Exhibit 1 - Appendix F

FUGRO WEST, INC.

DRAFT GEOTECHNICAL STUDY
J STREET DRAINAGE IMPROVEMENTS
VENTURA COUNTY WATERSHED PROTECTION DISTRICT
PROJECT NO. 82322
OXNARD, CALIFORNIA

Prepared for:
HDR Engineering, Inc.

January 2009
Fugro Job No. 3161.014

January 20, 2009
Project No. 3161.014

HDR Engineering, Inc.
3230 El Camino Real, Suite 200
Irvine, California 92602

Attention: Mr. William Young, P.E., Water Resources Section Manager

Subject: Draft Geotechnical Study, J Street Drain Improvements, Ventura County Watershed Protection District Project No. 82322, Oxnard, California.

Dear Mr. Young:

Fugro West, Inc., herewith presents this draft geotechnical study for the proposed J Street Drain Improvements in Oxnard, California. Geotechnical engineering services for this project were provided in accordance with our revised proposal dated May 7, 2007.

The draft report presents field exploration and laboratory test data compiled for this study, and summarizes our opinions and recommendations for site preparation and channel design based on preliminary assumptions. Once design concepts are finalized, updated recommendations can be developed, as appropriate.

If you have any questions or require additional information regarding this study, please contact the undersigned.

Sincerely,

FUGRO WEST, INC.

Loree A. Berry, P.E.
Project Engineer

Jon M. Everett, P.E., G.E.
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1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical study for the planned improvements to the J Street Drain in Oxnard, California. This study was performed in general accordance with our revised proposal to HDR Engineering, Inc. (HDR) dated May 7, 2007. Our services for this project were authorized by a Subconsulting Agreement executed by Ms. Betty Dehoney of HDR, dated March 14, 2008.

The site is located as shown on Plate 1 – Site Location Map. The site layout is shown on Plate 2 – Site Exploration Plan and Profiles.

1.2 PROJECT DESCRIPTION AND SITE CONDITIONS

1.2.1 Project Description

Our understanding of the proposed project, and the general scope of geotechnical services provided for this study, is based on discussions with Mr. William Young of HDR and representatives of the Ventura County Watershed Protection District (VCWPD). The goal of the project is to reduce local flooding in the City of Oxnard by increasing the capacity of the existing J Street Drain channel. Based on preliminary design information furnished by HDR, the proposed project will consist of the construction of a rectangular reinforced concrete channel that is about 30 feet wide and 8 feet deep. New culverts designed to handle the projected 100-year flow will replace the existing culverts.

1.2.2 Site Conditions

The existing J Street Drain Channel is a 2.2-mile long concrete-lined flood control channel located along the center line of J Street in Oxnard, California. J Street consists of two northbound lanes and two southbound lanes located immediately east and west of the channel, respectively. The channel begins at Redwood Street and flows south to where it discharges into the Ormond Beach Lagoon. J Street terminates at the south end where it meets Hueneme Road. The existing drain was constructed in the 1960's and consists of a 20- to 30-foot-wide trapezoidal channel with 1(h):1(v) side slopes that are about 4 to 6 feet deep. There are seven existing culverts along the alignment that allow traffic to pass over the channel. The six culverts that handle street traffic are at Teakwood Street, Yucca Street, Bard Road, Pleasant Valley Road, Clara Street, and Hueneme Road, all of which consist of multi-barrel reinforced concrete box (RCB) structures. The seventh culvert, which allows the Ventura County Railroad to cross the channel south of Hueneme Road, consists of a parallel arrangement of five corrugated metal pipes (CMPs) ranging in diameter from approximately 4 to 5 feet. Residential development is present along both sides of the channel along most of the alignment. Commercial and industrial development is present south of the intersection of Hueneme Road.

1.3 PURPOSE AND SCOPE

The purpose of the geotechnical engineering study was to explore and evaluate the geotechnical conditions at the site, and to develop geotechnical opinions and recommendations for use in planning and design of the proposed channel improvements.

Our scope of services for the study included the following tasks.

1.3.1 Subsurface Exploration

The subsurface exploration program was performed to obtain geotechnical data for use in developing the recommendations in this report. The program consisted of electric Cone Penetrometer Tests (CPTs) and hollow-stem auger borings. The locations of the CPTs and borings are shown on Plates 1 and 2. Further details regarding the subsurface exploration program are presented in Appendix A.

