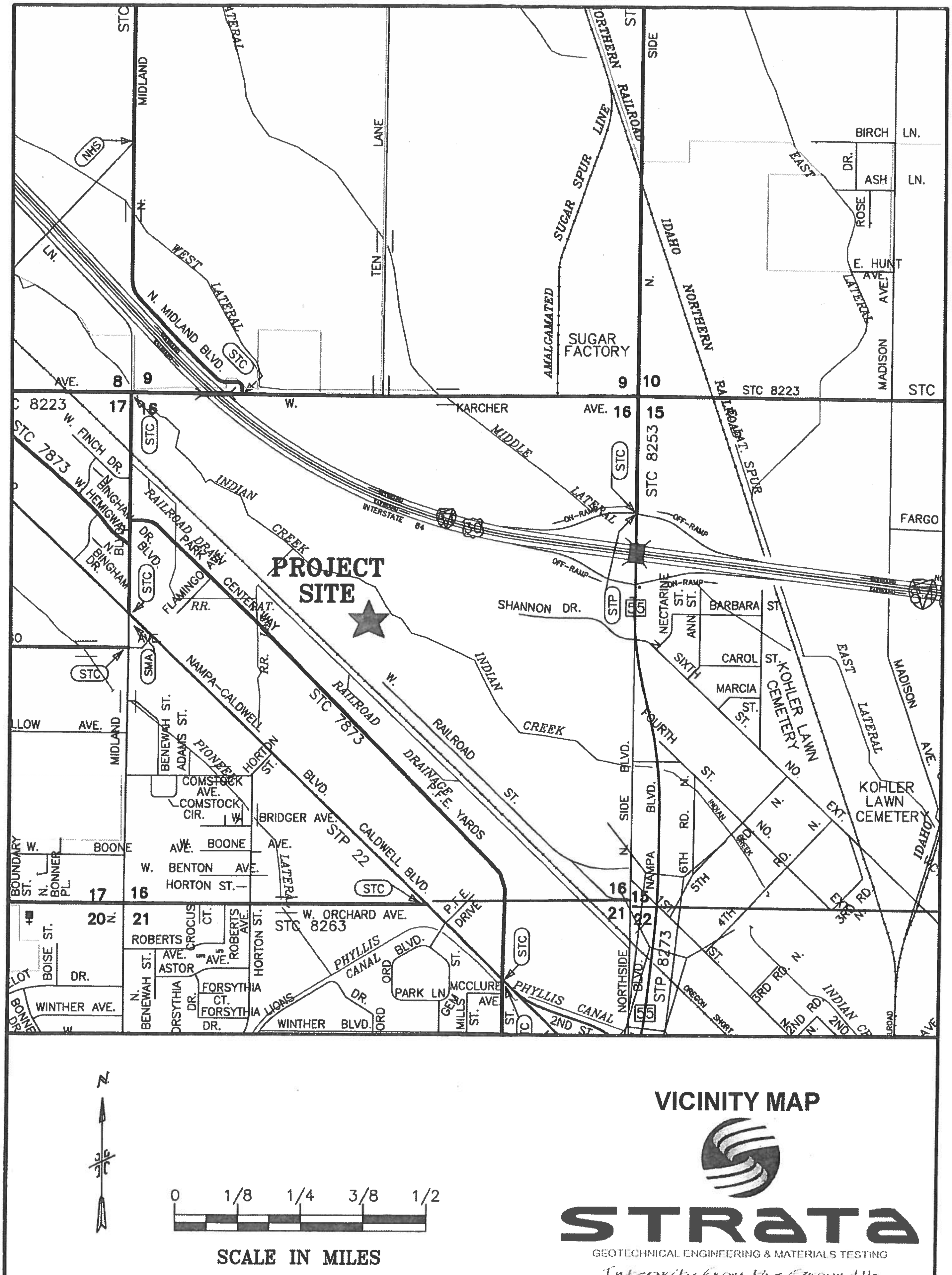
The opinions and recommendations contained herein are based on findings and observations made at the time of our subsurface evaluation. If conditions are exposed which appear to be different from those observed during our field evaluation and as described in this report, STRATA should be notified to 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 for the proposed Primary Clarifier No. 3 at the Nampa Waste Water Treatment Plant. 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. Also, we recommend a pre-construction survey be completed on all nearby structures that are considered to be potential candidates for disturbance, settlement or other adverse performance associated with the planned construction.

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 warranties, either expressed or implied.

The following plates accompany and complete this report:

Plate 1:	Vicinity Map
Plate 2:	Site Plan
Plate 3:	Unified Soil Classification System (USCS)
Plate 4:	Perimeter Drain Detail
Plate 5:	Proposed Clarifier Underdrain System Typical Section
Appendix A:	Exploratory Boring Logs



VICINITY MAP

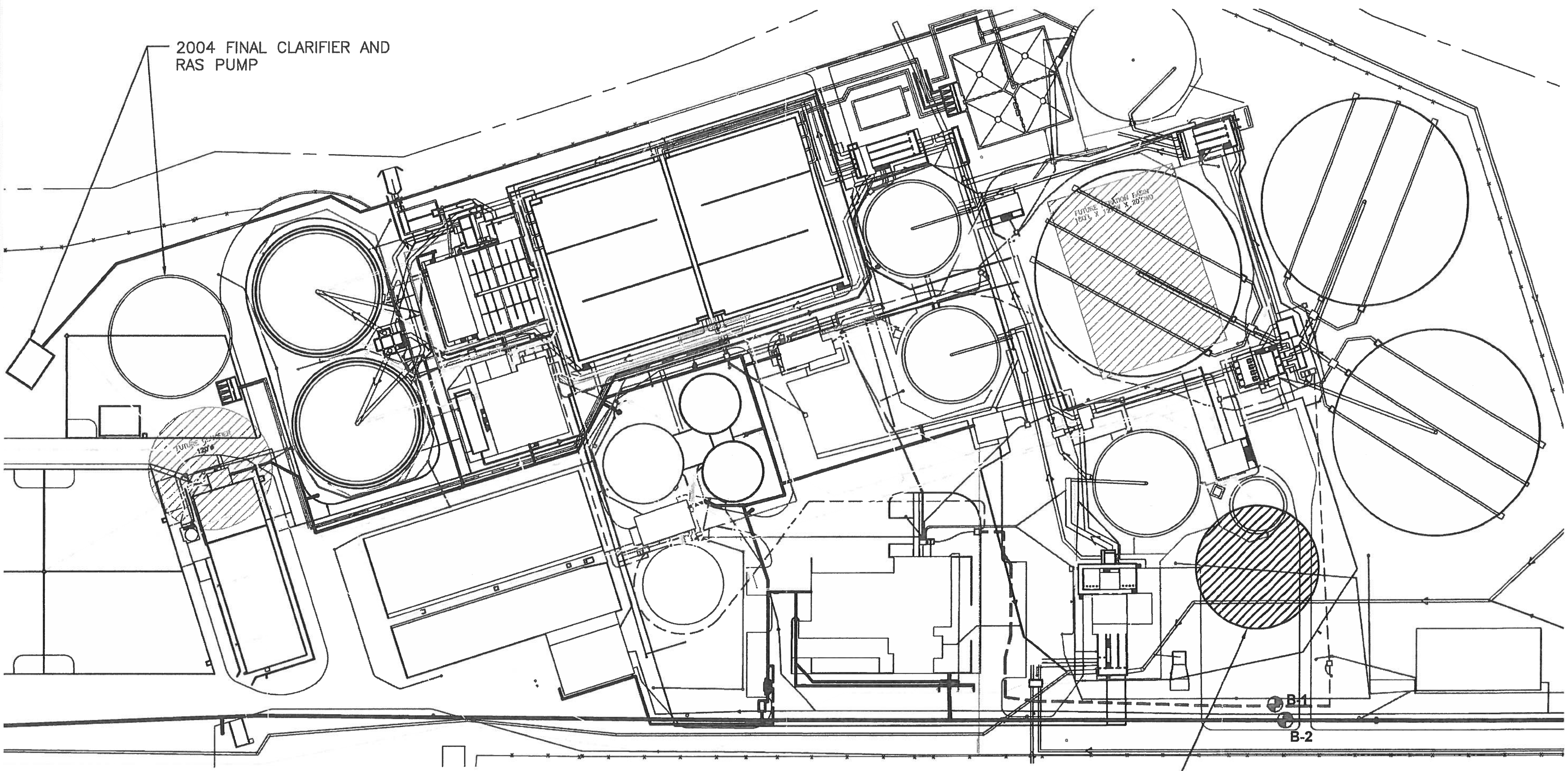
STRATA

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

Integrity from the Ground Up

Reference: I.T.D. Caldwell City Map

MONWAT B05202A PLATE: 1



LEGEND

B-1 ● Approximate Location of Borings Observed by Strata on October 10, 2005.

SITE PLAN
Nampa WWTP
Nampa, Idaho

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING
Integrity from the Ground Up

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.

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Reference: Site Plan From Drawing by Montgomery Watson Engineering, Named "wwaterplant.dwg", Dated 3/1/2004.

MONWAT B05202A PLATE: 2

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.

SOIL CLASSIFICATION CHART

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	▽	Groundwater at Time of Drilling	BK	Bulk Sample
‖	Rock Core			RG	Ring Sample
□	Shelby Tube 3–Inch OD Undisturbed Sample				

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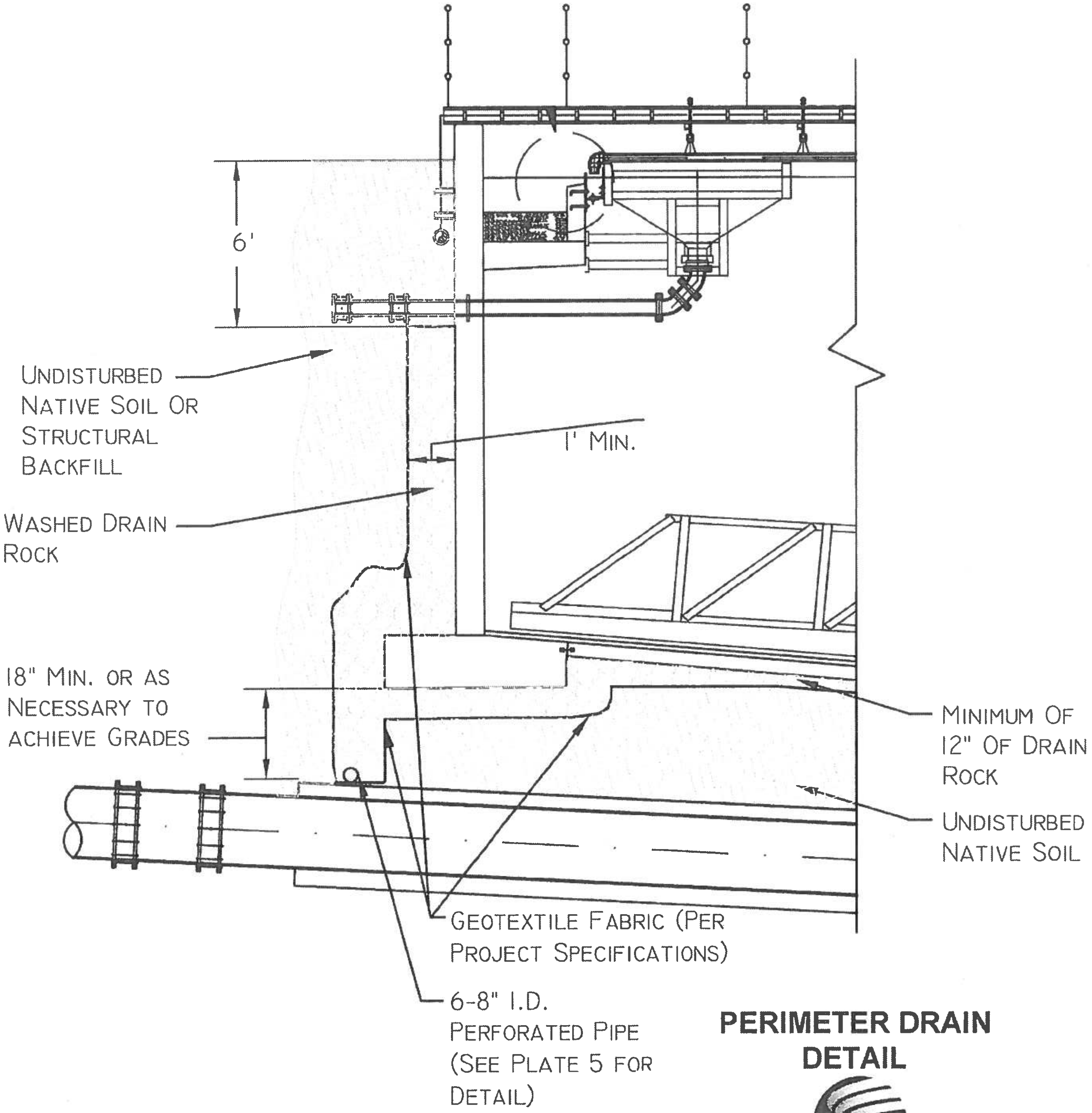
Integrity from the Ground Up



PERIMETER DRAIN DETAIL

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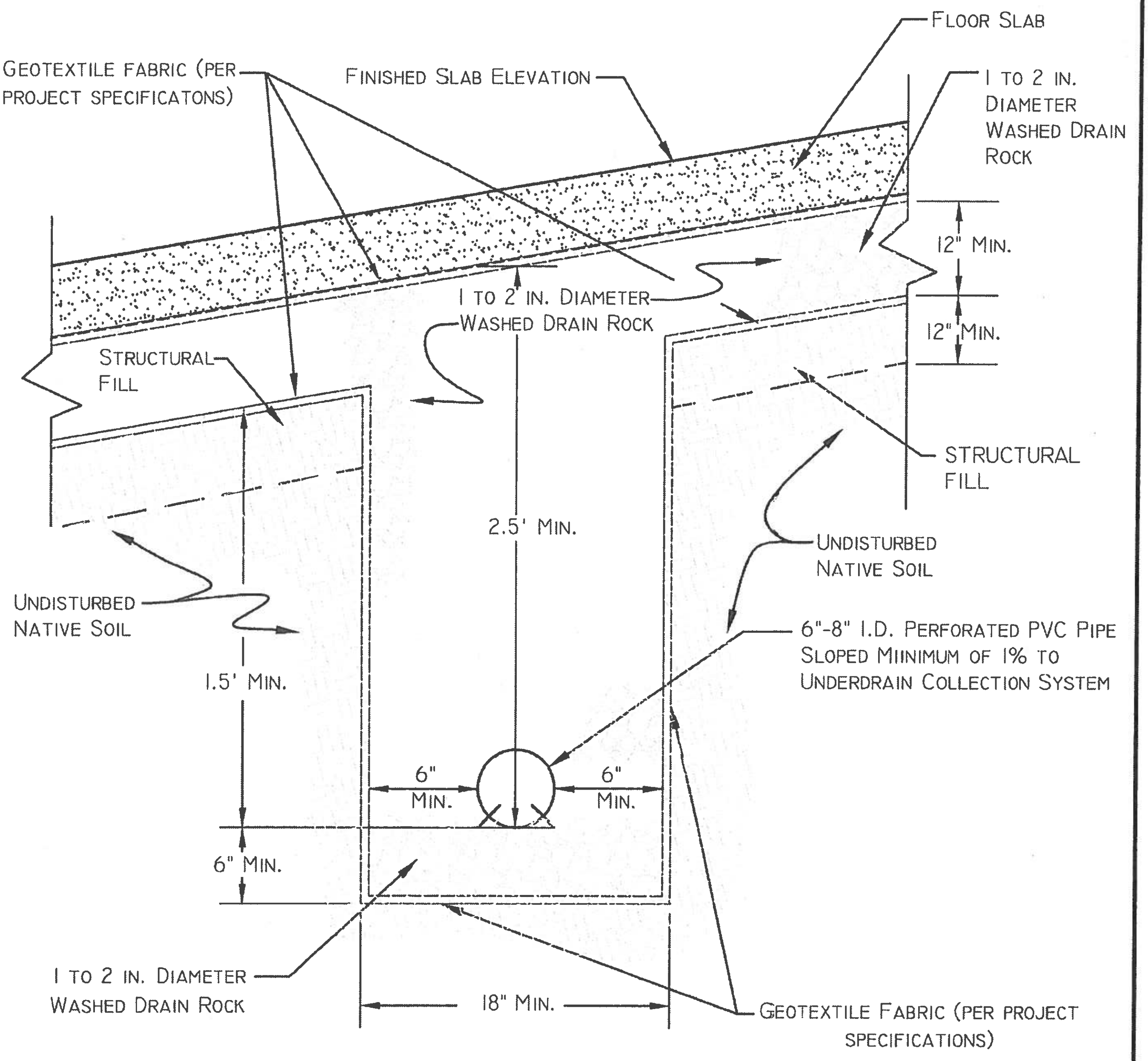

Informational Only (Not To Scale)

PROPOSED CLARIFIER UNDERDRAIN SYSTEM TYPICAL SECTION

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APPENDIX A

Boring No. 1 Subsurface Soil Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE TYPE	BLOWS PER 6 INCHES	$N_{(60)}$ SPT BLOWS PER FOOT	$N_{(60)}$ BLOWS PER FOOT	POCKET PENETRO-METER (TSF)	(%) PASSING 200 SIEVE	REMARKS Note: BGS = Below Ground Surface
SILT (Native) – tan, firm to stiff, moist to saturated at 6.4 feet.		ML								Trace gravel.
	1									
	2								95	Upper aquifer groundwater encountered at 6.4 feet BGS. Lower aquifer groundwater rose to 3.25 feet after clay layer was penetrated. See report text for detailed explanation.
	3	∇(L)		▮	2 5 8	13	29			
	4									
	5									
	6	∇(U)		▮	8 4 2	6	11			
?— —?— —?— —?— —?—	?	?	?							
SILT with Sand– brown, medium dense, saturated.	7	ML								
	8			▮	3 4 6	10	24	0.5–1		
	9								81	
	10									
	11			▮	16 12 14	26	55	3.5–4.5		
Silty SAND – brown, medium dense, saturated.		SM	⁘							
	12		⁘							
	13		⁘	■	7 16 18	21	39			
	14		⁘							
	15		⁘							
	16		⁘	▮	9 11 12	23	38		12.1	
	17		⁘							
	18		⁘	■	13 21 22	32	48			
	19		⁘							
	20		⁘							

File: MONWAT	Production Well
Project: B05202A	Date Drilled: 10-10-2005
Drill Rig: BK-81	Boring Diameter: 8"
Depth to Groundwater: 6.4'	Logged By: AKL

STRATA
Geotechnical Engineering & Materials Testing
Integrity from the Ground Up

EXPLORATORY BORING LOG
B-1
Sheet 1 of 2

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Boring No. 1 Subsurface Soil Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE Type	BLOWS PER 6 INCHES	N(60) SPT BLOWS PER FOOT	N1(60) BLOWS PER FOOT	POCKET PENETRO-METER(TSF)	REMARKS Note: BGS = Below Ground Surface
CLAY with Sand – brown, soft to stiff, saturated.	21 22	CL	▨		4 1 1	2	3	0.5	
	23 24		▨		3 5 4	9	12	0.5–1	In situ Dry Density = 94.5 pcf In situ moisture = 26.9 % Atterberg Limits: LL = 31 PI = 15
	25 26		▨		7 10 10	20	26		Monitoring well installed to 20 feet 9 inches BGS. Screened 4 feet 3 inches to 20 feet 9 inches BGS.
Poorly–Graded SAND with Silt – brown, medium dense, saturated.		SM–SP							
Boring terminated at 26.5 feet BGS.	27 28 29 30 31 32 33 34 35 36 37 38 39 40								

File: MONWAT
Project: B05202A
Drill Rig: BK–81
Depth to Groundwater: 6.4'

Production Well
Date Drilled: 10–10–2005
Boring Diameter: 8"
Logged By: AKL

STRATA

EXPLORATORY BORING LOG
B-1
Sheet 2 of 2

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Boring No. 2 Subsurface Soil Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE Type	BLOWS PER 6 INCHES	$N_{(60)}$ SPT BLOWS PER FOOT	$N_{(60)}$ BLOWS PER FOOT	POCKET PENETRO-METER(TSF)	REMARKS Note: BGS = Below Ground Surface
SILT – tan, moist to saturated at 6.4 feet BGS.	1 2 3 4 5 6 7	ML ▽							Trace gravel.
Sandy SILT – brown, loose to medium dense, saturated.	8 9 10 11	ML							
Silty SAND – brown, medium dense, saturated.	12 13 14 15	SM							
Boring terminated at 15 feet BGS. Screened from 5 to 15 feet BGS.	16 17 18 19 20								

File: MONWAT	Monitoring Well
Project: B05202A	Date Drilled: 10–10–2005
Drill Rig: BK–81	Boring Diameter: 6"
Depth to Groundwater: 6.4'	Logged By: AKL

STRATA

EXPLORATORY BORING LOG

B-2

Sheet 1 of 1

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Strata Geotechnical Engineering Evaluation – May 18, 2004

REPORT

Geotechnical Engineering Evaluation
Proposed Clarifier and RAS Pump Station
Nampa Wastewater Treatment Facility
Nampa, Idaho

May 18, 2004

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING

BOISE COEUR D'ALENE MOSCOW IDAHO FALLS POCATELLO SPOKANE TRI-CITIES

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING
8653 W. Hackamore Drive, Boise, Idaho 83709
208 376-8200 / Fax 208 376-8201

May 18, 2004
File: MONWAT-B04012A

Mr. Daniel Barbeau, P.E.
MWH Global
671 E. Riverpark Lane, Ste 200
Boise, ID 83706

RE: **REPORT**
Geotechnical Engineering Evaluation
Proposed Clarifier and RAS Pump Station
Nampa Wastewater Treatment Facility
Nampa, Idaho

Dear Mr. Barbeau:

Strata, Inc. has performed the authorized geotechnical engineering evaluation for the Proposed Clarifier and RAS Pump Station at the Nampa Waste Water Treatment Facility in Nampa, Idaho. Our work was performed in general accordance with our proposal dated February 2, 2004. 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.

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 is critical.
- Localized dewatering within excavations will be required.
- Design for long-term dewatering and structural design of the clarifier and RAS pump station floor should account for the apparent artesian conditions at the site.
- The selected dewatering contractor must be experienced in similar dewatering applications.
- Foundation soil disturbance below the clarifier and RAS pump station, due to water issues or inappropriate equipment use, will affect construction and increase the potential for differential foundation performance.

The report presents our evaluation and assessment of the hydrogeologic conditions and provides preliminary recommendation or suggestions for approaching site dewatering. Also, our presentation provides preliminary estimates for pumping rates and times for assumed approaches based on our interpretation of the hydrogeologic conditions. The contractor may review or use these options, but should not rely on this work in planning and design for site dewatering and earthwork in wet soil conditions. The contractor should conduct independent site evaluation and other 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 STRATA be retained 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. Please contact us if you have any questions or further requirements.

Sincerely,

Chris M. Comstock, E.I.T.

H Robert Howard

H. Robert Howard, P.E.

CMC/HRH/nl

REGISTERED PROFESSIONAL ENGINEER
5/19/04
3339
STATE OF IDAHO
H. ROBERT HOWARD

REPORT
Geotechnical Engineering Evaluation
Proposed Clarifier and RAS Pump Station
Nampa Waste Water Treatment Plant
Nampa, Idaho

PREPARED FOR:

Mr. Larry Bennett, P.E.
Mr. Dan Barbeau, P.E.
Montgomery Watson Harza
671 E. Riverpark Lane, Ste 200
Boise, ID 83706

PREPARED BY:

Strata, Inc.
8653 W. Hackamore Drive
Boise, Idaho 83709
(208) 376-8200

May 18, 2004

TABLE OF CONTENTS

REPORT
Geotechnical Engineering Evaluation
Proposed Clarifier and RAS Pump Station
Nampa Waste Water Treatment Plant
Nampa, Idaho

INTRODUCTION

This report presents the results of our geotechnical engineering evaluation for the proposed Clarifier and RAS Pump Station to be located at the existing Nampa Waste Water Treatment Plant (WWTP), 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. Specifically, we focused on providing geotechnical recommendations for final planning and design and preliminary dewatering criteria for constructing the proposed Clarifier and RAS pump station. Also, as discussed in our proposal dated February 2, 2004, 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 the auger borings as pumping wells/observation wells and conducted groundwater aquifer tests. Aquifer test data were analyzed to evaluate the characteristics of the drawdown curves and to

estimate aquifer transmissivity. Specific yield (storativity) of the aquifer was estimated based on grain-size information and model calibration. One boring was constructed as a production well and three were designated as observation wells.

5. Reduced and analyzed aquifer test data to evaluate hydraulic continuity, potential boundary conditions and estimated hydrogeologic properties of the groundwater conditions encountered. These coefficients were utilized to evaluate potential dewatering designs and to provide construction recommendations.

6. Completed engineering and hydrogeologic analyses, using computer software systems including SEEP-W, AQTESOLV, WinFlow, MathCAD and hand performed calculations to help evaluate preliminary dewatering methods and configurations as discussed with MWH. Dewatering criteria included estimated hydraulic conductivity, contributions from nearby Indian Creek, estimated dewatering options and general dewatering considerations.

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. Prepared this final report. This report includes a brief geotechnical baseline interpretive section to be used by contractors to gain additional understanding of the conditions where excavation is planned for the RAS pump station and clarifier. The final report includes a site plan, boring logs and illustrative geologic cross sections. A draft report was submitted to MWH for review and comment. Five final report copies were provided.

PROPOSED CONSTRUCTION

We understand a new clarifier and RAS pump station are planned immediately west of the existing clarifiers at the north and west end of the plant as shown on Plate 2, Site Plan. The RAS pump station will be located either northeast or southwest of the new clarifier. Preliminary design indicates the clarifier will be approximately 120 feet in diameter with 14-foot to 16-foot side water depth. The clarifier walls will be supported by a continuous foundation connecting to a 6-inch-thick reinforced concrete slab for the base of the clarifier. The clarifier slab will have a groundwater underdrain system connecting to a sump pit in order to allow dewatering of the clarifier for maintenance. The invert elevation of the connecting pipes beneath the clarifier floor will be up to 27 feet below the existing ground surface. The connecting pipes will be encased in up to 6-inch-thick concrete. The existing sludge bed in the vicinity of the proposed clarifier will be demolished prior to or during clarifier construction.

The RAS pump station will be supported on conventional footings with a thickened slab to resist hydrostatic pressures. The RAS pump station will be approximately 36 by 64 feet wide and may extend up to 15 feet below the existing ground surface. We understand the RAS pump station will not have a groundwater dewatering system and will be designed to resist buoyancy forces for the partially submerged structure. Two potential locations for the RAS pump station are illustrated on Plate 2.

SITE EVALUATION

Strata subcontracted the installation of four borings near the proposed clarifier and RAS pump station on February 17, 2004. Exploration locations are presented on Plate 2, Site Plan and were documented by taping and pacing from existing site features. Borings were established based on input from the design team, Assistant City Engineer, Mr. Case Houson, and our understanding of proposed construction. Borings were generally advanced in five-foot intervals using a CME-75 drill rig equipped with 8-inch outside diameter (2-inch monitoring wells) and 12-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 generally obtained in the borings 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 ten feet for 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. 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.

Following geotechnical exploration, wells were installed within the four borings. The initial boring near B-2 encountered artesian pressure conditions at approximately 29 feet. The boring was sealed using bentonite slurry throughout the clay layer and a new boring, B-2, was constructed to the top of the clay layer approximately 15 feet east of the decommissioned boring. Boring B-2 was installed as a pumping well with 4-inch diameter PVC casing. The well was installed to an approximate depth of 23 feet below the existing ground surface and included 13 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. Pumping wells were permitted through IDWR and can reportedly be lawfully utilized to assist the dewatering program during construction.

Monitoring wells were installed within B-1, B-3, and B-4 and were constructed similarly as described above, but with 2-inch diameter PVC casing. Monitoring wells were also permitted with IDWR and received flush-mount casing at the ground surface. All four wells received compatible locks and the keys were provided to MWH and WWTP personnel.

General Site Conditions and Geology

The site is located at the northwest boundary of the WWTP near an existing sludge drying bed. Boring B-2, the pumping well, is approximately 112 feet from Indian Creek. Indian Creek is approximately 70 feet from the edge of the proposed clarifier. Indian Creek consists of a natural channel that exhibited alternating layers of silt and sand on its exposed banks. The site slopes gently to the north from the Union Pacific Railroad tracks and generally drains to Indian Creek.

The generalized project geology, based on our fieldwork and review of geologic references is shallow fill overlying alluvial silty sand, sandy clay and sand. Basalt bedrock is typically encountered between 40 and 50 feet below the existing ground surface. The alluvial soil encountered during exploration, and observed in the banks of Indian Creek, 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 current subsurface and historic data (presented in the discussion section), the apparent thickness and lateral extent of the alluvial soil layers appears to be relatively consistent throughout the plant. However, isolated sand and gravel lenses are possible, and will influence the hydrogeologic characteristics of the 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. We have prepared Plate 4, Illustrative Geologic Cross Sections A-A' and B-B', to illustrate our interpretation of the general soil and groundwater conditions that may be encountered during construction. The cross section locations are shown on Plate 2, Site Plan.

Subsurface Conditions

Generally, the soil conditions near the proposed clarifier were relatively consistent. However, the elevation of the contact between layers varies across the site. Soil conditions encountered during our recent exploration are relatively similar compared to other borings completed at the site. The following discussion delineates the soil profile within recent borings. Specific layer contacts and geotechnical data can be referenced to the boring logs in Appendix A. Appendix B presents a brief baseline section, which interprets the geologic conditions at the site and briefly discusses implications of the anticipated subsurface conditions as they apply to construction. We anticipate that Appendix B can be used by bidding contractors to help understand the geologic conditions at the site.

Boring B-1

Silty gravel with sand was encountered at the ground surface. Silty gravel with sand appeared to consist of pit run fill and was described as brown to tan, loose to medium dense and wet. Pit run extended to approximately 3.5 feet below the ground surface in this boring. Native alluvium, classified as silty sand with gravel, was encountered below pit run. Silty sand with gravel was dark brown to tan, loose to medium dense, and wet. The silty sand with gravel extended to approximately 19 feet where sandy lean clay was encountered. The clay was brown, very soft to soft, saturated, and extended approximately 28 feet below the existing ground surface. Well-graded sand, described as tan, medium dense and saturated, was encountered below the sandy lean clay. The well-graded sand extended to at least the termination depth of exploration at 31.5 feet.

Boring B-2

Boring B-2 also encountered pit run fill to an approximate depth of 4 feet below the existing ground surface. Silty sand was encountered below pit run and extended to a depth of 23 feet below the existing ground surface. Silty sand was similar to the silty sand with gravel encountered in B-1. Sandy lean clay, as described in B-1, was encountered from 23 to 29.5 feet below the ground surface. The sandy lean clay, however, was very soft in the location encountered. Well-graded sand, as described in B-1, was encountered from 29.5 feet to the termination depth of the boring at 31.5 feet.

Boring B-3

Silty sand with gravel consisting of pit run was encountered in B-3 to a depth of 17 feet below the existing ground surface. It is our opinion this material was used to backfill the excavation for the existing clarifier. Silty sand, as previously described, was encountered at 17 feet and extended to 22.5 feet where sandy lean clay was encountered. Sandy lean clay extended from 22.5 to 29.5 feet below the existing ground surface. Well-graded sand was encountered from 29.5 feet to 31.5 feet below the existing ground surface, where the boring was terminated.

Boring B-4

Pit run fill was observed to a depth of 4 feet. Silty sand was encountered at 4 feet and extended to approximately 18 feet below the existing ground surface. Very soft sandy lean clay was encountered below the silty sand and extended to approximately 29 feet below the existing ground surface, where well-graded sand was encountered. Well-graded sand extended to at least the termination depth of exploration at 31.5 feet.

Groundwater Conditions

As previously presented, Strata encountered artesian pressure in decommissioned boring B-2. Artesian pressure was not encountered in B-1, B-3 or B-4. We understand the WWTP project has a history of artesian conditions that vary across the site. From measurements taken during our recent exploration prior to boring decommissioning, the artesian pressure near the clarifier center appears to rise to approximately two feet above the existing ground surface. However, it is our opinion artesian conditions may be encountered at any depth below or within the clay later, typically extending to about 29 feet below the ground surface (about elevation 2423).

The static groundwater level above the clay layer was encountered from 6.6 to 11.5 feet below the existing ground surface. Near surface, static groundwater can be expected to extend to the top of the clay layer at between 19 and 22 feet below the ground surface.

Our interpretation of these unique hydrogeologic conditions is that two distinct aquifers exist at the site. The sandy clay layer encountered in all borings appears to act as a confining layer between the two aquifers. The upper system is an unconfined aquifer consisting of near surface groundwater. The lower system is confined and exhibits artesian pressure. Plate 4 illustrates the two distinct aquifers that exist at the site.

Groundwater levels in the upper aquifer fluctuated slightly with heavy precipitation. The elevation of Indian Creek also appears to influence the static groundwater levels at the site within the upper aquifer.

Table 1 below presents groundwater measurements taken within the upper aquifer on February 19, 2004. Elevation data was provided by MWH. The maximum groundwater elevation is expected to be as high as 2451 based on our review of available geotechnical data and reported boring elevations. Artesian influence, as a result of constructing the clarifier, may cause the maximum groundwater elevation to be higher following completion of construction. The elevation of artesian water pressure is estimated at 2454.

Table 1. Groundwater Measurements for B-1 through B-4 on 2/19/04

Boring	Depth to Groundwater, ft (B.E.G.S.)*	Approximate Groundwater Elevation, ft
B-1	6.6	2446.7
B-2	9.5	2442.3
B-3	11.5	2441.9
B-4	9.0	2444.2

*Below Existing Ground Surface

Bedrock was not encountered during exploration. 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, an aquifer pumping test was performed within the upper aquifer, utilizing the four borings. A 36 gallon-per-minute (gpm) submersible pump was utilized in boring B-2 to discharge water from the well. Two Solinst™ Levelogger pressure transducers were used 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 the nearby sludge bed, and later pumped 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 February 23, 2004. The 36-gpm pump discharge was throttled to about 2.8 gpm and was set at a depth of 22.5 feet below the existing ground surface. The test was performed for approximately 4 days. Drawdown was measured in the pumping well, and in wells B-1, B-3, and B-4. The water levels in monitoring wells B-1, B-3 and B-4 experienced drawdown of approximately 0.20, 0.15 and 0.14 feet respectively. Heavy precipitation during the course of the test caused groundwater levels within the upper aquifer to increase to above static levels prior to terminating the aquifer test. Groundwater drawdown data was gathered from each of the three monitoring wells, including the pumping well.

On March 13, 2004, a qualitative drawdown test was performed utilizing the existing dewatering system for the existing two clarifiers adjacent to the proposed clarifier. The underdrain system for the existing clarifiers reportedly slopes to a sump basin that discharges groundwater to the clarifier. Strata installed two Solinst™ pressure transducers within borings B-2 and B-3. The source from which groundwater was discharged consisted of the drainage system for the two clarifiers. Specific hydrogeologic analyses cannot be performed as the specific construction of the underdrain system and discharge rates are not well understood. However, this information proved valuable to qualitatively estimate groundwater drawdown based on the existing dewatering system being active. As a result of extended dewatering of the two existing clarifiers, the static groundwater levels for B-1 through B-4 were drawn down by approximately 2.5, 0.3, 4.0, and 2.3 feet, respectively. Groundwater levels appeared to come to equilibrium after approximately 4 days of monitoring the system. Based on data from the pressure transducer installed in B-3, the pump appears to discharge water for 45 to 70 minutes and shut off for four hours. According to the datalogger data, the drawdown and recharge in B-3 was approximately 0.39 feet. It is our opinion the dewatering system discharges a large volume of water initially as a result of draining the clarifier backfill consisting of pit run gravel. Once the clarifier backfill has been dewatered, the recharge to the aquifer is estimated at 0.39 feet in 4.2 hours, or approximately 1.1 inches per hour. This infiltration rate can be correlated to a hydraulic conductivity of approximately 2.5×10^{-5} ft/s (7.8×10^{-4} cm/s), which is similar to values estimated from aquifer test data.

Laboratory Testing

Select soil samples were tested to assess Atterberg limits, pH, resistivity, in situ density and moisture content, shear strength, triaxial consolidation and grain size distribution. Laboratory testing was performed in general accordance with ASTM standards. The results of laboratory testing are presented on exploratory boring logs and in Appendix C, Laboratory Test Results. 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 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. It is our opinion that site dewatering is possible, assuming a well-planned, practical approach is 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 between 3.5 and 23.0 feet below the existing ground surface. The sandy clay layer acts as a confining layer and the lower artesian aquifer was typically encountered between 28.5 and 29.5 feet below the existing ground surface.

Analyses

Aquifer test data from the upper aquifer were used to develop time-drawdown curves for each observation well and the pumping well. Well construction, measured pumping rates, subsurface 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 the 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 preliminary analyses indicate the hydraulic conductivity of the upper aquifer for preliminary design will be 1 x 10^{-5} to 5 x 10^{-5} feet per second (ft/sec) (10^{-4} to 10^{-3} cm/sec). The hydraulic conductivity for the lower aquifer is estimated between 5 x 10^{-5} to 5 x 10^{-4} feet per second (ft/sec) (10^{-3} to 10^{-2} cm/sec). The soil and hydrogeologic conditions outlined on Plate 4 were used as a model for the aquifer and dewatering analyses. 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 dewatering options could be evaluated. Complex analytical or numerical modeling is not warranted without additional hydrogeologic data. Gathering additional hydrogeologic data for the site would be time consuming and costly. In addition to the aquifer complexities, the pumping tests were performed during significant precipitation events and in between active groundwater dewatering around the existing clarifiers. Indian Creek and the groundwater gradient also influence the groundwater system through variable flux contributions, and recharge resulting from hydraulic gradients. These and many other factors complicate the hydrogeologic evaluation of the site conditions. Therefore, it will be crucial for the contractor to carefully plan and implement dewatering operations anticipating the variability of the soil and hydrogeologic conditions.

Dewatering Issues

Preliminary dewatering analyses indicate groundwater drawdown of the upper aquifer could be supplemented using the existing dewatering system for the two adjacent clarifiers. Further, boring B-2 was constructed as a pumping well to facilitate aquifer testing and to supplement dewatering at the site during construction. By monitoring groundwater elevations and the elevation of Indian Creek during precipitation events, it became apparent that the upper system will be influenced by precipitation as well as changes in elevation of Indian Creek to an unknown extent. However, it is our opinion the low hydraulic conductivity of the onsite soils within the upper aquifer could allow dewatering of the site without removing the influence from Indian Creek. As groundwater is drawn down adjacent to the clarifier, the hydraulic gradient in the vicinity of the dewatered area will increase, thus increasing the influence of precipitation, groundwater gradient, and Indian Creek. Sheet piles, slurry cut off walls, or other methods of removing influence from groundwater gradient and Indian Creek could be utilized at the site, but may not be warranted from a cost perspective.

The majority of construction appears to be planned within the upper aquifer. Wells that penetrate the clay layer have the potential to allow artesian infiltration to the upper aquifer, which may complicate dewatering above the clay layer. However, our evaluation of site conditions indicate it will be necessary to remove some artesian pressure from the lower aquifer in order to reduce the potential for heave or breaching of the soil at the base of the excavation. Based on abandoned boring B-2 data, site research and discussions with WWTP personnel, we estimate that artesian pressure acting at the base of the clay layer is equivalent to between a 30 and 32-foot-tall column of water. Artesian pressure of approximately 1,900 psf is estimated to be acting at the base of the clay layer.

Dewatering during high groundwater periods (October through April) will require an increase in dewatering quantities and time required to dewater both aquifer systems, which reflects the importance of planning the construction during winter or low water table periods. A specific dewatering design will be required that incorporates the construction schedule, groundwater levels during construction, methods of dewatering and other schedule and construction specific considerations. It will be the contractor's responsibility to develop a detailed dewatering regime for their capabilities and anticipated equipment, schedule and construction approach. The dewatering criteria presented in later sections may be referenced, but should not be relied on by the contractor to develop a specific dewatering plan due to the limited data.