Eleven (11) Cone Penetrometer Test (CPT) soundings were advanced to depths ranging from approximately 33 feet to 50 feet below the ground surface (bgs). Data from the CPT soundings are presented on the CPT logs in Appendix A (Plates A-1 through A-11).

Eight (8) hollow-stem auger borings were drilled to depths of approximately 30 to 40 feet bgs. Soil samples were obtained from the borings for laboratory testing. Three of the hollow-stem auger borings were completed as groundwater monitoring wells to depths of approximately 30 feet bgs. Descriptions of the geotechnical conditions observed in the borings are presented on the boring logs in Appendix A (Plates A-13 through A-20).

1.3.2 Laboratory Testing

Laboratory testing was performed on selected soil samples obtained from the exploratory borings. Samples were analyzed for unit dry weight and moisture content, grain size, Atterberg Limits (Plasticity Index), shear strength, and corrosion potential. The results of the laboratory tests are presented in Appendix B of this report.

1.3.3 Geotechnical Analysis and Report Preparation

The data obtained from the subsurface exploration and laboratory testing program were reviewed and evaluated to characterize the geotechnical conditions along the alignment and to develop parameters for use in design of the project. The results were compiled into this report, which includes geotechnical opinions and preliminary recommendations regarding:

- Description of the subsurface soil and groundwater conditions observed in the subsurface borings;
- Assessment of the soil engineering properties, based on field observations and laboratory testing;
- General geohazard and seismic design criteria;
- Suitability of excavated material for use as compacted fill;

- Description of applicable dewatering, temporary excavation, and shoring methods and construction considerations (does not include design of groundwater dewatering or shoring system);
- Bearing pressure, lateral earth pressure, and settlement estimates for proposed box culverts; and
- Evaluation of corrosion potential for buried ferrous metal and concrete.

2.0 GEOLOGIC AND SUBSURFACE CONDITIONS

2.1 REGIONAL SETTING

The Oxnard Plain is located within the Transverse Ranges geologic/geomorphic province of California. That province is characterized by generally east-west trending mountain ranges composed of sedimentary and volcanic rocks ranging in age from Cretaceous to Recent. Major east-trending folds, reverse faults, and left-lateral strike-slip faults reflect regional north-south compression and are characteristic of the Transverse Ranges. The project site is located in the seismically active southern California area, and the project will most likely be subjected to strong earthquake ground motion during its lifetime.

2.2 LOCAL SETTING

The Oxnard Plain is predominately underlain by alluvial soils. The earth materials exposed along the project alignment consist of fine- to coarse-grained alluvial fan deposits. Additionally, artificial fill materials associated with roadways, buildings, and other development are also present in the project area.

2.2.1 Artificial Fill (af)

Up to 4 feet of artificial fill was observed in the borings. Generally, the fill consists of asphaltic concrete and base material (where applicable) overlying medium dense clayey to silty sand. The majority of the non-pavement artificial fill materials were likely derived from the underlying alluvial materials. Because of the similarity in material types, it was often difficult to differentiate the fill from the underlying alluvium. Therefore, the differentiation shown on the boring logs may vary from actual conditions encountered during construction.

2.2.2 Alluvium (Qal)

Native soils observed in the borings and encountered in the CPTs at the ground surface or below the artificial fill consisted of predominately coarse-grained alluvial deposits with interbedded fine-grained deposits of variable thickness and consistency. The coarse-grained deposits consisted of loose to medium dense sands, silty sands and clayey sands. The fine-grained material consisted of soft to stiff silts and clays.

2.3 GROUNDWATER

Groundwater was observed in all of the borings at depths ranging from about 4½ feet to 11 feet bgs. Groundwater levels published by the California Geologic Society (CGS, 2002) indicate historic groundwater levels are within 5 feet below the ground surface. Based on the subsurface soil and groundwater conditions observed in the borings, it is possible that groundwater may reach the existing ground surface during storm events. It must be noted that groundwater conditions can vary seasonally and/or in response to changes in rainfall and other factors not evident at the time of our subsurface exploration, such as irrigation, land use, and groundwater withdrawal.