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 that dewatering will occur during low groundwater periods (October through April). Groundwater elevations for the upper aquifer that were encountered during recent exploration appear to have been at low levels. 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. The lower aquifer was modeled as a confined system beginning at the base of the clay layer, with artesian pressure that rises to approximately 30.0 to 32.0 feet above the base of the clay layer. The aquifer test analyses assume fully penetrating wells pumping at a maximum constant rate with no well losses, and that drawdown from individual wells are additive. 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.

The dewatering criteria presented in subsequent sections assume that all portions of the selected dewatering configuration are pumped simultaneously. We have assumed a flux boundary for Indian Creek based on our hydrogeologic analyses in the proximity of the creek. We have also included our estimate of leakage from the partially confining clay layer. To develop the dewatering criteria, we assumed Indian Creek will recharge to the system, however the influence will not be significant due to the low hydraulic conductivity of the site.

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, the influence from Indian Creek, the artesian pressure at the site resulting from disturbance at the base of excavations, and an upper aquifer groundwater gradient sloping to the north.

The following text discusses potential dewatering options, schedule and 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 options presented below for dewatering of the upper aquifer can be constructed using full-depth or iterative methods. Full-depth refers to installing the selected method of dewatering to the top or just below the top of the clay layer using one excavation or installation method. An iterative approach is typically accomplished by successive excavation to construct successive dewatering facilities. The area is dewatered and another dewatering facility is installed at a lower elevation. The process is repeated until several iterations or benches of dewatering facilities have been installed to dewater the area. Both methods are intended to achieve construction site dewatering to the required depth below ground surface.

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.

Trench Drain Option

One method to dewater the upper aquifer within the planned clarifier area is a gravity trench drain and sump pit system. The system could be constructed to the clay layer encountered at 19 to 23 feet below the existing ground surface. If the upper soil is removed to just above the static groundwater, the trench drain will require an excavation about 12 to 16 feet deep to construct the system. Alternatively, the iterative approach could be implemented. An appropriately sized perforated pipe could be placed at the base of each trench and sloped a minimum of two percent to one or several sump pits, where the groundwater could be pumped to an approved discharge location. If utilized, trench drains should completely surround the area to be dewatered and be backfilled with free-draining material.

We do not anticipate that near-vertical excavations constructed within the silty sand below the groundwater table will remain stable. Therefore, trenches must be backfilled with free-draining material to keep the trench stable and allow dewatering. A method of achieving the base of the excavation must be implemented that allows a stable side slope. Contributions from Indian Creek and the potentially leaky clay layer will affect pumping rates. Total pumping rates will also be affected by the height of the static water table during construction. Excavation below the water table will be difficult if this option is implemented. Total pumping rates to dewater the upper aquifer using this system could range between

10 and 50 gallons per minute for the clarifier excavation. Artesian influence through the clay layer will increase total pumping rates.

Well Point Option

Closely spaced well points are another 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 table between well point locations will be significantly higher than at the well. Well losses are expected to be significant for wells constructed in the silty sand comprising the upper aquifer. Due to well losses, complete dewatering of the upper aquifer may not be possible. Total pumping rates for a configuration of well points can be expected to be similar as discussed in the trench drain option section above, providing the lower aquifer does not influence the upper system.

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 in order to achieve the necessary surface area to allow dewatering and construction of the clarifier and RAS pump station. We anticipate this approach could be used for both the upper and lower aquifer dewatering approaches. However, dewatering of the upper aquifer can induce artesian infiltration from the lower aquifer, which may affect the dewatering schedule and pumping rates.

Once the groundwater has been drawn down, and the clarifier 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, runoff, or artesian aquifer seepage. Excavations extending below the water table will be unstable and flow. The contractor must prevent flowing soil (soil not dewatered) at the sides and at the base of the excavations. This dewatering option may require a long time period of pumping in order to draw groundwater levels down sufficiently to allow construction. Pumping rates will be relatively small if the lower aquifer does not influence the upper aquifer. However, sand boils and excavation heave is expected, resulting in pumping rates between 100 and 500 gpm for each large diameter well point.

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 clarifier 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 construction of the clarifier occurs. 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.

Confined Aquifer Dewatering

It will be necessary to dewater the lower aquifer at the site in order to help reduce the potential for the base of the excavation to possibly heave and to control sand boils and flows. Artesian conditions are anticipated below the clay layer encountered at approximately 28 to 30 feet during exploration. Well-graded sand is anticipated below the clay layer comprising the majority of the confined aquifer system at the site. Strata observed isolated small layers of silt and silty sand within the clay layer. Further, our research at the site indicated there may be locally variable soil conditions below the base of the clay layer. Wells constructed to dewater the confined aquifer should be screened through the entire interval below the clay. As soil conditions are expected to vary, discharge estimates for wells completed within the confined aquifer are expected to vary.

Our preliminary analyses indicate the artesian pressure from the confined system must be drawn down by no less than 15 feet in order to reduce the influence from the confined aquifer and help limit excavation base instability to isolated sand boils and localized seepage rather than possible excavation heave. This may require an extended period of pumping from the confined aquifer. The exact number of wells installed to this aquifer will require modifications specific to actual conditions encountered and the response of the aquifer to pumping. Further, standpipe piezometers should be installed within the proposed clarifier area to monitor the artesian pressure or head of the lower aquifer, and verify the head has been reduced sufficiently, prior to initiating excavation. Standpipe piezometers must be constructed to seal the annular space within the clay layer to eliminate mixing of the two aquifers in accordance with Idaho Department of Water Resources (IDWR) requirements. Excavation heave causing infiltration from the lower aquifer could complicate the dewatering options within the upper aquifer. It may be difficult to distinguish the response of dewatering the upper aquifer from influence from the artesian pressure flowing from the lower aquifer.

General Well and Pump Considerations

Establishing a successful dewatering program will be contingent on individual spacing, pumping rates and well construction if this option is utilized. 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 alluvium, which will be reflected by the actual number of wells needed, and the pumping requirements. 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, 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.

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. Total pumping rates of the lower aquifer could vary between 100 and 500 gpm, depending upon the amount of artesian pressure and the extent of dewatering of the lower aquifer that has occurred. Again, the estimated low pumping rates, particularly in the upper aquifer are related to the soil transmissivity.

Dewatering Schedule and Drawdown Verification

The schedule for dewatering the upper aquifer is fully contingent upon actual well spacings, well volumes, and the contractor's dewatering approach. Strata preliminarily modeled ten well points around the outside of the clarifier 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. While this is not a specific dewatering design, it can provide an illustration of the upper aquifer's slow response to pumping. Gravity trench drains as discussed above may require up to 30 days to achieve a mostly dewatered condition.

Strata also modeled the lower aquifer system using wells. We preliminarily analyzed six concentric wells installed and screened beyond the base of the clay layer. A pumping rate of 15 gpm per well achieved up to 15 feet of artesian aquifer drawdown in 30 days. More closely spaced dewatering locations have the potential to decrease the time required to achieve a dewatered condition for both aquifer systems.

The above estimated time periods to dewater each system illustrate the characteristics of both aquifers; however, these estimates should not be relied upon by the contractor. Actual dewatering schedules and aquifer responses will be fully contingent upon the contractor's selected well spacing, well pump volumes, well construction, or other construction approaches to dewatering. Careful pre-planning and initiation of dewatering prior to initiating excavation is required for the project in order to achieve the required drawdown in both aquifers and to protect the excavation.

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 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. Further, numerous piezometers and wells have been installed during site exploration and dewatering applications. These wells can be utilized to help monitor groundwater drawdown during dewatering. We recommend the contractor's dewatering plan outline the methods they will use to verify groundwater levels prior to initiating excavation.

DISCUSSION

Anticipated Use of Report Recommendations

The report findings and the preliminary 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. It should be noted that preliminary recommendations provided in this report are not the only geotechnical and hydrogeologic approach for the project. These preliminary recommendations have been developed based on design team interaction and a project approach that exhibits a specific balance of design risk and cost. As such, it is possible that other dewatering, excavation and construction procedures exist other than as presented in this report. The contractor should evaluate the subsurface and hydrogeologic conditions at the site specific to their capabilities and construction approach. This report is intended only as a guide to bidding contractors. The geotechnical and hydrogeologic recommendations discussed in subsequent sections should be used to help determine their applicability to the contractor's approach and the final project requirements.

Research

The baseline section (Appendix B) and geotechnical opinions and preliminary recommendations have been prepared, in part, through our review of previous exploration and our familiarity with the project. A prior evaluation was performed by Strata for a clarifier reconstruction project approximately 1000 feet east of the proposed clarifier and RAS pump station. This evaluation was referenced to supplement recent exploration and engineering analyses at the site:

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

The design and construction files for the above 1998 project were also reviewed to help understand the soil conditions exposed during reconstruction of Clarifier No. 3, and how these conditions may be interpreted to help understand effects that similar conditions may have on the proposed construction. It is our opinion this report provides supplementary information to this evaluation to understand the subsurface and hydrogeologic conditions at the site for design purposes.

Strata also interviewed Mr. Case Houson, Assistant City engineer, to help understand the geotechnical challenges present at the site. Further, Strata referenced other sources provided by Mr. Houson to help develop the subsurface profile at the site beyond borings completed as part of this evaluation. Other references utilized 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.*

Our recent site exploration and review of the above documents and discussions indicate the subsurface conditions exhibit generally consistent depositional trends as indicated on Illustrative Geologic Cross Section A-A' on Plate 4. Further, hydrogeologic aquifer parameters characterized as part of this study are relatively consistent with findings from the October 5, 2000 Kleinfelder Report. The aquifer characteristics measured during our aquifer test indicate a slightly lower hydraulic conductivity than as discussed in the Kleinfelder Report. However, it is our opinion the soil conditions within the upper aquifer will vary across the site

Discussions with MWH regarding construction of the existing clarifiers and aeration basin in 1981 and 1982 indicate dewatering of the area was reportedly accomplished by a mass excavation using localized excavation pits and large volume centrifugal pumps. Sand boils were reportedly observed at the base of the excavation, an indication of influence from the lower, artesian aquifer. Water collection galleries and sump pits were constructed to collect localized excess artesian groundwater and isolated seeps from the excavation sidewalls. Also, sheet piling was installed between Indian Creek and the mass excavation, we assume to provide protection for a breach and to control seepage from the creek.

Key Design and Construction Issues

It is our opinion there are several important aspects of design and construction that require careful consideration and planning. The apparent soil and hydrogeologic conditions at the site will require additional dewatering and construction procedures to facilitate a successful project. We are providing the following items that, in our opinion, must be addressed or discussed as part of the planning, design, construction and long-term maintenance of the project. They are as follows:

- **Two distinct aquifer systems exist at the site.** We have discussed the conditions encountered during exploration and reiterate the need for dewatering applications to address both aquifer systems. The systems appear to act independently of each other, but will interact if the clay layer is penetrated or excavation heave occurs. Excavation heave can manifest in sand boils, excavation sidewall instability and possible flooding of the excavation area.

- **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 may take a significant quantity of time. It is our opinion, both the upper and lower groundwater systems must be drawn down to sufficient levels as mentioned in this report prior to initiating excavation. Dewatering of both aquifer systems must occur prior to initiating excavation to reduce the potential for excavation instability and heave. The contractor has the option of excavating as the groundwater is being actively drawn down or achieving total 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 will have to be constructed at the base of the clarifier and possibly at the RAS excavations to collect localized perched groundwater and artesian infiltration to the excavation.

- **Design for long-term dewatering and for the clarifier and RAS floor should account for the apparent artesian conditions at the site.** It is our opinion the planned construction could cause heave and allow the artesian pressures to seep into the upper aquifer if the pressure in the lower aquifer is not controlled. This could manifest in additional artesian pressure applied to the base of the clarifier and an increased static or transient groundwater table within the upper aquifer. Project design and planning should account for the potential artesian condition with respect to clarifier slab design and long-term maintenance dewatering of the clarifier.

- **The dewatering contractor must be experienced in similar dewatering applications.** We strongly 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 and RAS, 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 clarifier and the RAS pump station.

GEOTECHNICAL OPINIONS AND RECOMMENDATIONS

Based on our understanding of the proposed construction, interpreted site geologic and hydrogeologic conditions, and results from preliminary 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. We consider planning for construction dewatering in advance of excavation construction and excavation stability to be critical to a successful, construction schedule and budget sensitive project.

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 between and beyond the test boring locations. However, the subsurface conditions will 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.

This evaluation has been prepared based on our current understanding of the proposed project. This report provides geotechnical findings and recommendations for construction and final opinions and recommendations for civil engineering, planning, design and preparation of construction documents.

Design Assumptions

We have assumed the contractor will accomplish construction by open-excavating the clarifier area following dewatering. Connecting utilities and piping between the RAS and clarifier will likely be constructed using a trench excavation and portable shoring or trench boxes. Alternatively, the proximity of the RAS pump station could allow for one dewatering design for the RAS pump station and clarifier construction area. The dewatering design for the clarifier may cause groundwater levels to be sufficiently low in order to allow excavation for the shallower RAS pump station without additional dewatering considerations. 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 pour water pressure causing local liquefaction, which may lead to both side slope and foundation soil instability of excavations. Further, our settlement estimates and geotechnical recommendations for soil parameters are contingent upon following report recommendations for compaction, site preparation and dewatering.

Site and Subgrade Preparation and Excavation Characteristics

Topsoil or soil containing significant vegetation and organics were not encountered during exploration. However, we anticipate some topsoil, containing vegetation and organics or uncontrolled fill could be encountered during construction. These soils are 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. We expect demolition will occur to remove the existing sludge bed prior to clarifier construction. The sludge material within the bed should not be allowed to contaminate the pit run gravel exposed at the ground surface in all boring locations. Demolition activity should use caution in order to help avoid disturbing the granular pit run. If the granular pit run that is anticipated below the sludge bed is disturbed, it will be necessary to recompact the soil to structural fill requirements in areas that will not be removed to construct the clarifier. Concrete debris and asphalt removed from the site as part of demolition should not be utilized as structural backfill. Boulders or cobbles were not observed during excavation. However, imported pit run or pit run encountered onsite may contain cobbles larger than 6 inches in diameter. We recommend any material larger than 6 inches be removed from structural fill prior to placement.

Site stripping, demolition, and clarifier excavation can commence and continue to one foot above the static groundwater table as dewatering proceeds. Excavation to achieve the clarifier subgrade should not extend into saturated soil. Thus, a minimum of one foot of dewatered soil must be maintained above the static groundwater level during dewatering and excavation. Once the clarifier subgrade has been achieved, the groundwater must be maintained two feet below the subgrade during clarifier construction.

As soil is removed to allow clarifier construction, the static forces resisting ground heave will reduce. As such, the potential for the base of the excavation to heave due to artesian pressure is moderate to high. We expect partial release of artesian pressure to manifest as heave in the form of sand boils and flowing soil at the base of the excavation. Excavation should be terminated immediately if heaving or extensive sand boil conditions are observed. Following termination of excavation, the artesian pressure should be controlled through localized dewatering or installing additional lower aquifer dewatering features to control the water where the unstable excavation occurs.

We anticipate excavation within the clay soil will be necessary to achieve the piping invert elevations beneath and at the center of the clarifier. The potential for heave is greatest at this point in the excavation. The contractor should maintain contingency plans to rapidly remove water that may infiltrate the clarifier excavation from the lower aquifer. Sand or clay boils observed at the base of the clarifier or pipe excavation may require over-excavation and backfill with drain rock to render a stable and uniform soil foundation that will allow the clarifier to perform as designed.

We anticipate the subgrade for the proposed clarifier and RAS pump station will consist primarily of silty sand with some portions being constructed within the soft sandy clay. The subgrade should be achieved using smooth blade, tracked equipment. Soil that is disturbed during subgrade preparations should be excavated to firm soil and replaced with granular structural fill. Disturbing the native soil may result in inconsistent subgrade support conditions for foundations and slabs.

The on site soil has the potential to infiltrate the drain rock planned as part of the proposed underdrain system and for clarifier wall backfill. Therefore, we recommend placing a woven or non-woven geotextile fabric at the base of the subgrade to help prevent 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 heave potential and 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. The contractor should reference Appendix B for an interpretive description of the subsurface conditions that may be encountered during construction.

Wet Weather/Wet Soil Construction

The onsite silty 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 subgrades and 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 depth 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 preparations 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 excavations 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 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. 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 and RAS pump station 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 center of the clarifier and RAS pump station. 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 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, providing the site has been dewatered as recommended. Given the relatively loose condition of the silty 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 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 at 1.5:1(H:V), 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 adjacent to, and 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.

The interpreted subsurface conditions, as indicated on Plate 4, and the engineering properties of the soil, will have the potential to vary. We recommend geotechnical assessment of the soil conditions during construction to maintain project safety and production. The assessment may take the form of qualitative, visual observations of the general soil conditions and performance as the soil is exposed. This may also include obtaining soil samples for laboratory testing and analyses, consulting with the project contractor and their operators relative to excavation ease (or difficulty), constructability and other safety issues. The contractor may use OSHA as a resource to provide periodic advice and to address questions or concerns.

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, ML or CL soil as designated by the Unified Soil Classification System, Plate 3. Granular structural fill should consist of crushed well-graded, sand and gravel classified as GW or GP by the Unified Soil Classification System and contain less than 10 percent passing the #200 sieve. Structural fill should consist of particles no larger than 6 inches in diameter. Granular drain rock should have particles no larger than three inches and should be a washed product capable of free drainage. The on site silty sand may be reused as structural fill providing it is moisture conditioned sufficiently to allow the contractor to achieve compaction requirements. On site soil containing vegetation, organics 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.

Backfilling should be accomplished in accordance with MWH project specifications We recommend structural fill be placed in maximum twelve-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 D698 (Standard Proctor), or to 65 percent relative density based on ASTM D4253 and D4254 if the material contains more than 30 percent material passing the ¾ inch sieve. If material utilized for structural fill does not have the gradation for relative compaction or relative density testing, a minimum of five complete passes should be applied to the soil using a large (five 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 evident by pumping, rutting or visual contamination of gravel placed over native soil, it will be necessary to remove the disturbed area to firm native soil and replace it with approved granular structural fill.

These compaction requirements assume large (five 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 five-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. Structural fill should never be placed over disturbed or frozen subgrades. We recommend Strata be retained to evaluate the condition and suitability of on site soil for reuse as structural fill and to monitor compaction during structural 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. The RAS pump station may be backfilled with approved structural fill. All retaining walls greater than four feet high should be designed to resist sliding, overturning, bearing and slope stability failures.

Foundations

We anticipate the underdrain subgrade and bearing soil for RAS pump station and ring wall foundations of the clarifier will consist of silty sand encountered in borings B-1 through B-4. Isolated layers of well-graded sand or sandy clay may be encountered at the footing bearing elevation. Subgrades for deep pipe utilities are expected to consist of the sandy clay layer. We recommend all footings bear on a minimum of 12 inches of granular 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. We recommend footings bearing on granular structural fill over undisturbed soil as described above be designed utilizing an allowable bearing pressure of 1,500 pounds per square foot (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 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 inches. 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 foundations to maintain an undisturbed condition prior to placing granular structural fill.

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 5, Perimeter Drain Detail and as discussed in the Permanent Dewatering Section of this report.

We have discussed the potential for excavation heave to occur and for the upper aquifer to be influenced by the artesian pressure at the site. Due to the proximity of excavations to the base of the clay layer, it is our opinion that following completion of construction, artesian pressure will continually infiltrate the upper aquifer. We recommend structural design of the clarifier slab account for potential artesian conditions during normal system operation. The artesian pressure at the base of the clarifier slab when the clarifier is in full operation is estimated to range between 50 and 150 psf for the water surface at elevation 2453.4. If the site groundwater is not drawn down to below the clarifier slab prior to maintenance, hydrostatic pressure of up to 1500 psf may act on the base of the structure. The clarifier and RAS pump station structures should be designed to resist this hydrostatic pressure with respect to slab and footing structural design and buoyancy effects. The artesian pressure is a result of imbalanced water pressure between the artesian conditions at the base of the clarifier slab and the static water surface in the clarifier. If the water surface of the clarifier drops below design level due to fluctuations or malfunctions of the system, the pressure imbalance will be greater and more pressure may be applied to the clarifier floor slabs. We suggest the City consider an automated system be established that initiates the underdrain system if the interior water surface elevation in the clarifier falls below design level as a result of system malfunction or fluctuations. Alternatively, the system could be designed to resist buoyancy forces if the interior water surface falls below design elevation without the underdrain system being activated.

Lateral Earth Pressure and Coefficient of Friction

All retaining and foundation wall systems should be designed to resist lateral earth pressure from the retained soil behind the structure and surcharge from equipment, slopes or vehicles adjacent to the walls. We recommend a coefficient of friction of 0.35 be used for footing and wall design for concrete cast directly on the silty sand or sandy clay. Concrete cast directly on granular structural fill may use a coefficient of friction of 0.50 for design.

We recommend lateral earth pressures for conventional wall systems be estimated using the following equivalent fluid pressures from Table 2.

Table 2. 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.

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. Clarifier walls will be subject to load influences from adjacent equipment structures and foundations. Depending on actual static or dynamic loads, surcharge loads greater than 15 feet away from the wall will have negligible internal effect.

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 2000 International Building Code (IBC) will be utilized for structural design. Section 1615 of the 2000 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 2000 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 2000 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 five percent critical damping ratio in accordance with the IBC, Section 1615. 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.

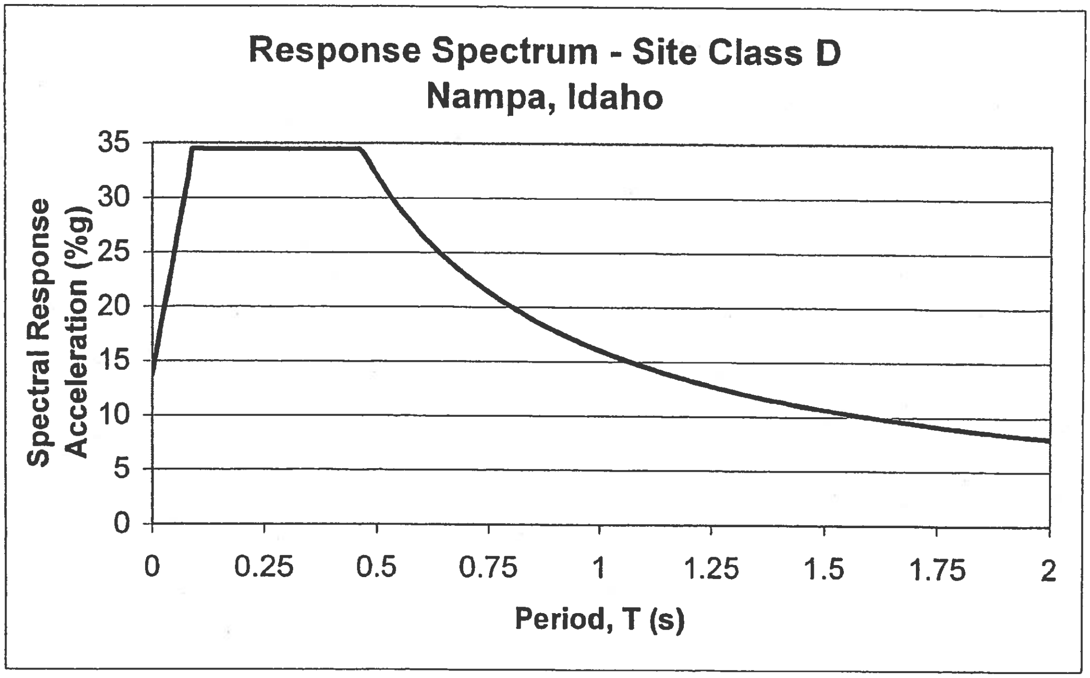


Figure 1. Spectral Response Acceleration

For engineering design, reliability-based accelerations can also be selected according to the "National Seismic Hazard Maps" (Frankel, et al., 1996) published by the U.S. Geological Survey. For Nampa, Idaho (zip code: 83687), these maps recommend the following values for the peak horizontal ground acceleration (PGA) and the spectral accelerations (for 5 percent critical damping ratio) corresponding to three different periods:

Table 3. Spectral Response Accelerations

NAMPA, IDAHO - ZIP CODE 83687			
Type of Acceleration	Probability of exceedence in 50 years		
	10 Percent (RP of 500 years)	5 Percent (RP of 1,000 years)	2 Percent (RP of 2,500 years)
Peak Ground Acceleration	0.066g	0.09g	0.14g
Spectral Acceleration at 0.2 seconds	0.15g	0.20g	0.32g
Spectral Acceleration at 0.3 seconds	0.13g	0.18g	0.28g
Spectral Acceleration at 1.0 second	0.047g	0.06g	0.096g

For example, if one uses a PGA value of 0.066g, there is a 10 percent chance that this value may be exceeded during the next 50 years. Alternatively, this value corresponds to a return period (RP) of about 500 years. The above accelerations are for sites that correspond to a shear wave velocity of about 2500 ft/second. The above values may be slightly larger at the Nampa WWTP site due to a localized response of the soil profile above bedrock.

Strata performed a liquefaction triggering analysis for the silty sand encountered during exploration. SPT N_{60} values obtained during exploration were corrected according to a procedure developed by Seed and Idriss (1971, modified). The National Seismic Hazard Maps (Frankel, et al., 1996) published by the USGS were referenced for probabilistic based peak ground accelerations. The Cyclic Stress Ratio (CSR) can be used to perform the triggering analyses and is defined as a measure of the force that is applied to the soil during earthquake loading. The CSR was developed using the peak ground accelerations as mentioned above, using the Seed and Idriss 1971 modified procedure. $N_{1(60)}$ values were obtained using field SPT N_{60} values corrected based on overburden stress, rod length, fines content and boring diameter. The CSR was compared to the $N_{1(60)}$ value using a graphical reference to evaluate the potential for liquefaction to be triggered at the

BASELINE INTERPRETIVE SECTION

This baseline interpretive section discusses the assumed geotechnical site conditions for the proposed Clarifier and RAS Pump Station at the Nampa Waste Water Treatment Plant (WWTP). This report attachment is intended, in part, to assist the selected contractor in preparing the anticipated construction means and methods relative to proposed construction. Further, this segment may supplement the contractor's evaluation of their anticipated project approach, such that the owner and design team can evaluate the contractor's approach and understanding of the apparent subsurface conditions at the site.

Based on field exploration results, we subdivided the soils into four general units; pit run gravel, silty sand, sandy lean clay, and well-graded sand. These units are referenced on the boring logs and Plate 4, Illustrative Cross Sections A-A' to B-B':

Pit Run Gravel

Imported Pit run gravel was encountered at the ground surface in all borings and was observed throughout the proposed construction area. The pit run sand and gravel typically extended to between 3 and 4 feet below the existing ground surface. This material was observed to be in a loose to medium dense state with near saturated soil conditions due to heavy precipitation. Particles were typically well rounded and the soil contained varying quantities of silt and sand.

Implications

Depending on the time of year construction occurs, the pit run sand and gravel may be near saturated or in an over-optimum moisture condition. Vertical excavations into this material which are less than 4 feet may experience local sloughing and caving.

Silty Sand

This silty sand encountered in all borings exhibited variable degrees of relative density. SPT blow counts performed within this soil unit varied between 5 and 35. Some gravel was encountered within the silty sand in boring B-1, however gravel was not observed in other borings. The silty sand varied with respect to silt content, and well graded sand lenses were observed within the soil unit. Silty sand was encountered below pit run sand and gravel and typically extended to between 18 and 23 feet below the existing ground surface. During the exploration process, the silty sand tended to heave into the augers as groundwater flowed into the 8-inch hollow stem auger core.

Implications

This soil unit has the potential for excavation instability if hydrostatic conditions are present. Excavations performed below the groundwater table within the soil unit will experience flowing soil and generally unstable conditions. Portions within the soil unit having lower silt contents will generate higher volumes of groundwater during dewatering. Alternatively, portions of the silty sand that contain high silt content will have the potential to perch groundwater during dewatering operations. Further, this soil experienced variable grades of relative density. Loose areas within this soil unit may require localized shoring or a flatter slope. We anticipate this soil will comprise the majority of the upper aquifer. Conventional dewatering wells installed within this soil unit may experience relatively high well loss. Vibratory compaction performed over this soil unit could cause localized liquefaction and soil disturbance.

Sandy Lean Clay

The sandy lean clay encountered in all borings was described as brown, very soft to soft and saturated. The sandy clay exhibits low to medium plasticity and contains up to 40% medium sand particles. However, the behavior of the soil will be controlled by the medium plasticity clay. As previously discussed, the clay layer is extremely soft in isolated locations. This layer comprises the confining layer between the two aquifers as discussed in the report text.

Implications

Excavations performed within the sandy clay layer have the potential for instability. The material is easily disturbed by even light construction equipment. Large excavation equipment such as trackhoes, backhoes, or heavy trucks have the potential to deeply rut the material and cause significant disturbance. Due to its low to moderate plasticity and potential artesian conditions at the site, this material has relatively high potential for heave at the base of excavations. Further, this material is saturated and will be difficult to utilize as structural fill at the site. Compaction efforts utilizing granular material placed above the clay may experience pumping conditions for the first layer of structural fill placed above the clay. This material also has the potential to contaminate granular material as groundwater fluctuates between soils. This material typically has relatively poor support characteristics for pipe bedding and other construction activities. Granular fill may be required to increase the performance of subgrades achieved within this layer.

Well Graded Sand

The well graded sand was encountered in all borings below the sandy lean clay. Well graded sand was described as tan, loose to medium dense and saturated.

Implications

Based on our understanding of the proposed construction, it is our opinion this material may not be exposed during construction. The well graded sand has the potential to generate larger volumes of artesian groundwater. If excavations performed at the center of the clarifier expose this material, artesian pressures may be encountered and this material will flow readily. It may be necessary to expose this soil in order to complete dewatering of the lower aquifer. Local variations including silty sand and well graded sand with gravel may be encountered below the clay layer.

Groundwater

We have previously discussed the hydrogeologic conditions at the site within the main report. Based on exploration results and review of construction files and other geotechnical reports, it is our opinion the area for the proposed clarifier consists of an upper and lower aquifer. The upper aquifer had a saturated thickness of approximately 13.5 feet during exploration. The groundwater gradient varied locally, but it generally viewed as sloping north towards Indian Creek. The upper and lower aquifers are separated by a partially confining layer comprising the sandy lean clay as described in the sandy lean clay section above. Artesian conditions exist within the lower aquifer as indicated by exploration results and our review of the construction files for past projects at the site. The reason for the artesian conditions is not fully understood. Based on our aquifer field testing and hydrogeologic analyses, it does not appear the confining, sandy lean clay layer allows significant influence to the upper aquifer as a result of artesian pressure from the lower aquifer. If excavation heave occurs, the clay later will allow artesian influence to the upper aquifer. If heave does not occur, the two systems should be seen as operating independently of one and other. Due to the nature of the low hydraulic conductivity of the upper aquifer, it is our opinion that dewatering performed at the site will require a relatively long pumping or dewatering schedule. Groundwater does not readily leave the pore space of the silty sand comprising the upper aquifer. An aggressive, iterative dewatering process for the upper aquifer should be anticipated. The lower aquifer will also require an extended pumping period to reduce artesian pressure at the base of the clay layer.

Limitations

The above discussion presents baseline statements for the anticipated subsurface conditions that may be encountered during construction. These baseline conditions were developed as a result of previous geotechnical work in the area, geologic research, and recent site exploration. This baseline section is an interpretation of the site subsurface conditions and does not expressly present the actual subsurface soil conditions at the proposed construction. The above description of the subsurface conditions should not be construed as a guarantee or warranty the above conditions will be encountered during construction. Further, specific ground behavior is contingent upon the selected contractors construction approach and actual means and methods of excavation and site preparation methods including dewatering.

motion indicate that there will be a 50 percent probability that liquefaction will be triggered (falls on the line). Peak ground accelerations for the 2,500-year return period will likely trigger liquefaction. It is Strata's opinion that return periods of 1,000 and 2,500-years have a small probability of occurrence for ground motion to occur relative to the operational life of the structure. Further, most SPT N_{60} values obtained at the site are higher than the worst-case SPT value utilized for analyses, which suggests that liquefaction will only occur locally in this soil.

Permanent Dewatering

We understand it will be necessary to periodically empty the clarifier for cleaning or maintenance. Emptying the clarifier will cause an imbalance of water pressure at the base the slab and walls due to a high groundwater level outside the structure. A permanent drain system is required 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 clarifier be dewatered to a depth of at least 6 inches below the base of the clarifier slab.

We recommend underdrains be constructed in concentric circles starting approximately 10 feet, radiating from the center pivot and at an approximately 20 to 25-foot radial spacing. A perimeter underdrain should be located outside ring wall footings as outlined on Plate 5, Perimeter Drain Detail. The wall backfill should include a minimum of one foot of drain rock placed to within six feet of the final finished ground surface and connecting to the perimeter underdrain trench as shown on Plate 5. 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 clarifier slab and be sloped a minimum of 1 percent to connect to the manifold or collection discharge pipe to remove water that infiltrates the drain rock. We anticipate the perimeter underdrain and interior underdrains may be installed deeper below the clarifier slab to meet grades and pipe invert requirements near the center of the structure.

design. Asphalt concrete should meet ITD Class A or B 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. Asphalt construction and final surface smoothness, joints and density should meet ITD specifications.

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 two 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

It is our opinion 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. Also, we recommend a pre-construction survey be completed on all nearby structures that are considered to be potential candidates for disturbance, settlement or other adverse performance associated with the planned construction.

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 warranties, either expressed or implied.

The following plates accompany and complete this report:

Plate 1:	Vicinity Map
Plate 2:	Site Plan
Plate 3:	Unified Soil Classification System (USCS)
Plate 4:	Illustrative Geologic Cross Sections A-A’ and B-B’
Plate 5:	Perimeter Drain Detail
Plate 6:	Proposed Clarifier Underdrain System Typical Section
Appendix A:	Exploratory Boring Logs
Appendix B:	Interpretive Baseline Section
Appendix C:	Laboratory Test Results

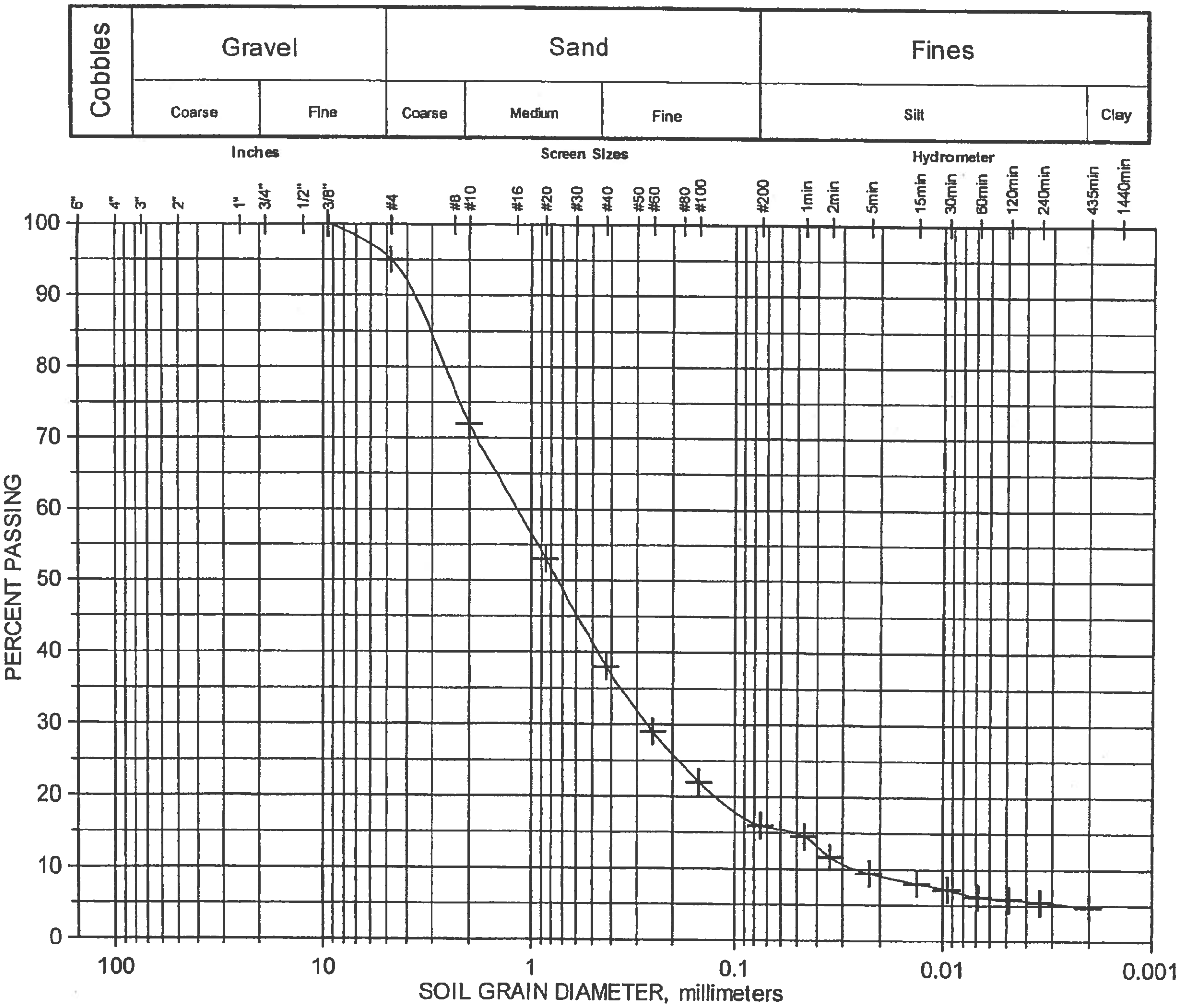
APPENDIX A
Exploratory Boring Logs

APPENDIX C
Laboratory Test Results

GRADATION ANALYSIS

ASTM D422

Project: Nampa WWTP
Client: MWH
File: MONWAT B04012A
Sample No: B4L0234
Sample Location: B-1 @ 12.0'
Description: Silty Sand w/Trace Gravel
Date Received: 2/17/04
Date tested:3/4/04 by RC