2.4 POTENTIAL VARIATION OF SUBSURFACE MATERIALS

The borings and CPT soundings performed for this study were spaced about 400 to 1,000 feet apart along the proposed channel alignment (Plates 1 and 2). Therefore, there is a potential for variation in the consistency, density, and strength/hardness of the materials. There is also potential for oversized materials (greater than 8 inches in diameter), perched water, zones of poorly consolidated soils, or other conditions not indicated in the boring logs and CPT logs. If significant variation in the geologic conditions is observed during grading, we recommend that the geotechnical engineer, in conjunction with the project designer, evaluate the impact of those variations on the project design.

2.5 SEISMIC CONSIDERATIONS AND GEOHAZARDS

2.5.1 Potential for Strong Ground Shaking

The site is located in the seismically active southern California region and ground shaking generated from future earthquakes on local or regional faults should be anticipated.

Based on a regional probabilistic seismic hazard evaluation using averaged results from the ground motion attenuation relations, the California Division of Mines and Geology (CDMG, 2002) estimates peak horizontal ground acceleration (PGA) ranging from 0.59g to 0.62g for a 10 percent probability of exceedance in a 50-year exposure period. CDMG (2002) also indicates that the predominant earthquake moment magnitude is about M7.3 and the modal distance is about 2 kilometers (km) for the project area.

2.5.2 Ground Rupture Potential

No known active or potentially active faults have been mapped by other investigators beneath or trending toward the site. In addition, the site is not located within an Alquist-Priolo Special Studies Zone. Therefore, in our opinion, the ground rupture potential due to faulting is considered to be low.

2.5.3 Liquefaction Potential

Soil liquefaction occurs as a result of a loss of shear strength or shearing resistance in loose, saturated soils subjected to earthquake-induced ground shaking. Typically, soil liquefaction occurs within the upper 50 feet of the soil profile and can be manifested at the ground surface by the formation of sand boils, ground surface settlement, lateral spreading, and/or ground oscillation.

Like most of Oxnard and Port Hueneme, the J Street Drain is located within a liquefaction hazard zone as mapped by CDMG (2002). Granular subsurface soils and high groundwater suggest liquefaction settlement could occur along the alignment.

The magnitude of liquefaction-induced settlement along the channel alignment was estimated using the CPT-Analyst software program at each of the 11 CPT soundings performed for this project. A design groundwater level of 5 feet below existing ground surface along J Street was used at all locations. The design earthquake input parameter was the site PGA, which is described in Section 2.5.1. The range of estimated liquefaction settlements at each CPT location is presented in the following table.

Table 1 - Estimated Liquefaction-Induced Settlement Along Channel Alignment

CPT Number	Station	Estimated Settlement (in.)
CPT – 1	126+20	2 – 3
CPT – 2	109+00	3 – 4
CPT – 3	102+15	3 – 4
CPT – 4	83+35	2 – 3
CPT – 5	74+35	5 – 6
CPT – 6	60+40	4 – 5
CPT – 7	52+60	4 – 5
CPT – 8	41+20	4 – 5
CPT – 9	35+30	3 – 4
CPT – 10	29+20	5 – 6
CPT - 11	16+40	7 – 8

The results of the liquefaction analyses indicate that some segments of the alignment could experience more seismic settlement than others during the design-level earthquake.

3.0 RECOMMENDATIONS

Geotechnical recommendations for concrete channel design are presented below.

3.1 SITE PREPARATION

Prior to channel construction, the existing concrete channel, unsuitable fill materials, or any other deleterious materials should be demolished or stripped and removed from construction areas. Underground structures (e.g., pipelines, old foundations, etc.) and soils disturbed during the demolition process also should be removed.

3.2 EXCAVATIONS

3.2.1 Excavation Conditions

The earth materials encountered in the borings excavated for this study consisted primarily of granular soils deposited in an alluvial environment. The fines content of the sampled granular materials ranged from about 1 to 37 percent. It should be noted that granular material with low fines content like those encountered in our explorations are particularly susceptible to caving. Appropriate shoring or laying back of trench walls should be utilized to reduce the potential for caving.

Based on our observations during drilling, we anticipate that conventional heavy grading equipment in good working order should be capable of excavating the earth materials encountered along the alignment of the channel improvements. However, smaller equipment may be necessary where working space is limited.

Groundwater was observed at the exploration locations at depths ranging from about 4½ feet to 11 feet below the existing ground surface. Therefore, dewatering will likely be required at most locations along the alignment since the observed groundwater levels were at or above the bottom of proposed construction excavations. Where shallower excavations do not extend below the groundwater level, the excavation bottom will likely be locally wet, soft, and yielding. For this condition, the bottom of the excavation should be stabilized prior to construction of channel improvements so that the subgrade is firm and unyielding.

3.2.2 Special Subgrade Stabilization Measures

As indicated above, stabilization of channel excavation bottom conditions may be needed if the subgrade is soft or yielding. The contractor, after considering input from the design engineer, geotechnical engineer, and owner, should be responsible for design and implementation of any subgrade stabilization techniques. Some methods that have been used successfully to stabilize subgrade include:

- Rock stabilization blanket - Geotextile fabric (such as Mirafi HP570) can be placed along the excavation bottom and covered with a 1- to 2-foot thick layer of 4-inch minus crushed rock. A layer of ¾-inch crushed rock sufficient to fill the voids is then spread on top of the coarser material and can be covered with a non-woven filter fabric (such as Mirafi 180N) if fill soil will be placed on the stabilization blanket; or
- Soil-cement - The soft subgrade can be overexcavated, mixed with portland cement, and replaced to form a layer of cement-stabilized soil.

3.2.3 Dewatering

For excavations extending below anticipated groundwater elevations, pumping of free water from open excavations using portable sump pumps may not be adequate to maintain excavations in a dry and stable condition. Instead, an integrated system of fixed dewatering wells may be required. Dewatering systems should be designed, installed, and operated by an experienced contractor specializing in groundwater dewatering systems and should be capable of lowering the groundwater surface to a level below the required depth of excavation. Groundwater levels should be maintained at least 3 feet below any point on the excavated surface (defined by the elevation of any overexcavated surface) and should provide excavation sidewalls free of groundwater seepage. The dewatering system should be designed, installed and operated so as to minimize the potential for settlement and damage to adjacent improvements and property.

Before selecting or implementing a dewatering system, we recommend that a dewatering test program be conducted to evaluate the feasibility and efficiency of the proposed dewatering system. Dewatering operations will require permitting in accordance with National Pollutant Discharge Elimination System (NPDES) regulations and possibly other local permits. It is recommended that groundwater along the channel alignment be tested for the presence of environmental contaminants in order to evaluate the need for treatment prior to discharge or disposal.

To aid in the dewatering design, pump testing was performed in two of the monitoring wells, MW-1 and MW-2. The results of the pump testing were used to evaluate the hydraulic conductivity of the aquifer at two locations along the channel alignment. Details regarding the pump test procedures and results of the evaluation are described in Appendix C - Hydraulic Conductivity Testing.

3.2.4 Temporary Excavations and Shoring

Excavations more than 4 feet deep should be sloped, shored, or shielded in accordance with federal and state standards, project specifications, and safe construction practices. The contractor is responsible for providing and maintaining safe excavations according to Occupational Safety and Health Administration (OSHA) regulations.

In areas where the right-of-way is of sufficient width, temporary excavations could potentially be laid back no steeper than 1:1. However, loose to medium dense sands with varying amounts of silt, clay, and gravel were encountered in the borings. Per OSHA (1926), unsupported excavations for Type C soils (sands and gravels) should be sloped no steeper than 1.5(h):1(v), and even flatter slopes may be warranted depending on exposed soil conditions. Temporary excavations should be monitored for stability during construction and be modified if necessary. Excavations lacking adequate sidewall support could move or be unstable and result in damage to existing improvements and utilities adjacent to the channel alignment. The use of unshored excavations will likely limit traffic access near the top of temporary slopes.

Where there is insufficient width or where other factors would prohibit the use of temporary construction slopes, a shoring system will likely be required. The selection, design, and installation of any shoring system needed for the project should be made by the contractor in accordance with OSHA regulations.