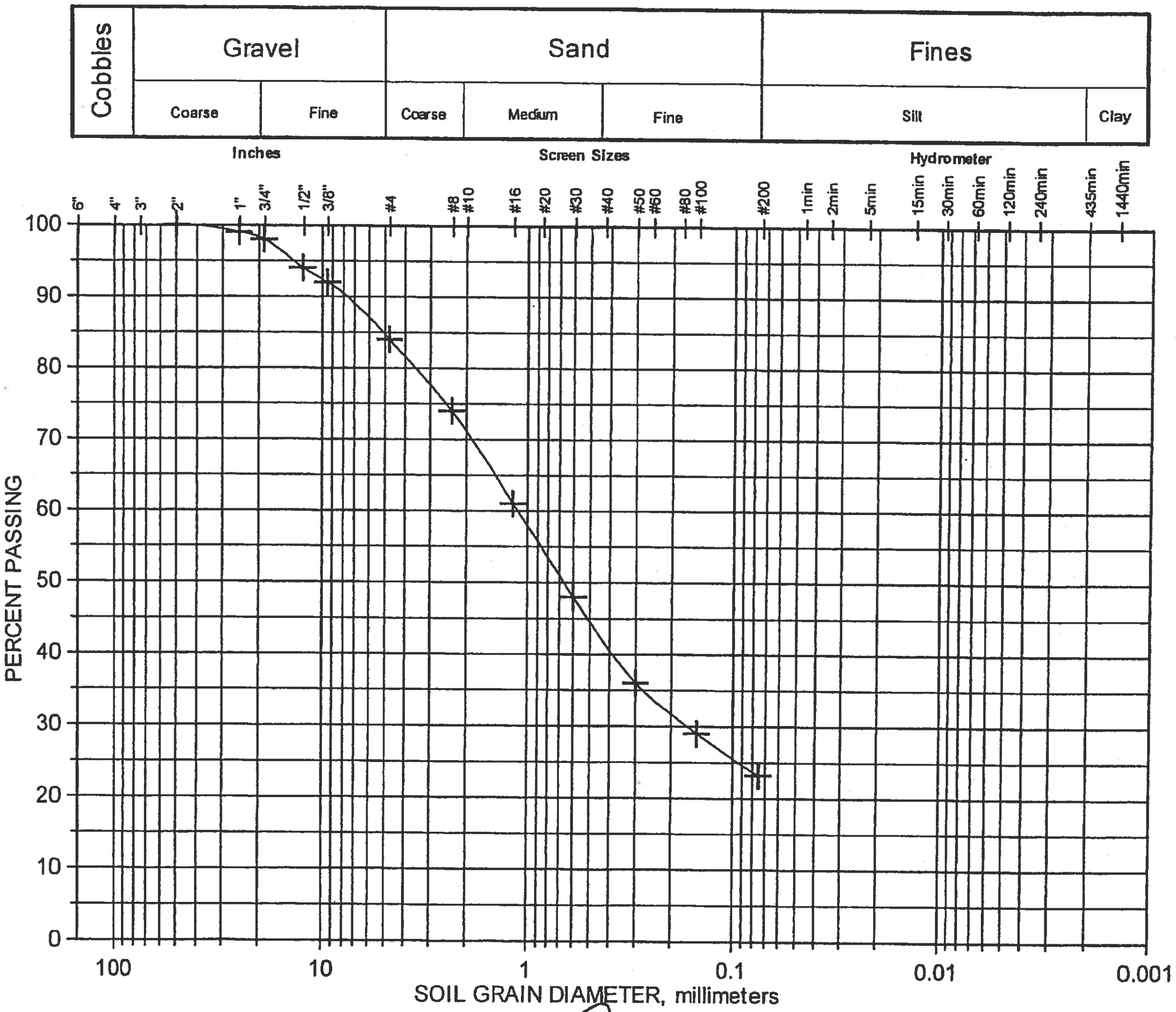


Reviewed by:

GRADATION ANALYSIS

ASTM D422

Project: Nampa WWTP
Client: MWH
File: MONWAT B04012A
Sample No: B4L0235
Sample Location: B-2 @ 8.0'
Description: Silty Sand w/Trace Gravel
Date tested:2/24/04 By:tc

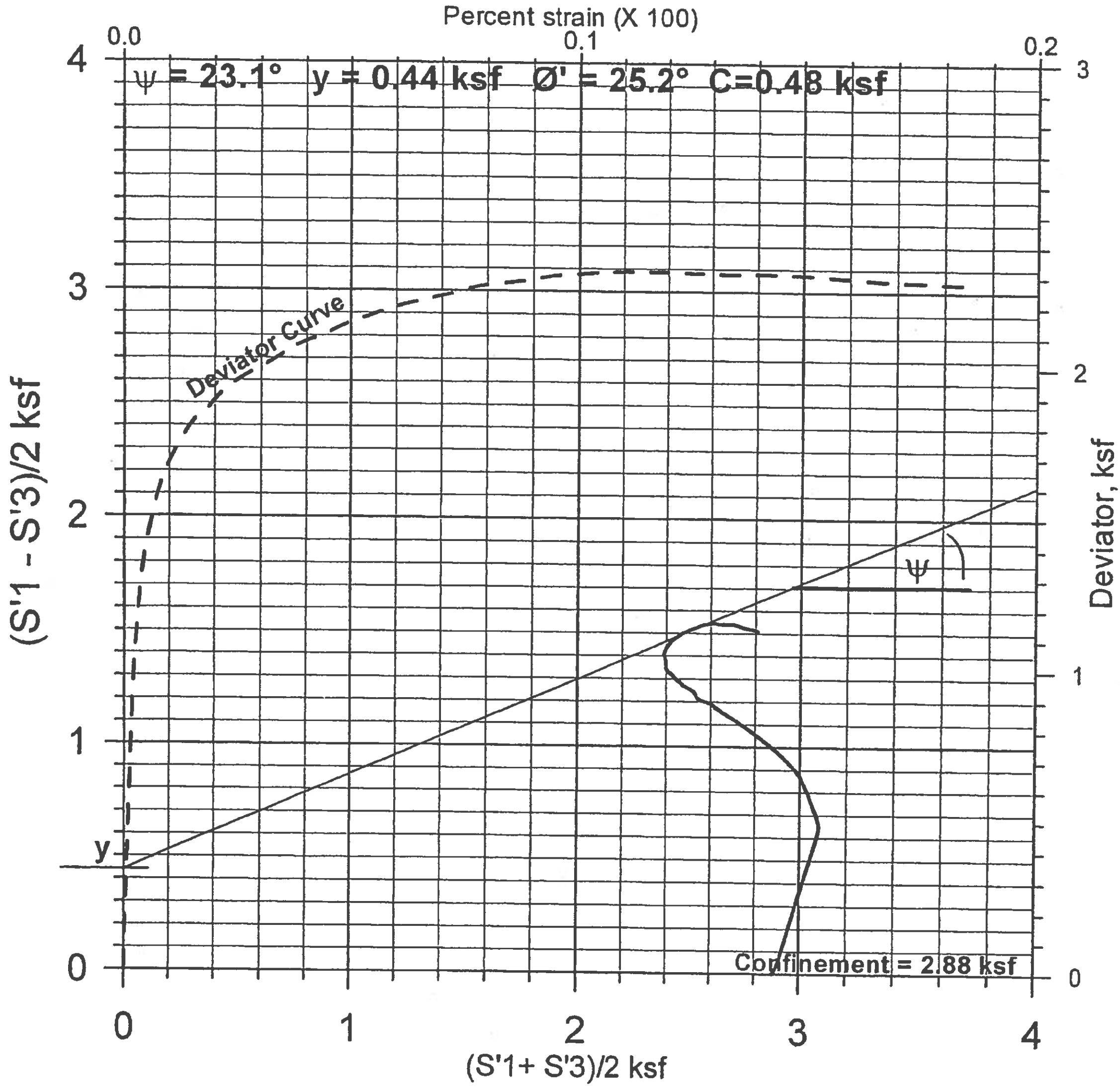


Reviewed by:

STRESS PATH
ASTM D4767 CU

Project: Nampa WWTP
Client: MWH
File No: MONWAT B04012A
Sample: In Situ Rings (condition:good)
Location: B-3 @ 26.0' - 26.5'
Dry Density: 79.6 pcf @ 40.4% moisture content
Soil Type: Sandy Lean Clay (CL)
Sample#: B4L0477 Tested 3/19/04 tc

Atterberg Limits
LL = 38
PI = 19



Reviewed by:

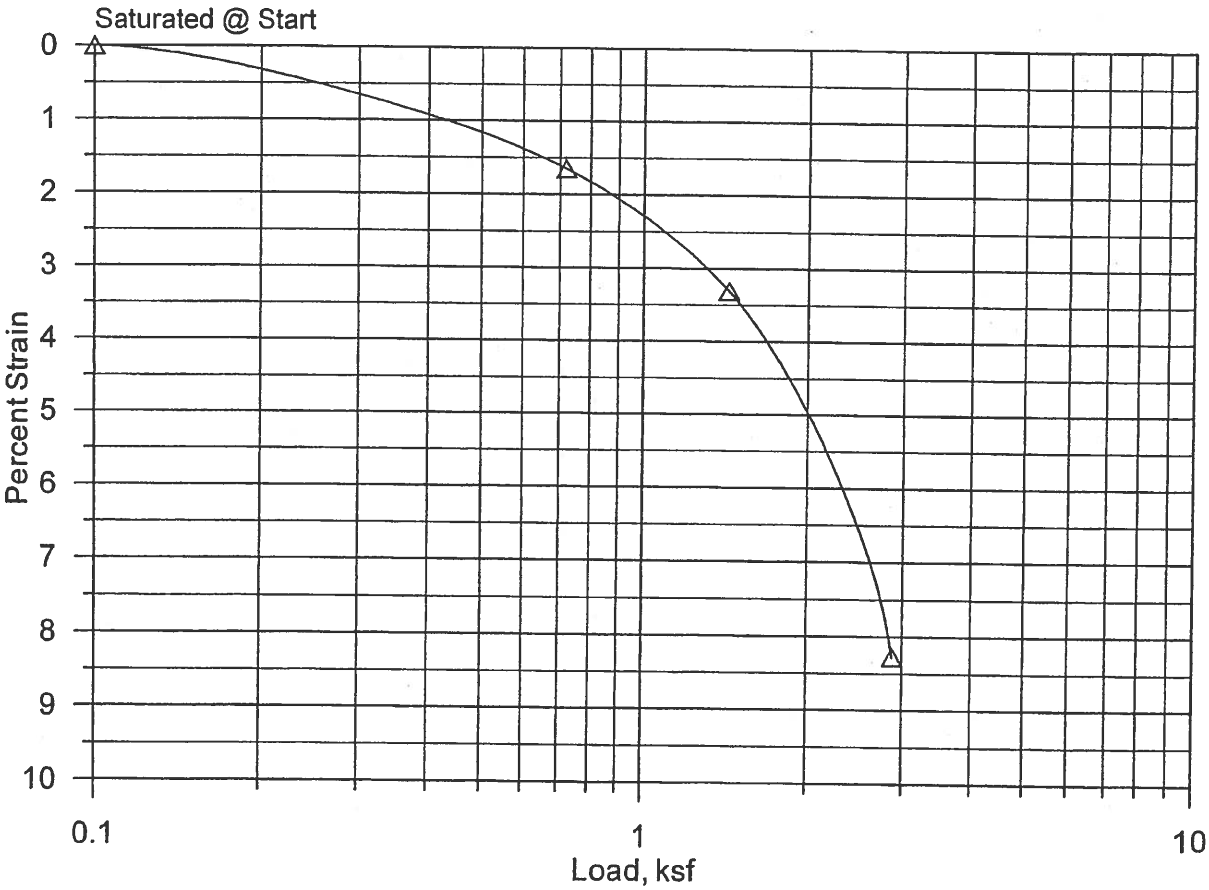
STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING

CONSOLIDATION TEST RESULTS

ASTM D-4767
(Triaxial Cell)

Project:Nampa WWTP
Client: MWH
File Name: MONWAT B04012A
Date Tested: 3/16/04
Sample Number: B4L0477
Sample Location: B-3 @ 26.0' - 26.5'
Sample Description: Pending
Initial test conditions: In-Situ (Rings)
Water Content:40.4%
Dry Unit Weight:79.6 pcf

Atterberg Limits
LL = 38
PI = 19



Reviewed By

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING

Summary of Test Results

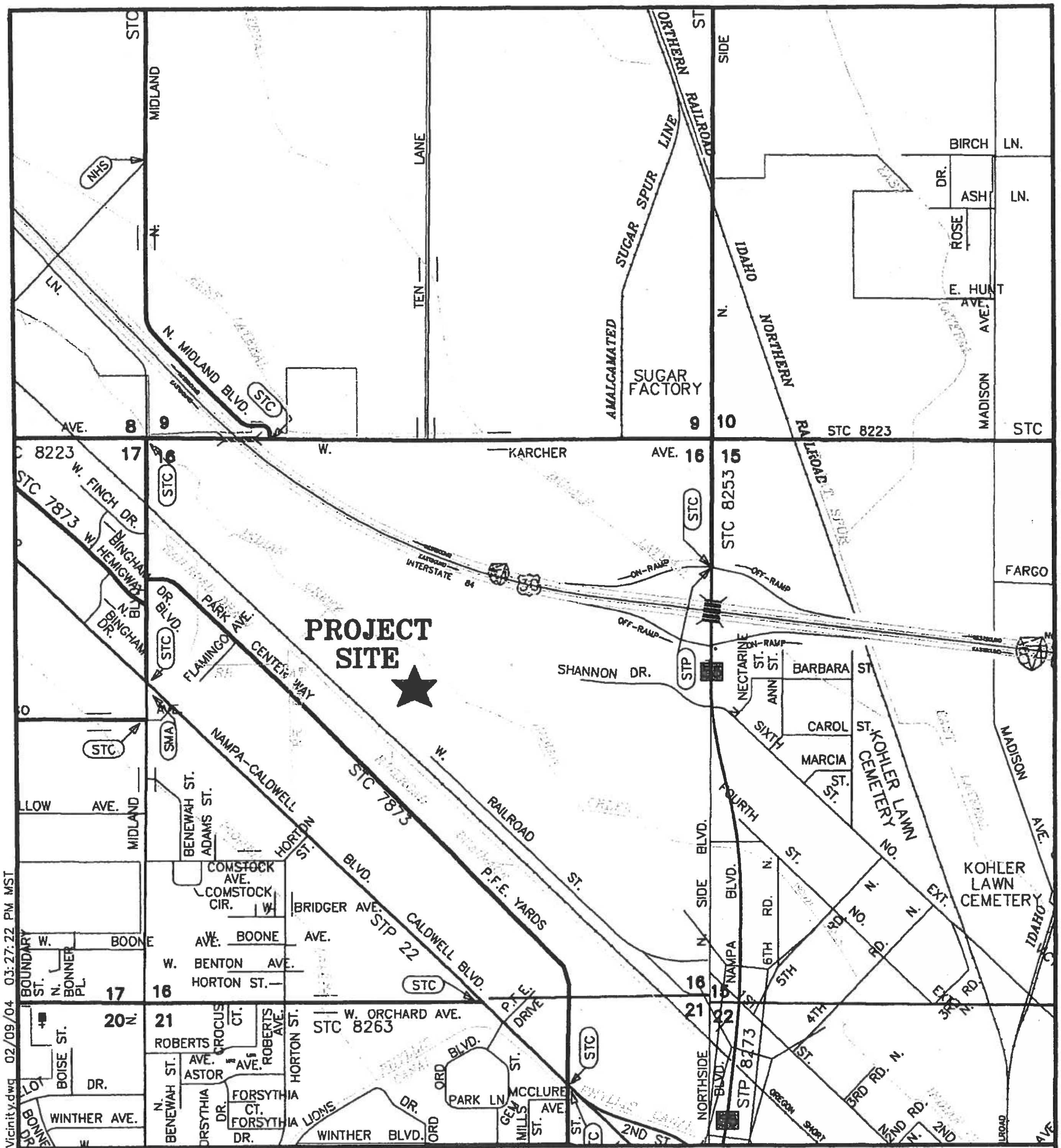
Project: Nampa WWTP
Report to: MWH

Report Date: 3/22/2004
File Name: MONWAT
Project No.: B04012A

Test Pit -	Depth - Elev.	Lab Number	Description and remarks (classification)	Insitu Dry Density, pcf	In situ Moisture, %	Passing No. 200,%	Atterberg Limits		Fines Class.
							LL	PI	
B-1	12.0'	B4L0234	Silty Sand			16*			
	25.0 - 26.5'	B4L0239	Sandy lean clay		28.4	61			
B-2	8.0'	B4L0235	Silty Sand			23*			
	20.0 - 21.5'	B4L0240	Silty Sand		15.2	14			
	25.0 - 26.5'	B4L0242	Sandy lean clay				38	19	CL
B-3	25.0 - 26.0'	B4L0238	Sandy lean clay		45.4	78			
	26.0 - 26.5'	B4L0477	Sandy lean clay	79.6	40.4	78			
	30.0 - 31.5'	B4L0241	Well-graded Sand		21.0	6.9			
B-4	25.5 - 26.0'	B4L0236	Sandy lean clay	81.2	41.9				
	26.0 - 26.5'	B4L0237	Sandy lean clay			63			

* See Gradation

Reviewed by: ______________



N.

0 1/8 1/4 3/8 1/2

SCALE IN MILES

VICINITY MAP

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Reference: I.T.D. Caldwell City Map

File: MONWAT-B04012A

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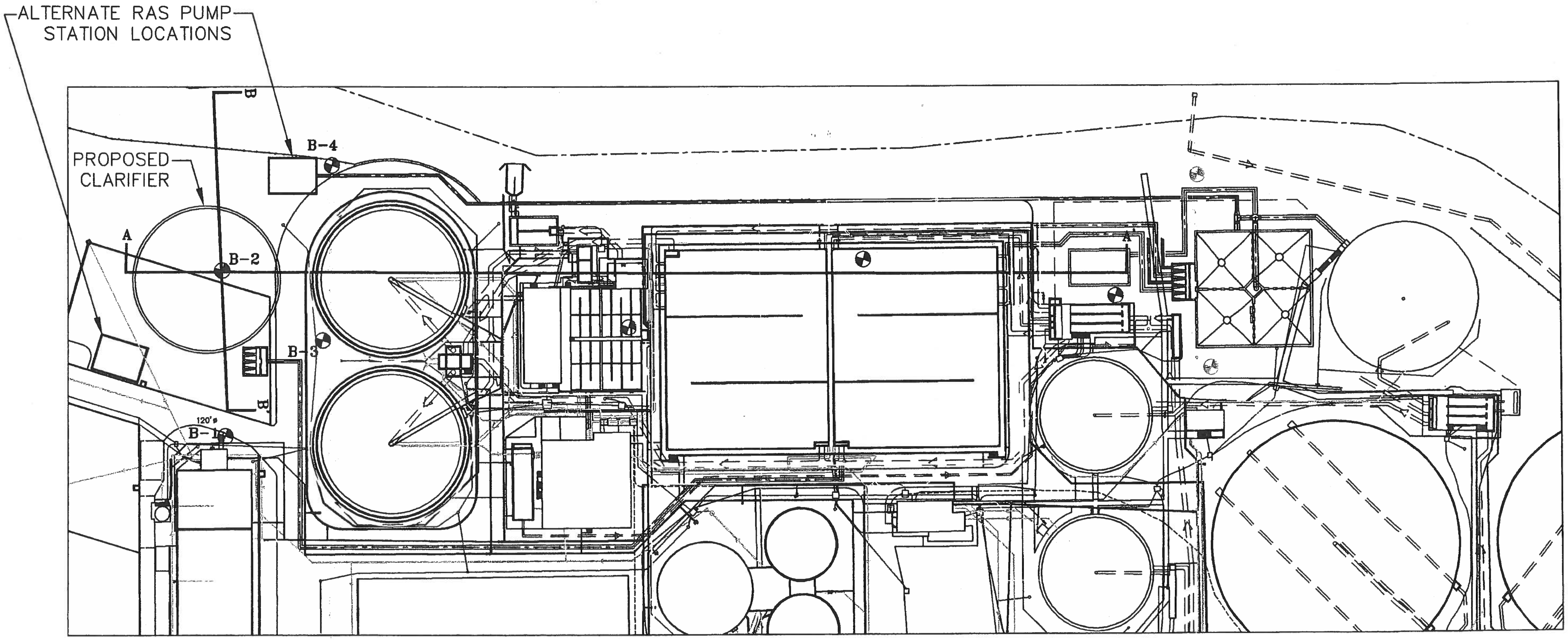

LEGEND

● B-1 Approximate Location Of Test Boring Observed By Strata, Inc. On February 17, 2004.

● "Previous Geotechnical Borings performed at the site" by CH2MHILL, 1979

● "Previous Geotechnical Borings performed by Kleinfelder, 2000

SCALE
0 50 100 150 200
1 inch = 100 ft.

N

SITE PLAN, Nampa WWTP

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING
File: MONWAT-B04012A
PLATE 2

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 MWHGlobal, "fig101A.pdf" Named "Site Plan and Topography".