We anticipate that potential shoring methods could consist of cantilevered sheet piling or cantilevered soldier beam and lagging systems. Lateral pressures applicable for the design will depend on the type of shoring system selected by the contractor, surcharge loads due to construction equipment and traffic, and any dewatering methods that are used.

3.2.5 Operations

To help reduce the potential for caving/sloughing of the excavation sidewalls from construction equipment and/or traffic vibration, we suggest that the contractor maintain a setback equal to the depth of the excavation. However, if local soil conditions create a sidewall-stability hazard, the project geotechnical engineer should be consulted to evaluate alternative minimum distances needed between the edge of the excavation and construction equipment, vehicle traffic, and stockpiled materials, so that the potential for sidewall instabilities can be minimized.

As a general guideline, heavy equipment should be excluded from a zone located between the top of the excavation and a 1h:1v projection from the bottom of the adjacent sidewall. This is a general guideline and may need to be modified in the field for specific geotechnical conditions. The contractor should consult the project geotechnical engineer regarding excavation procedures.

3.3 FILL MATERIALS

Based on limited laboratory testing performed as part of this study, much of the onsite soil appears to satisfy requirements for general fill as described below. General fill may be used for fill beneath the channel bottom, beneath the channel wall footings, and behind the channel walls (outside of the drainage envelope as described in Section 4.6.4).

3.3.1 General Fill

Soil generated during removal of the existing channel may be suitable for use as general fill provided that oversize materials are removed and debris and other deleterious materials are excluded.

General fill materials should meet the following requirements:

- No rocks larger than 3 inches in maximum dimension.
- No more than 15 percent material larger than 2 inches.
- Non-expansive (EI $\leq$ 20).
- Plasticity Index $<$ 10.
- Less than 30 percent passing the No. 200 sieve.

3.3.2 Imported Fill

Imported fill materials may be used for general fill, provided that the imported fill satisfies the requirements in Section 3.3.1. Imported fill material should be evaluated by the geotechnical engineer to verify suitability for its intended use.

3.3.3 Drainage Materials

Drainage material should be placed behind the channel walls in accordance with Section 4.6.4, and consist of clean, coarse-grained material with no more than 5 percent passing the No. 200 sieve. Acceptable drainage materials include "Pervious Backfill" conforming to Item

300-3.5.2, Standard Specifications for Public Works Construction (Greenbook, 2006), "Permeable Material" conforming to Item 68-1.025, Caltrans Standard Specifications (Caltrans, 2006), or three-quarter-inch uniformly graded rock or gravel. All drainage materials should be enclosed in a filter fabric, such as Mirafi 140N or equivalent.

3.4 GENERAL FILL PLACEMENT

Fill should be placed and compacted at a moisture content within 2 percent of the optimum moisture and compacted to at least 90 percent relative compaction as determined by ASTM D1557. Fill should be spread in lifts no thicker than about 8 inches prior to being compacted. Each layer should be spread evenly and thoroughly blade-mixed during the spreading to provide relative uniformity of material within each layer. Soft or yielding materials should be removed and replaced with properly compacted fill material prior to placing the next layer.

3.5 OVEREXCAVATION AND BACKFILL

All soils disturbed as part of channel demolition and any existing artificial fill soils exposed during demolition should be overexcavated to expose undisturbed native material. The overexcavation should extend at least 2 feet beyond the outside edge of channel wall footings. Any soft, loose, or unstable soil or other deleterious material should be removed entirely and replaced with engineered compacted fill. Backfilling of excavations should be performed in accordance with Section 3.4. Backfill materials below the channel bottom or wall footings should consist of stabilization materials as described in Section 3.2.2 and/or general fill materials that meet the minimum requirements in Section 3.3.1 of this report. All overexcavation, removal, and backfill activities should be performed under the observation and testing of Fugro.

4.0 CHANNEL FOUNDATION DESIGN

Channel retaining walls and culverts should be designed and constructed in accordance with recommendations below.

4.1 ALLOWABLE BEARING PRESSURE

Retaining wall footings may be sized for dead load plus probable maximum live load using a maximum net allowable bearing pressure of 1,500 pounds per square foot (psf) for footings founded on compacted fill as described above. The recommended allowable bearing pressure includes a factor of safety for general shear failure in excess of 2.5. A one-third increase in the allowable bearing pressure may be used for transient loads such as seismic or wind forces.

Fugro estimates that most of the RCB structures will exert a contact pressure of 500 psf or less on underlying soils. As they become available, the dead weights, live loads, and structure dimensions for all culverts should be provided to Fugro in order to verify the design bearing pressures.

4.2 MODULUS OF SUBGRADE REACTION

The design of slabs may be based on an analogy with a beam on an elastic half-space. A modulus of subgrade reaction (k) of 175 pounds per cubic inch can be used for the design of the channel bottom founded on granular compacted fill.

4.3 SETTLEMENT

Provided the channel wall footings are designed and constructed in accordance with recommendations herein, we anticipate that total settlement from static loads generally should be on the order of about 1 inch or less if bearing on a minimum of 2 feet of compacted fill.

For the assumed maximum static bearing pressure of 500 psf, static settlements of reinforced concrete box (RCB) culverts are estimated to be on the order of 2 – 3 inches, most of which will occur during or soon after construction. These structures will also be subjected to the estimated liquefaction settlements presented in the table above.

The future loading requirements for the new culvert at the railroad crossing are unknown. Fugro assumes that the new structure will consist of a multi-barrel RCB similar to the other proposed structures. Settlements for the proposed railroad crossing structure are expected to be on the order of the estimated settlements discussed above for RCBs at the other locations. Fugro should confirm the estimated settlement for the railroad crossing structure once the structure type and loading conditions are known.

4.4 RESISTANCE TO LATERAL LOADS

Ultimate sliding resistance generated through a concrete-on-soil interface may be computed by multiplying dead weight structural loads, less buoyant forces where applicable, by a of coefficient 0.4.

Ultimate passive earth resistance may be estimated using an equivalent fluid weight of 350 pcf for drained conditions and 175 pcf for undrained conditions, based on a friction angle of 35 degrees and an average total unit weight of about 120 pcf. The undrained passive resistance is provided to allow consideration of drainage facilities constructed behind the channel walls that become clogged.

Sliding resistance and passive pressure may be used together without reduction in conjunction with minimum factors of safety of 1.5 and 2.0, respectively.

4.5 UPLIFT PRESSURES

Groundwater levels may rise above the channel bottom due to a rise in the groundwater table or flood conditions in the channel. Therefore hydrostatic uplift pressures should be considered in design.

For uplift design, dead weight loads should exceed uplift pressures along the foundation bottom. Dead weight loads may be estimated using the total unit weight of the concrete channel and the total unit weight of soil above any footing extensions beyond the channel walls. The weight of water in the channel should be considered a live load and therefore omitted from the cumulative dead weight loads. Uplift pressures should be estimated assuming the groundwater level is at the ground surface.

4.6 LATERAL EARTH PRESSURES

4.6.1 General

Walls that are free to rotate or translate laterally (e.g., cantilevered) through a horizontal-distance-to-wall-height ratio of no less than 0.004 are referred to as unrestrained or yielding. Such walls can generally move enough to develop active conditions. Walls that are unable to rotate or deflect laterally (e.g. fixed at the top) are referred to as restrained or non-yielding.

If backfill materials behind the channel walls consist of cohesionless soils, then unrestrained walls can usually be designed for active earth pressure conditions, which are lower than at-rest conditions. For cohesionless backfills, restrained walls should be designed for at-rest earth pressure conditions. For cohesive backfills, both unrestrained and restrained walls should be designed for at-rest conditions because cohesive soils creep, undergo stress relaxation, and cannot sustain active conditions. If backfill materials are expansive, then lateral earth pressures will be increased as a result of swelling pressures.

Based on our understanding of the project, the channel walls will be unrestrained. If wall backfill consists of material conforming to general fill requirements presented in Section 3.3.1, unrestrained channel walls should be designed for active conditions.

4.6.2 Equivalent Fluid Weights

Table 2 presents recommended equivalent fluid weights for level backfills for static conditions for the at-rest and active cases under either drained or undrained conditions. Drained conditions imply that drainage measures are incorporated into the wall to preclude the buildup of hydrostatic pressure as described in Section 4.6.3.