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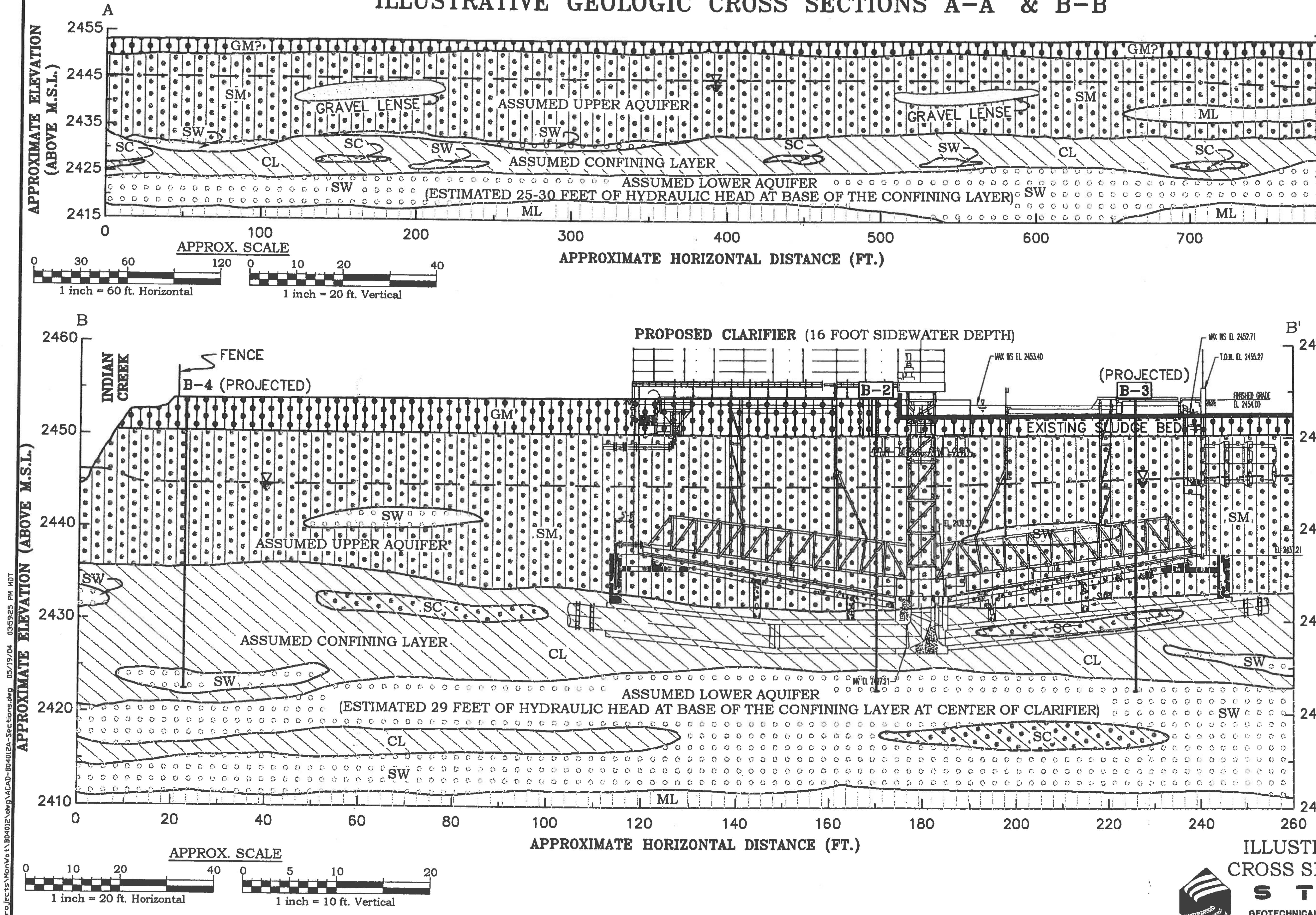
ILLUSTRATIVE GEOLOGIC CROSS SECTIONS A–A' & B–B'



LEGEND

GM	Silty GRAVEL with sand (Pit run sand and gravel fill) – brown to tan, loose to medium dense, moist.
SM	Silty SAND with gravel – dark brown to tan, loose to medium dense, wet to saturated. Some areas do not contain gravel.
SC	Clayey SAND dark brown to tan med. dense, saturated.
SW	Well graded SAND – tan, medium dense, saturated.
ML	SILT
CL	Sandy LEAN CLAY – brown, very soft to soft, saturated. Partial confining layer between two aquifers
▽	Groundwater.

ILLUSTRATIVE GEOLOGIC
CROSS SECTIONS A-A' & B-B'

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING

MONWAT-B04012A

PLATE: 4

* FOR GENERAL REFERENCE ONLY, Soil unit contacts are approximate. Clarifier approximately located based on project plans by MWH, Undated potential RAS pump station locations shown on plated site plan

Reference: MWH Global – Site Plan.

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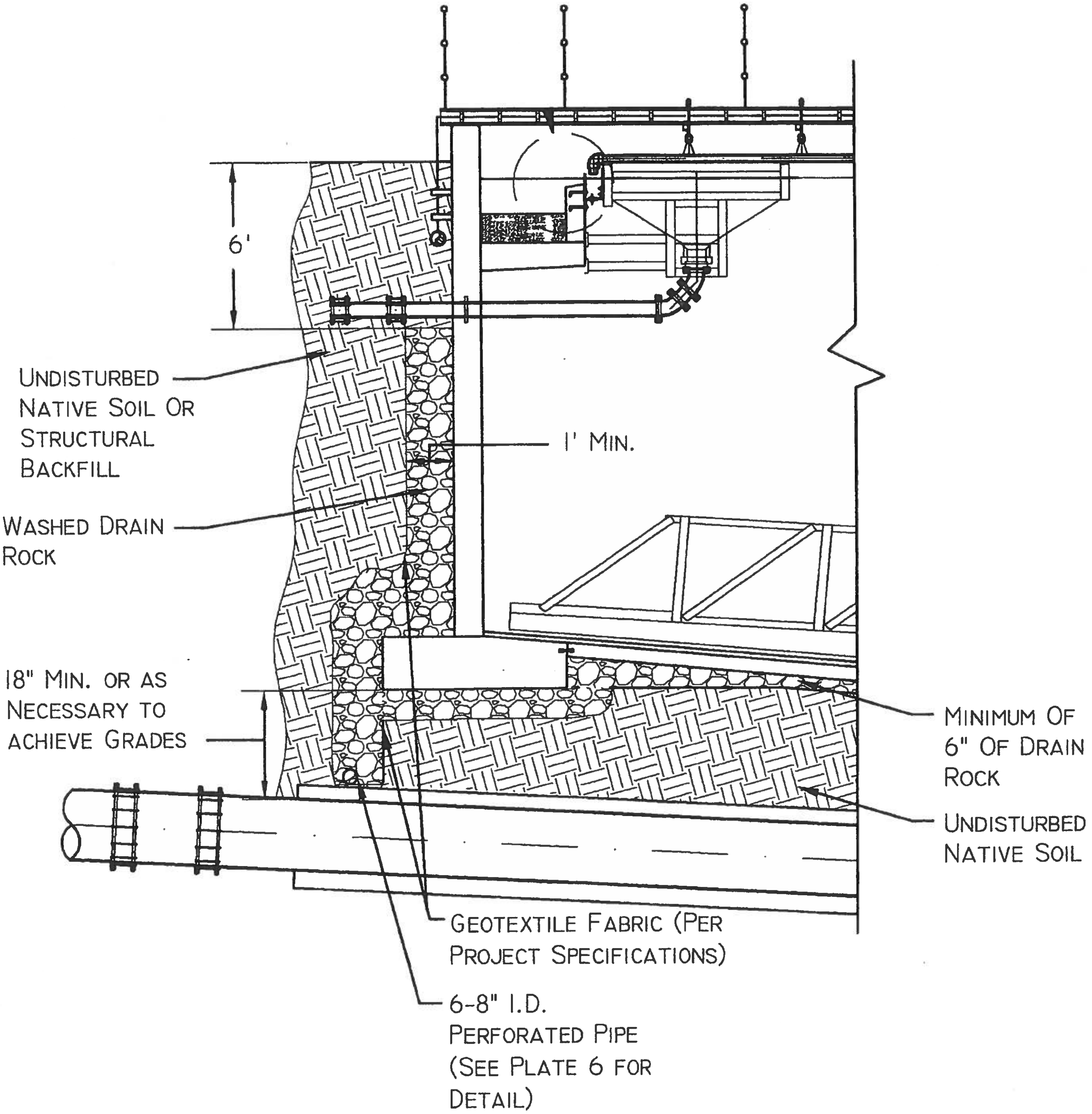

PERIMETER DRAIN DETAIL

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING
File: MONWAT-B04012A

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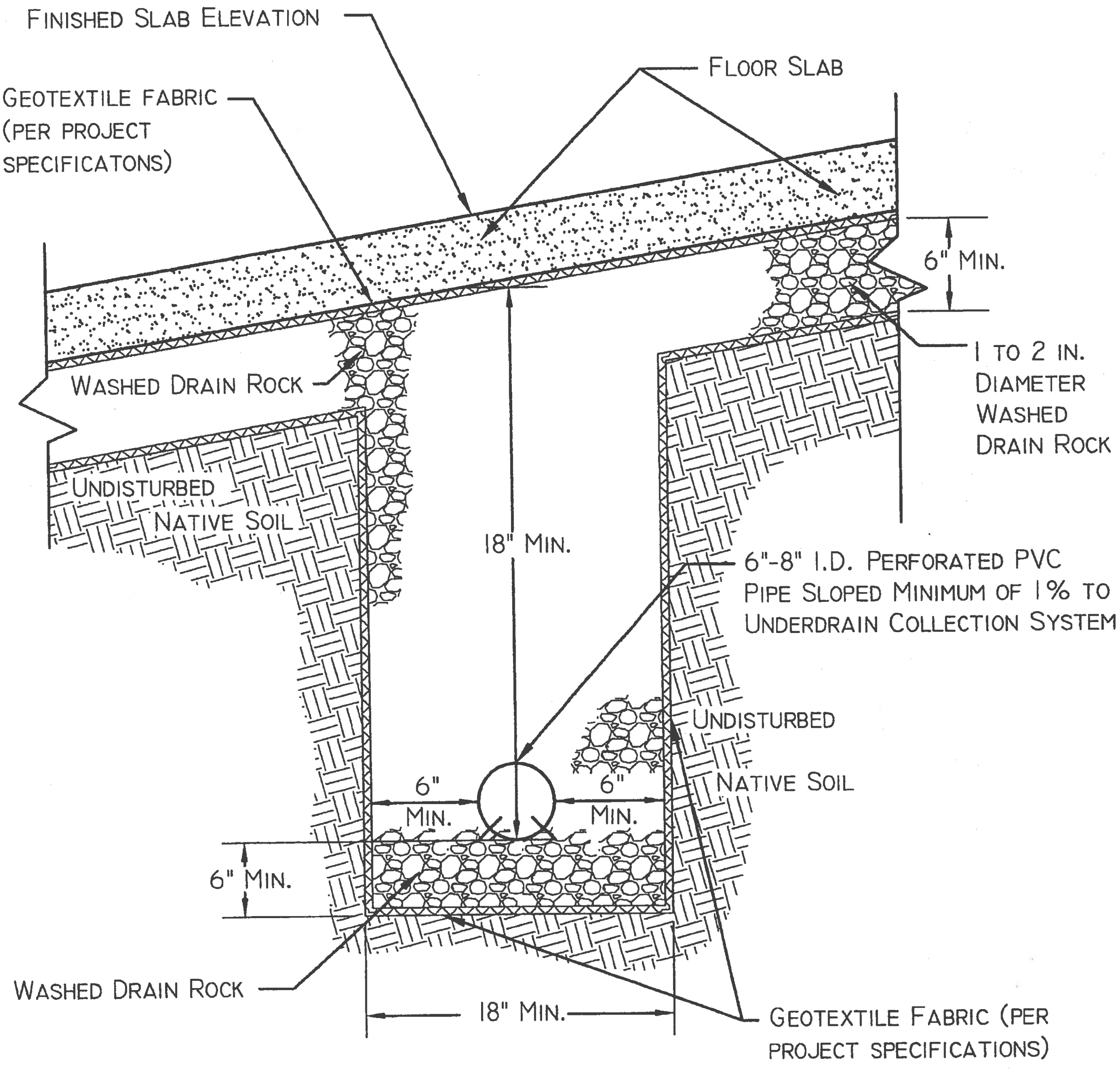

Informational Only (Not To Scale)

PROPOSED CLARIFIER UNDERDRAIN SYSTEM TYPICAL SECTION

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GEOTECHNICAL ENGINEERING & MATERIALS TESTING

File: MONWAT-B04012A

PLATE: 6

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Boring No. B-1 Subsurface Soil Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE Type	BLOWS Per 6 Inches	SPT Blows Per Foot	Moisture (%)	Dry Density (pcf)	POCKET Penetro-meter(tsf)	REMARKS
Silty GRAVEL with sand (pit run fill) – brown to tan, loose to med. dense, wet.	1 2 3	GM								
—?— —?—		?								
Silty SAND with gravel (Native) – dark brown to tan, loose to medium dense, wet to 6.6 feet then saturated.	4 5 6	SM		▮	2 4 1	5				
▽	7 8 9									
. . . except no gravel.	10 11			▮	7 12 12	24				
	12 13 14									
	15 16			▮	3 4 5	11				
	17 18									
—?— —?—	19	?								
Sandy LEAN CLAY – brown, very soft to soft, saturated.	20	CL								

File: MONWAT B04012A
Boring Number: B-1
Surface Elevation: 2453.4
Date Drilled: 02/17/2004
Drill Rig: CME 75
Boring Diameter: 8"
Depth to Ground Water: 6.6'
Logged By: C.M.C.

EXPLORATORY BORING LOGS

STRATA

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

Sheet 1 of 2

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Boring No. B-1 Subsurface Soil Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE Type	BLOWS Per 6 Inches	SPT Blows Per Foot	Moisture (%)	Dry Density (pcf)	POCKET Penetro-meter(tsf)	REMARKS
Sandy LEAN CLAY – brown, very soft to soft, saturated.	21	CL	▨	▮	1 2 1	4			0.0	
	22 23 24		▨							
	25 26		▨	▮	1 3 2	6	28%		0.0 to 0.5	Some clayey sand layers observed Resisitivity=2050 ohm–cm pH=7.5
	27		▨							
—?— —?—	28	—?—								
Well graded SAND – tan, medium dense, saturated.	29	SW	○ ○ ○							
	30 31		○ ○ ○	▮	7 10 3	13				±5" of sand heave. 2" silty sand layer observed at 31'.
Boring Terminated At 31.5'.	32 33 34 35 36 37 38 39 40									

File: MONWAT B04012A

Surface Elevation: 2453.4

Drill Rig: CME 75

Depth to Ground Water: 6.6'

Boring Number: B–1

Date Drilled: 02/17/2004

Boring Diameter: 8"

Logged By: C.M.C.

EXPLORATORY BORING LOGS

STRATA

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

Sheet 2 of 2

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Boring No. B-2 Subsurface Soil Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE Type	BLOWS Per 6 Inches	SPT Blows Per Foot	Moisture (%)	Dry Density (pcf)	POCKET Penetrometer(tsf)	REMARKS
Silty SAND with gravel (Native) – dark brown to tan, medium dense to dense, saturated.	21 22	SM			8 15 16	35	15.2			±4' Sand heave.
—?— —?— —	23	?								
Sandy LEAN CLAY – brown, very soft, saturated.	24 25 26 27 28 29	CL			0 0 0	0			0.0	Some clayey Sand layers observed Atterberg limits: Liquid limits = 38 Plasticity limits = 19
—?— —?— —?—		?								
Well graded SAND – tan, loose to medium dense, saturated.	30 31	SW			1 2 2	4				3" Clayey sand zone observed.
Boring Terminated At 31.5'.	32 33 34 35 36 37 38 39 40									

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File: MONWAT B04012A

Surface Elevation: 2451.9

Drill Rig: CME 75

Depth to Ground Water: 9.5'

Boring Number: B-2

Date Drilled: 02/17/2004

Boring Diameter: 12"

Logged By: C.M.C.

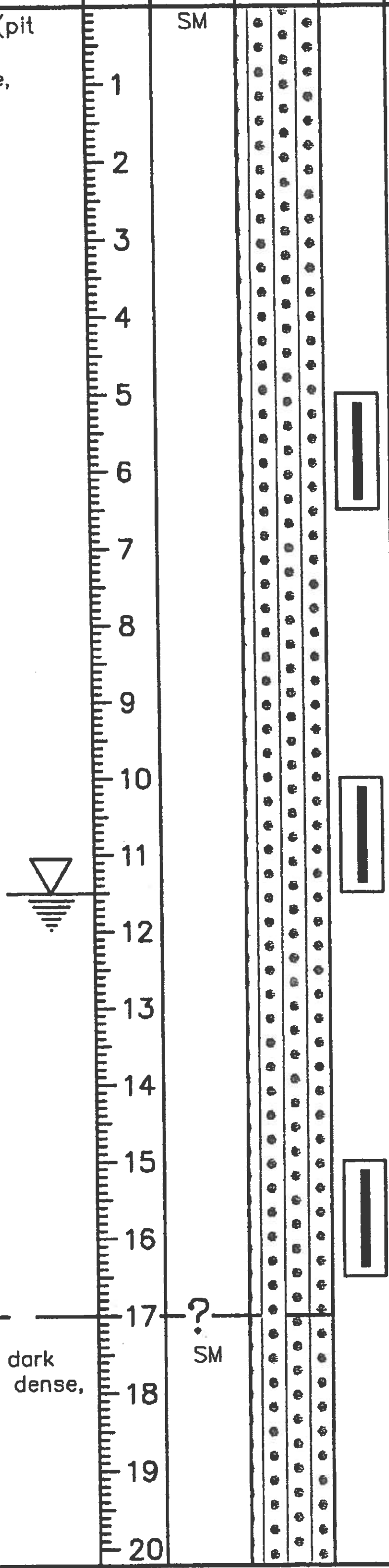
EXPLORATORY BORING LOGS

STRATA

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

Sheet 2 of 2

Boring No. B-3 Subsurface Soil Description



Boring No. B-3 Subsurface Soil Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE Type	BLOWS Per 6 Inches	SPT Blows Per Foot	Moisture (%)	Dry Density (pcf)	POCKET Penetro-meter(tsf)	REMARKS
Silty SAND with Gravel (pit run fill) – tan to grey, medium dense to dense, moist to 11.5 feet then saturated.	0	SM								
	5			▮	8 10 12	22				
	10			▮	8 13 21	34				
	11.5 ▽									
	15			▮	8 10 11	23				
?— —?—	17	? SM								
Silty SAND (Native) – dark brown to tan, medium dense, saturated.	17–20									

File: MONWAT B04012A	Boring Number: B-3
Surface Elevation: 2453.4	Date Drilled: 02/17/2004
Drill Rig: CME 75	Boring Diameter: 8"
Depth to Ground Water: 11.5'	Logged By: C.M.C.

EXPLORATORY BORING LOGS

STRATA

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

Sheet 1 of 2

Boring No. B-3 Subsurface Soil Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE Type	BLOWS Per 6 Inches	SPT Blows Per Foot	Moisture (%)	Dry Density (pcf)	POCKET Penetrometer(tsf)	REMARKS
Silty SAND (Native) – dark brown to tan, medium dense, saturated.	21 22	SM	⁘	■	6 8 11	21				
?— —?— —?—		?								
Sandy LEAN CLAY – brown, very soft to soft, saturated.	23 24 25 26 27 28 29	CL	▨	■	0 5 6	5*	45.4	79.6	0.0 to 0.5	Some clayey Sand layers observed * 3" sampler adjusted to 2" sampler (SPT) blow count.
?— —?— —?—		?								
Well graded SAND – tan, medium dense, saturated.	30 31	SW	○	■	2 6 3	9	21.0			
Boring Terminated At 31.5'.	32 33 34 35 36 37 38 39 40									

File: MONWAT B04012A	Boring Number: B-3
Surface Elevation: 2451.9	Date Drilled: 02/17/2004
Drill Rig: CME 75	Boring Diameter: 8"
Depth to Ground Water: 11.5'	Logged By: C.M.C.