**Table 2. Recommended Equivalent Fluid Weights (pcf)
for Retaining Wall Design**

Wall Backfill	Drained		Undrained	
	Active	At-Rest	Active	At-Rest
Compacted Cohesionless Soil	30	60	80	90

4.6.3 Channel Wall Construction

Fill Placement. Backfill materials behind channel walls and beneath channel bottoms can consist of onsite soils satisfying recommendations in Section 3.3.1, and should be compacted to at least 90 percent relative compaction. Channel wall backfill should be placed outside of the drainage material described herein, to at least a 1(h):1(v) line projected upward from the heel of the channel wall footing. Wall backfill geometry may need to be modified to satisfy OSHA regulations for temporary excavations summarized in Section 3.2.2.

Compaction Adjacent to Walls. Backfill within 5 feet, measured horizontally, behind the retaining structures should be compacted with lightweight hand-operated compaction equipment to reduce the potential for induction of large compaction-induced stresses. If large or heavy compaction equipment is used, compaction-induced stresses can result in increased lateral earth pressures on the retaining walls. If lightweight, hand-operated compaction equipment will not be used, further evaluation of the potential for compaction-induced stresses may be warranted.

Drainage Measures. Drainage measures, as described in Section 3.3.3, should be provided behind channel walls to preclude the buildup of hydrostatic pressures. As previously specified, clean, coarse-grained material with no more than 5 percent passing the No. 200 sieve should be enclosed in or protected with a filter fabric, such as Mirafi 140N or equivalent.

4.6.4 Corrosion Potential

Three corrosion tests were performed on representative samples of subsurface materials. The soils tested were predominantly coarse-grained materials with varying amounts of fines. The results are summarized in Table 3. The resistivity values indicate that the tested samples are mildly to moderately corrosive to ferrous metals. Measured sulfate and chloride values were generally low for all of the samples tested and suggest those soils are generally not corrosive to concrete and steel reinforcing. The test results should be evaluated by a corrosion specialist to confirm the opinions regarding the potential corrosion impacts from the onsite soils to the channel and other construction materials proposed for the project. We also recommend that any imported fill used as backfill against concrete structures be tested to evaluate corrosion potential.

Table 3. Summary of Chemical Test Results

Boring No.	Material Description	Resistivity (ohms/cm)	pH	Chloride (ppm)	Sulfate (percent)
DH-1	Sand (SP)	7,893	9.0	<2	<0.0005
DH-3	Clayey sand (SC)	1,100	8.1	10	0.0480
DH-5	Silty sand (SM)	1,188	8.2	56	0.0991

5.0 LIMITATIONS

This geotechnical study report has been prepared for HDR solely for the planning, design, and construction of the proposed J Street Drain Improvements.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere. Any statements, or absence of statements, in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessments.

In performing our professional services, we have used generally accepted geologic and geotechnical engineering principles and have applied that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers currently practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report.

Results, evaluations, conclusions, and recommendations contained in this report are directed at, and intended to be utilized within, the scope of work contained in the proposal executed by Fugro and the client. This report is not intended to be used for any other purposes. Fugro makes no claim or representation concerning any activity or condition falling outside its specified purposes to which this report is directed, said purposes being specifically limited to the scope of work as defined in said agreement. Inquiries as to said scope of work or concerning any activity not specifically contained therein should be directed to Fugro for determination and, if necessary, further investigation.

We recommend that Fugro West, Inc., be retained to review and comment on geotechnical aspects of the project plans and specifications before they are finalized. This can allow Fugro West, Inc., to evaluate if the recommendations in this report have been properly interpreted and implemented in the design, specifications, and drawings.

Users of this report should recognize that the construction process is an integral design component with respect to the geotechnical aspects of the project. Because geotechnical engineering is inexact due to the variability of the natural processes, unanticipated or changed conditions can occur. Proper geotechnical observation and testing during construction is thus imperative in allowing the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, we recommend that Fugro West, Inc., be retained during site grading, excavation, and construction of foundations to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those anticipated.