EXPLORATORY BORING LOGS

STRATA

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

Sheet 2 of 2

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Boring No. B-4 Subsurface Soil Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE Type	BLOWS Per 6 Inches	SPT Blows Per Foot	Moisture (%)	Dry Density (pcf)	POCKET Penetrometer(tsf)	REMARKS
Silty GRAVEL with sand (pit run fill) – brown to tan, loose, wet.		GM								
	1									
	2									
	3									
—?— —?—	4	?								
Silty SAND (Native) – dark brown to tan, loose, wet to 9.0 feet then saturated.		SM								
	5			▮	5 5 3	8				
	6									
	7									
	8									
▽	9									
	10			▮	5 5 5	10				
	11									
	12									
	13									
	14									
	15			▮	3 3 2	6				
	16									
	17									
—?— —?— —?—	18	?								
Sandy LEAN CLAY – brown, very soft to soft, saturated.		CL								
	19									
	20									

File: MONWAT B04012A

Surface Elevation: 2453.2

Drill Rig: CME 75

Depth to Ground Water: 9.0

Boring Number: B–4

Date Drilled: 02/17/2004

Boring Diameter: 8"

Logged By: C.M.C.

EXPLORATORY BORING LOGS

STRATA

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

Sheet 1 of 2

F:\Projects\MonWat\B04012\dwg\B04012A-Boring 4 Sheet 1.dwg 5/18/2004 2:56:37 PM MDT

Boring No. B-4 Subsurface Soil Description	DEPTH (In Feet)	USCS CLASS	SYMBOL	SAMPLE Type	BLOWS Per 6 Inches	SPT Blows Per Foot	Moisture (%)	Dry Density (pcf)	POCKET Penetro-meter(tsf)	REMARKS
Sandy LEAN CLAY – brown, very soft, saturated.	21	CL	▨	■	0 0 2	1*			0.0	
	22 23		▨	☐						No Shelby sample retained.
	24		▨							
	25 26		▨	■	0 1 2	3	41.9	81.2	0.0	Some clay sand layers observed * 3" sampler adjusted to 2" sampler (SPT) blow count.
	27 28		▨							
—?— —?— —	29	?								
Well graded SAND – tan, medium dense, saturated.	30 31	SW	○	■	3 4 11	15				
Boring Terminated At 31.5'.	32 33 34 35 36 37 38 39 40									

File: MONWAT B04012A
Surface Elevation: 2453.2
Drill Rig: CME 75
Depth to Ground Water: 9.0'

Boring Number: B-4
Date Drilled: 02/17/2004
Boring Diameter: 8"
Logged By: C.M.C.

EXPLORATORY BORING LOGS

STRATA

GEOTECHNICAL ENGINEERING & MATERIALS TESTING

Sheet 2 of 2

F:\Projects\MonWat\B04012\dwg\B04012A-Boring 4 Sheet 2.dwg 5/18/2004 3:01:39 PM MDT

Final Drainage Report by CH2M HILL

TECHNICAL MEMORANDUM

CH2MHILL®

Nampa Wastewater Treatment (WWTP) Plant Phase 1 Upgrades Final Drainage Report

PREPARED FOR:	City of Nampa, Idaho
PREPARED BY:	Jodi Burns/CH2M HILL Amanda Thompson/CH2M HILL
DATE:	December 17, 2014
PROJECT NUMBER:	480770

PROFESSIONAL ENGINEER
REGISTERED
13300
12/17/2014
STATE OF IDAHO
AMANDA MARIE SMITH

Project Definition

The project is located within the City's existing Wastewater Treatment Plan, located at 340 West Railroad Street in Nampa. The existing site is located adjacent to Indian Creek, is very flat and generally slopes from south to north towards Indian Creek.

Generally, the project consists of demolishing the following facilities: secondary effluent pump station (SEPS), secondary clarifier 3 (SC3), trickling filter 1 and the asphalt to the east and the north of aeration basin 2 (AB2) as well as several segments of sidewalk. The following facilities will be built: aeration basin 3 (AB3), primary effluent pump station (PEPS), and PEPS electrical building as well as the associated yard piping and an extension of the existing asphalt access road to the south of AB3.

The foot print of the trickling filter and the asphalt access road to be demolished will be restored to gravel. In addition, in the foot print of the SEPS and SC3 will be the new AB3 (open to the atmosphere). As a result, the post development condition of the site will contain less pervious area than the predevelopment condition.

Drainage Design Calculations

Based upon the work to be completed under the Nampa WWTP Phase 1 Upgrades project, runoff volume calculations were completed for the predevelopment and post development scenarios. The City of Nampa 2012 Engineering Division Development Policy Manual (September 2012) was used to develop the calculations for the pre- and post-development scenarios.

The peak runoff volume was calculated using the Rational Method. The 50 year, 24 hour duration storm according to Exhibit A and the runoff coefficients summarized in Table 2A of Section 106 of the City of Nampa 2012 Engineering Division Development Policy Manual (September 2012) were used to complete the analysis. Table 1 and Table 2 on following page summarize the pre and post development peak runoff volumes and are as follows:

- Table 1: Nampa WWTP Pre-Development Stormwater Calculations
- Table 2: Nampa WWTP Post-Development Stormwater Calculations

In summary, the post development peak runoff volume is approximately 1,275 cubic feet less than the pre-developed peak runoff volume.

Infiltration Gallery

Because the access road will be extended to the north of PC1 ad to the south of AB3, a new gutter will be constructed along the new access road to collect stormwater. The road will have a 2% cross slope towards the gutter. There is a high point in the new road and, therefore, a portion of the new access road will drain to the west, connecting to the existing gutter and to the existing inlet, which will discharge to the WWTP. The other portion of the road will drain to the east and will discharge to an infiltration gallery. The peak runoff stormwater volume calculations for the design of the infiltration gallery are summarized in Table 3 on the following pages.

Final Site Grading Plans

The Final Site Grading Plans for the project are contained within “Volume 4 – Drawings” of the project’s construction documents.

Permits

The contractor is responsible for applying for any necessary permits and will also be responsible for creating the Stormwater Pollution Prevention Plan (SWPPP).

TABLE 1

Nampa WWTP Pre-Development Stormwater Calculations

	Drawing Number	Contributing Drainage Area, A (ft^2)	Contributing Drainage Area, A (acre)	Runoff Coefficients, C	Rainfall Depth, I (in)[a]	Volume of Runoff, V (ft^3)[b]	Volume of Runoff, V (acre-in)[b]
Impervious Area							
Demo'd Chlorine Contact Structure (below grade)	050-D-114	1000	0.0230	0.9	1.80	135.00	0.0372
Demo'd Sidewalk (not to be replaced)	050-D-114	2743	0.0630	0.9	1.80	370.31	0.1020
Demo'd Parshall Flume	050-D-109	240	0.0055	0.9	1.80	32.40	0.0089
Demo'd Secondary Sludge Pump Station	050-D-109	686	0.0157	0.95	1.80	97.76	0.0269
Demo'd Trickling Filter Sidewalk	050-D-108	4597	0.1055	0.9	1.80	620.60	0.1710
Demo'd Trickling Filter	050-D-108	31954	0.7336	0.9	1.80	4,313.79	1.1884
Demo'd Primary Effluent Splitter Box	050-D-108	247	0.0057	0.9	1.80	33.35	0.0092
Demo'd Primary Effluent Splitter Box Sidewalk	050-D-108	396	0.0091	0.9	1.80	53.46	0.0147
Demo'd Secondary Clarifier No. 1	050-D-109	7263	0.1667	0.9	1.80	980.51	0.2701
Demo'd Secondary Effluent Pump Station	050-D-109	1784	0.0410	0.95	1.80	254.22	0.0700
Demo'd Asphalt Paving to be replaced by gravel	Multiple	9734	0.2235	0.9	1.80	1,314.09	0.3620
Pervious Area							
Gravel Surfacing	Multiple	260.00	0.0060	0.4	1.80	15.60	0.0043
					Total Volume	8,221.07	2.265

[a] Values based upon 2012 Engineering Division Development Policy Manual Section 106 - Drainage and Stormwater Design Policy. Intensity from Exhibit A for 50 Year Return Frequency for a 24 hour duration (I_{24HR}=0.075 in/hr)

[b] Equation Used: $V = C \cdot I \cdot A$ (Rational Method)

TABLE 2

Nampa WWTP Post Development Stormwater Calculations

	Drawing Number	Contributing Drainage Area, A (ft^2)	Contributing Drainage Area, A (acre)	Runoff Coefficients, C	Rainfall Depth, I (in)[a]	Volume of Runoff, V (ft^3)[b]	Volume of Runoff, V (acre-in)[b]
Impervious Area							
New Concrete Sidewalk	050-C-113	460	0.0106	0.9	1.80	62.10	0.0171
New Asphalt Road	050-C-109 & 050-C-108	3000	0.0689	0.9	1.80	405.00	0.1116
New Concrete Valley Gutter	050-C-109 & 050-C-108	851	0.0195	0.9	1.80	114.89	0.0316
Primary Effluent Pump Station & Electrical Building	050-C-108	3521	0.0808	0.95	1.80	501.74	0.1382
Concrete Stoops	050-C-108	48	0.0011	0.9	1.80	6.48	0.0018
New Concrete Sidewalk	050-C-109 & 050-C-108	1617	0.0371	0.9	1.80	218.30	0.0601
New Aeration Basin	050-C-109 & 050-C-114	23500	0.5395	0.9	1.80	3,172.50	0.8740
Vaults	050-C-109 & 050-C-114	260	0.0060	0.9	1.80	35.10	0.0097
Pervious Area							
Gravel Surfacing	Multiple	40,509.00	0.9300	0.4	1.80	2,430.54	0.6696
				Total Volume		6,946.64	1.914

[a] Values based upon 2012 Engineering Division Development Policy Manual Section 106 - Drainage and Stormwater Design Policy. Intensity from Exhibit A for 50 Year Return Frequency for a 24 hour duration (I_{24HR}=0.075 in/hr)

TABLE 3

Nampa WWTP Infiltration Gallery for New Roadway Stormwater Calculations

	Runoff Coefficients, C	Rainfall Depth, I (in)[a]	Contributing Drainage Area, A (ft^2)	Volume of Runoff, V (ft^3)[b]	Length (ft)	Width (ft)	Area of Swale (ft^2)	Depth of Trench (ft) (Assumed 33% Void Space)
Infiltration Gallery	0.9	1.80	2340	315.90	22.00	22.00	484.00	1.98

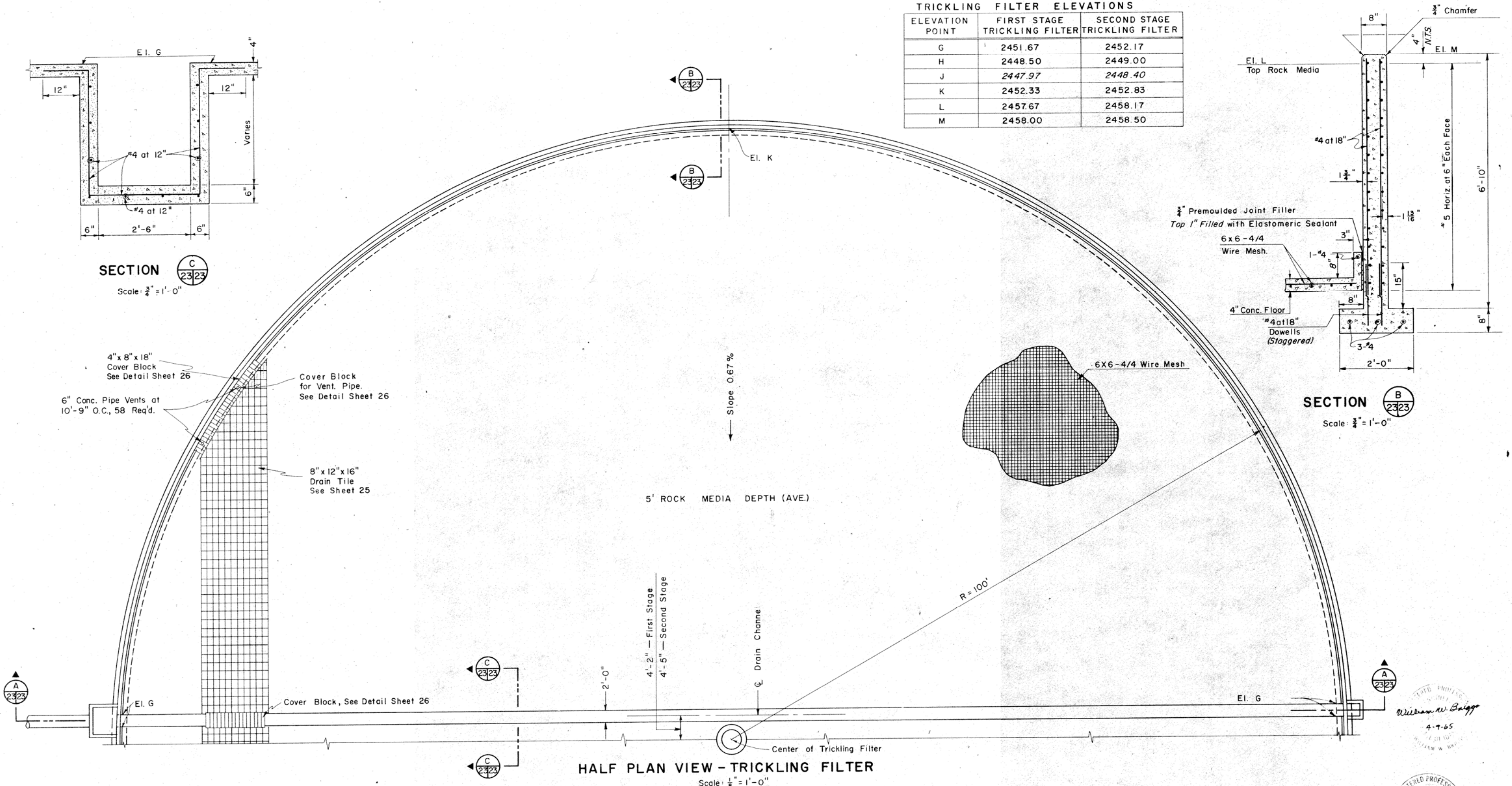
[a] Values based upon 2012 Engineering Division Development Policy Manual Section 106 - Drainage and Stormwater Design Policy. Intensity from Exhibit A for 50 Year Return Frequency for a 24 hour duration (I_{24HR}=0.075 in/hr)

[b] Equation Used: $V = C \cdot I \cdot A$ (Rational Method)

Record Drawings of Existing Facilities

TRICKLING FILTER ELEVATIONS

ELEVATION POINT	FIRST STAGE TRICKLING FILTER	SECOND STAGE TRICKLING FILTER
G	2451.67	2452.17
H	2448.50	2449.00
J	2447.97	2448.40
K	2452.33	2452.83
L	2457.67	2458.17
M	2458.00	2458.50



SECTION C 23 23

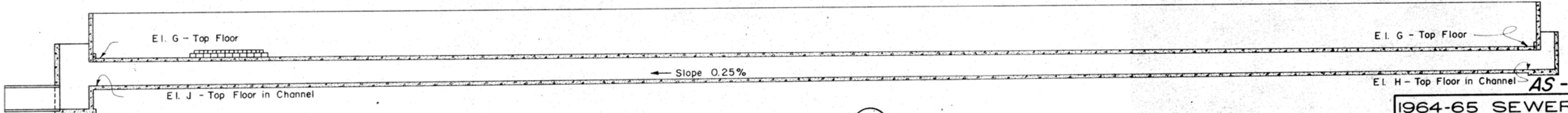
Scale: 3/4" = 1'-0"

HALF PLAN VIEW - TRICKLING FILTER

Scale: 1/8" = 1'-0"

SECTION B 23 23

Scale: 3/4" = 1'-0"



SECTION A 23 23

Scale: 1/8" = 1'-0"

NOTES:

1. Filter is symetrical about ℄ except for drain channel.
2. Trickling Filter has 4" of 1 1/4" max - 1/4" min. gravel under floor and drain channel.

William W. Briggs
4-9-65

Harold J. Cox
1347
4-9-65

AS - BUILT

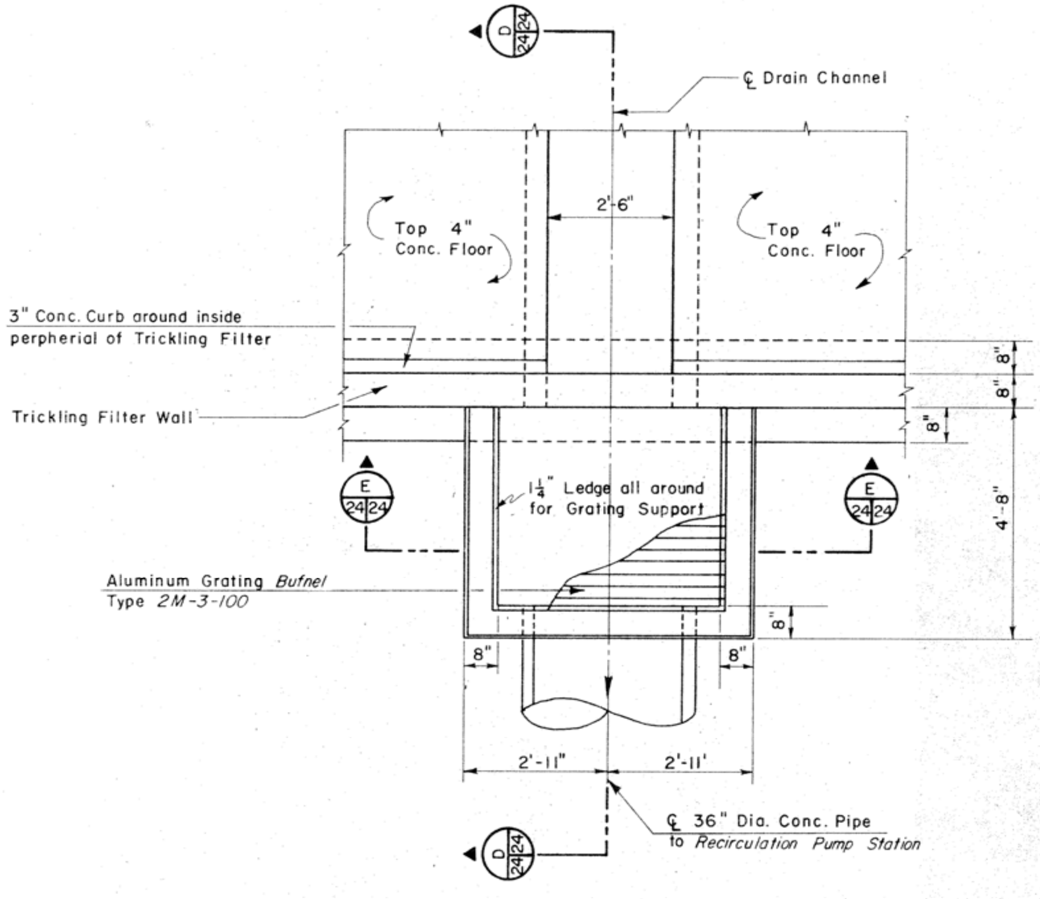
1964-65 SEWERAGE PROJECT
CITY OF NAMPA, IDAHO

FIRST & SECOND STAGE TRICKLING FILTERS
HALF FLOOR PLAN AND WALL DETAILS

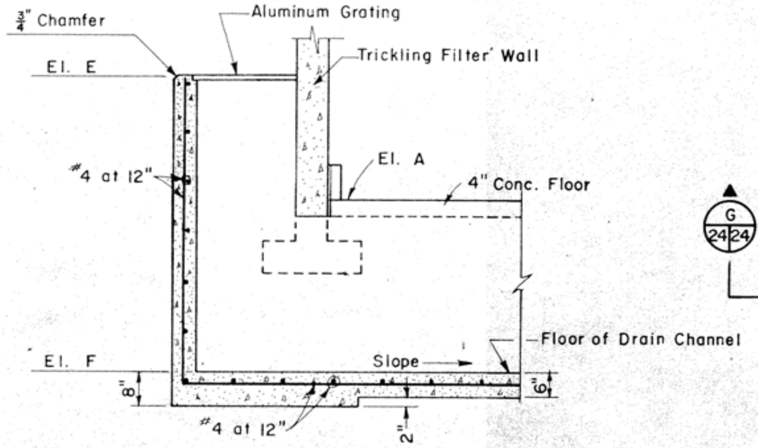
JOHNSON, UNDERKOFLER & W.W. BRIGGS
— Consulting Engineers —
Nampa, Idaho

DES. HJL	DR. L.D.V.	CK. WB	SHEET 23 OF 50
SCALE: As Shown	DATE: JAN., 1965		DWG. NO. D650111-23

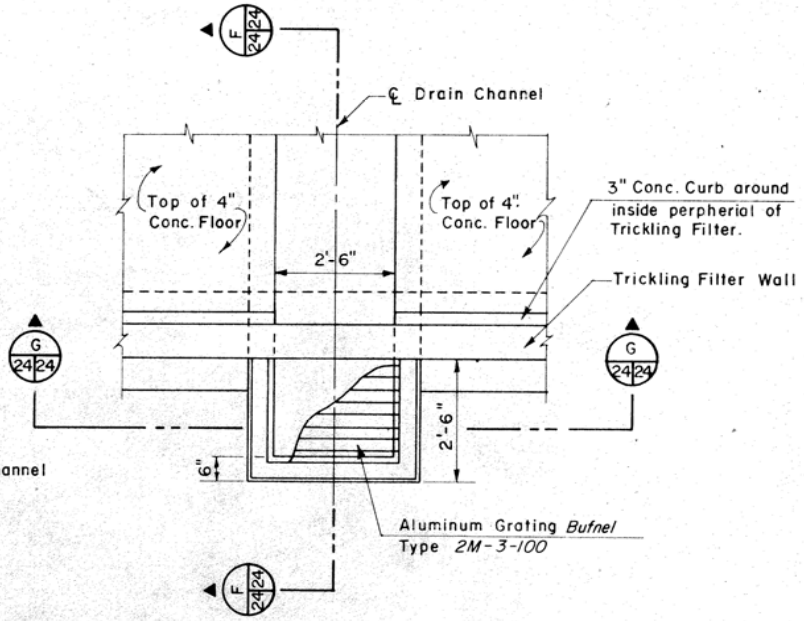
06-D0040



PLAN VIEW



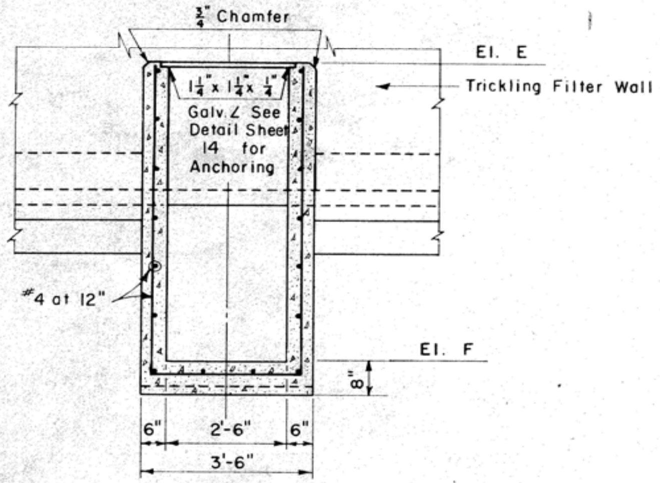
SECTION F 24|24



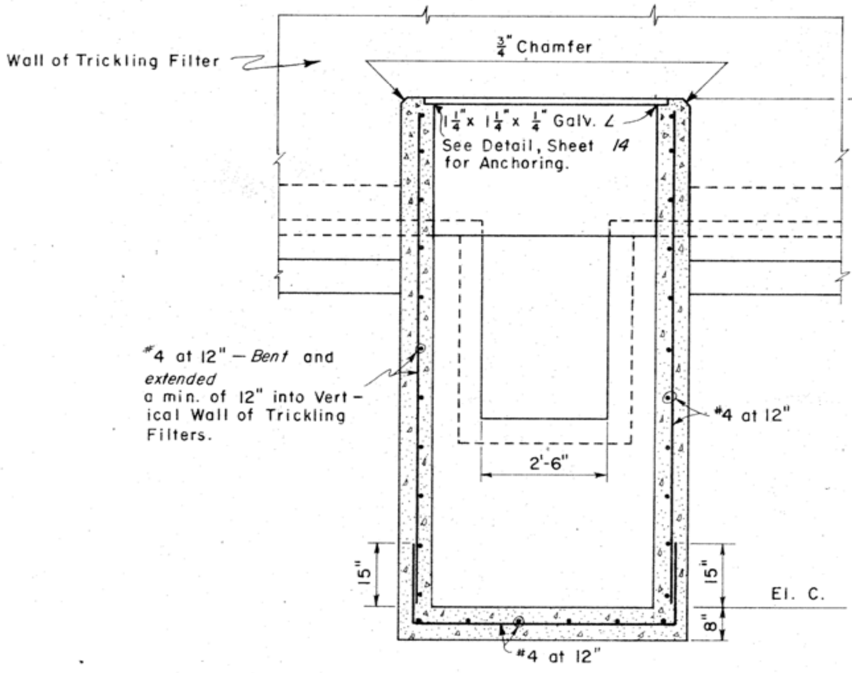
PLAN VIEW

DRAIN CHANNEL ELEVATIONS

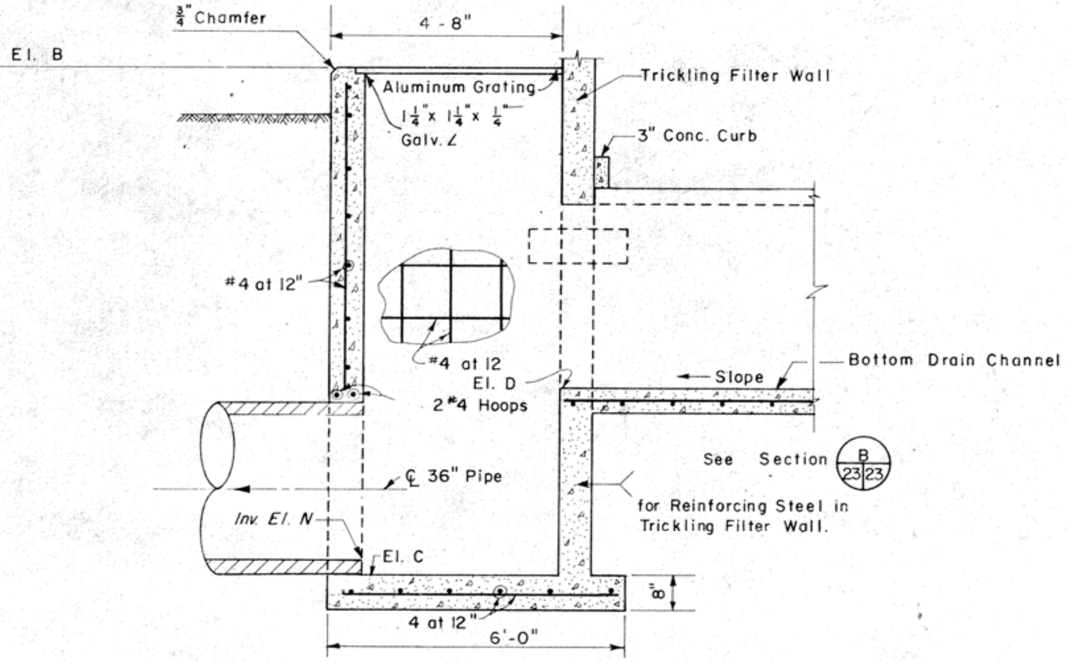
ELEVATION POINT	FIRST STAGE *TRICKLING FILTER NO.3*	*FIRST STAGE TRICKLING FILTER NO.4*	*SECOND STAGE TRICKLING FILTER*
A	2451.67	*2451.67*	*2452.17*
B	2454.50	*2454.50*	*2454.50*
C	*2444.27*	*2444.19*	*244473*
D	*2447.97*	*2447.95*	*2448.40*
E	2454.50	*2454.50*	*2454.50*
F	*2448.46*	*2448.50*	*2449.00*
N	*2444.59*	*2444.55*	*2445.13*



SECTION G 24|24



SECTION E 24|24



SECTION D 24|24

NOTES:

1. Drain Tile and Cover Blocks are not shown, see sheets 25 & 26.
2. All Construction Joints contain Water Stops.

William W. Briggs 4-4-65 WILLIAM W. BRIGGS

Harold J. Cox 1347 4-9-65 HAROLD J. COX

AS-BUILT

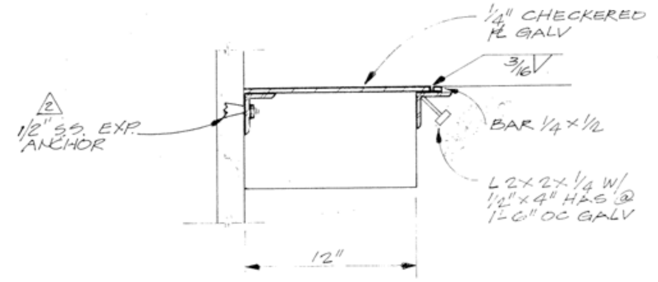
1964-65 SEWERAGE PROJECT
CITY OF NAMPA, IDAHO

TRICKLING FILTER
DRAIN CHANNEL DETAILS

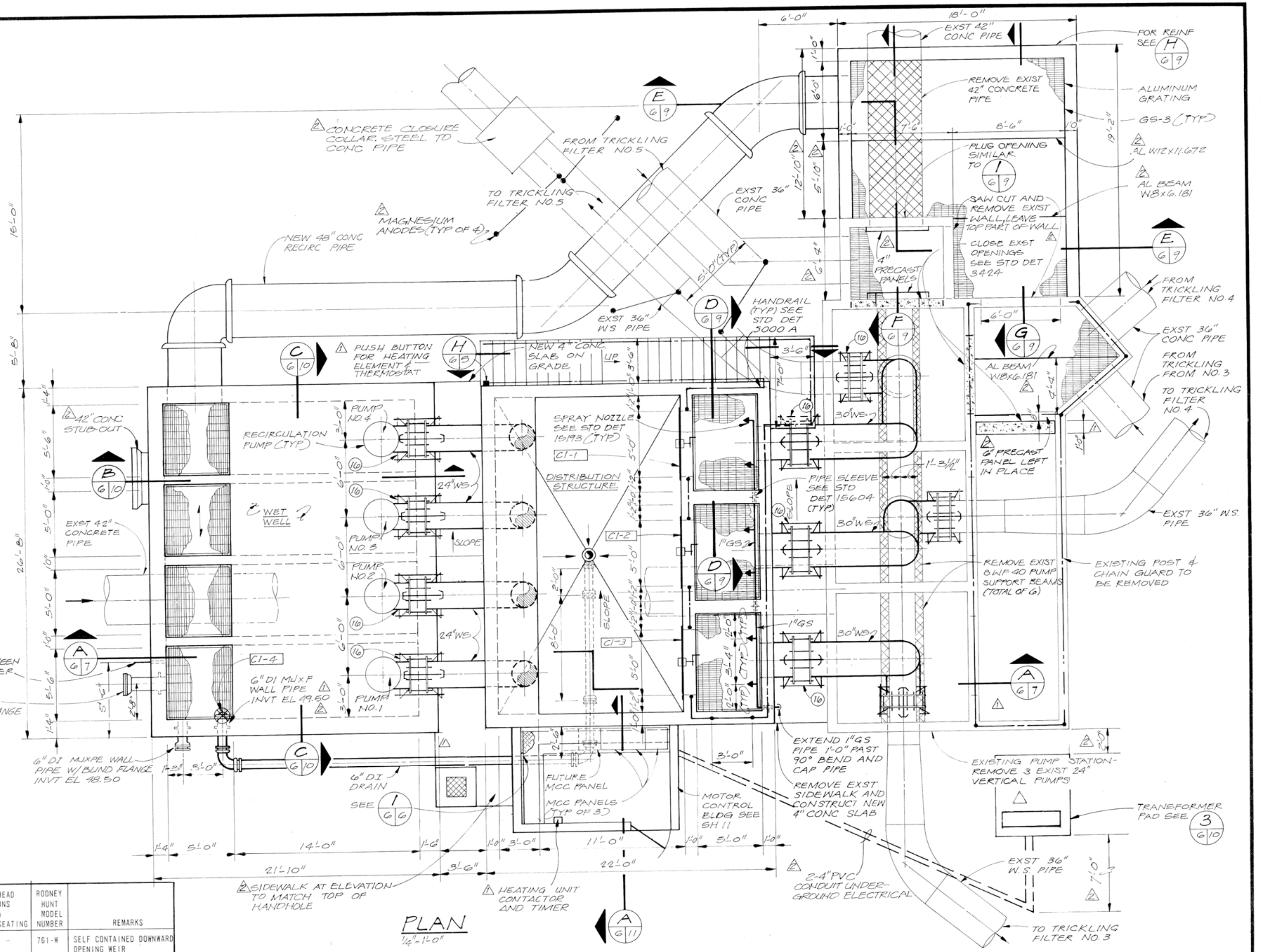
JOHNSON, UNDERKOFLER & W.W. BRIGGS
— *Consulting Engineers* —
Nampa, Idaho

DES. *W.W.*	DR. *L.D.W.*	CK. *WB*	SHEET 24 OF 50
SCALE: 1/2" = 1'-0"	DATE: JAN, 1965		DWG. NO. D650111-24

06-D0040



DETAIL 1/6 6
NTS



PLAN
1/4"=1'-0"

△
NOTE:
THE THREE D.I. WALL PIPES SHALL NOT EXTEND MORE THAN 12" FROM FACE OF WALL.
△2

STEPHEN E. AASHEIM 3141
Stephen E. Aasheim
12-5-78

NOTE: THE SYMBOL C-1 SHOWN ON THE DRAWINGS CORRESPONDS TO THE GATE NUMBERS SHOWN BELOW.

SLUICE AND SLIDE GATE SCHEDULE

GATE NUMBER	TYPE	GATE SIZE WIDTH (IN)	GATE SIZE HEIGHT (IN)	GATE MOUNTING	TYPE OF OPERATOR	OPERATOR MOUNTING	DRIVE STEM TO NUT RATIO	DESIGN HEAD CONDITIONS (FEET) SEATING	DESIGN HEAD CONDITIONS (FEET) UNSEATING	RODNEY HUNT MODEL NUMBER	REMARKS
C1-1 C1-2 C1-3	FABRICATED SLIDE GATE	60	36	FLAT BACK	HAND CRANK	BENCH STAND	2:1	2.5	–	761-W	SELF CONTAINED DOWNWARD OPENING WEIR
C1-4	SLUICE GATE	6 DIA	–	FLANGED CIRCULAR	HAND WHEEL	OFFSET TYPE, FLOOR STAND	1:1	–	18.0	160-M	

CH2M HILL

DES. WFJN
DR. LEWS
CHK. LRTR
APPD. RAFI

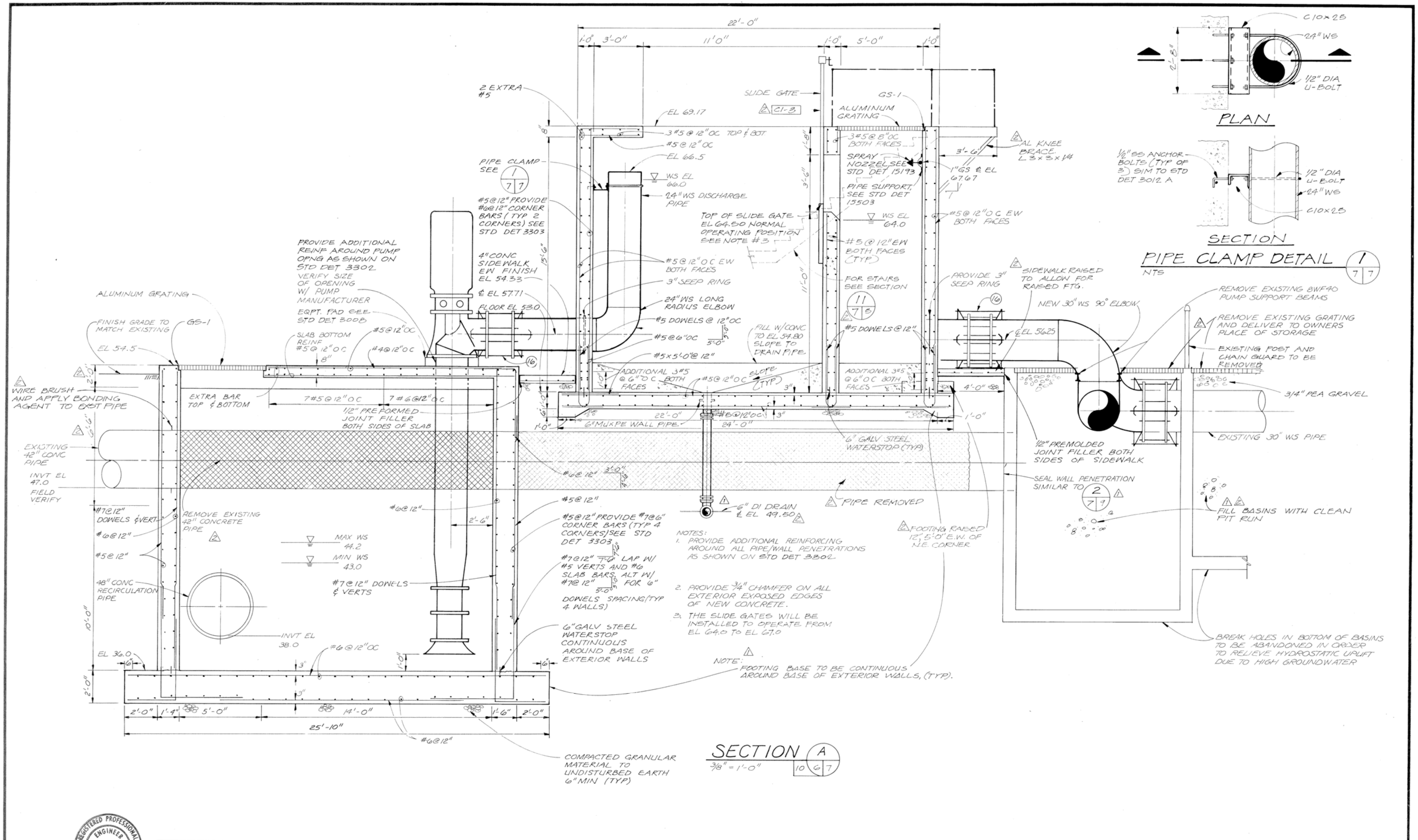
NO.	DATE	REVISION	BY	APPD.
△2	6-80	RECORD DRAWING	KIDM	LJBT
△	12/14/78	ADDENDUM NO. 1	KIDM	RDPR

WASTEWATER TREATMENT PLANT
CITY OF NAMPA, IDAHO
PHASE I

RECIRCULATION PUMP STATION — PLAN

SHEET 6
OF 15
DATE DEC 1978
DWG. NO. B9821-1

06-00051



NOTES:

1. PROVIDE ADDITIONAL REINFORCING AROUND ALL PIPE/WALL PENETRATIONS AS SHOWN ON STD DET 3302.
2. PROVIDE 3/4" CHAMFER ON ALL EXTERIOR EXPOSED EDGES OF NEW CONCRETE.
3. THE SLIDE GATES WILL BE INSTALLED TO OPERATE FROM EL 64.0 TO EL 67.0

NOTE: FOOTING BASE TO BE CONTINUOUS AROUND BASE OF EXTERIOR WALLS, (TYP).

PIPE CLAMP DETAIL 1/7 7

NTS

SECTION A/10 6 7

3/8" = 1'-0"

3141
STEPHEN E. AASHEIM
Stephen E. Aasheim

CH2M HILL	DES. WFJH
	DR. RJKE
	CHK. LRTR
	APPD. RAFI

NO.	DATE	REVISION	BY	APPD
2	6-80	RECORD DRAWING	KIDM	LJBT
1	12/14/78	ADDENDUM NO. 1	KIDM	RDPR

WASTEWATER TREATMENT PLANT
CITY OF NAMPA, IDAHO
PHASE I

RECIRCULATION PUMP STATION — SECTION

SHEET 7
OF 15
DATE DEC 1978
DWG. NO. B9821-1

06-00051