6.0 REFERENCES

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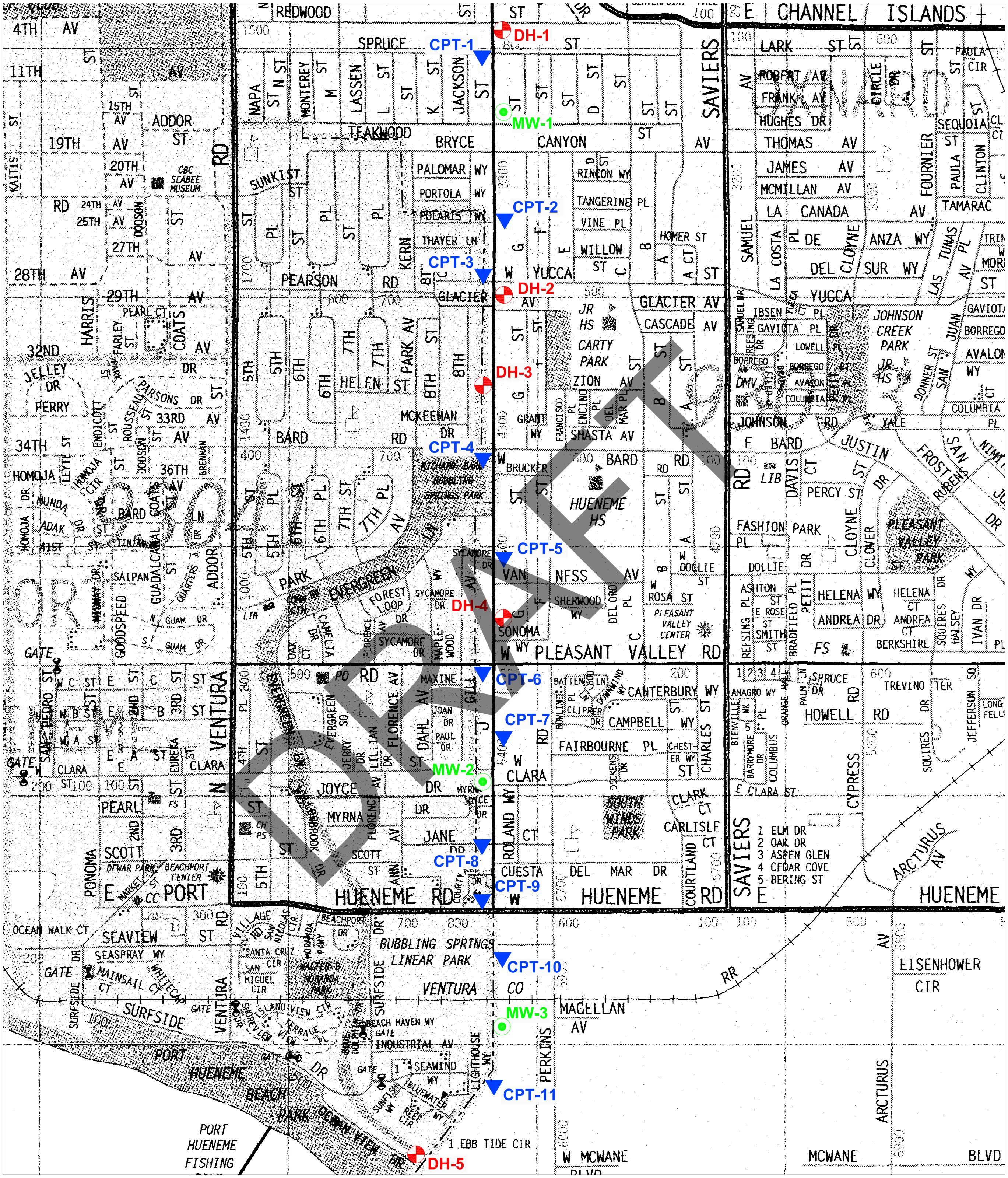
PLATES

HDR Engineering
Project No. 3161.014



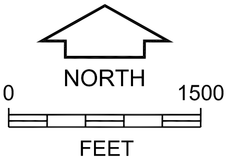
BASE MAP SOURCE: Thomas Guide 2002, Ventura County, p. 552

LEGEND

⊕ DH-5	Approximate drill hole location
◉ MW-3	Approxiamte piezometer location
▼ CPT-11	Approximate CPT location

SITE LOCATION MAP

J Street Drain Improvements
Oxnard, California



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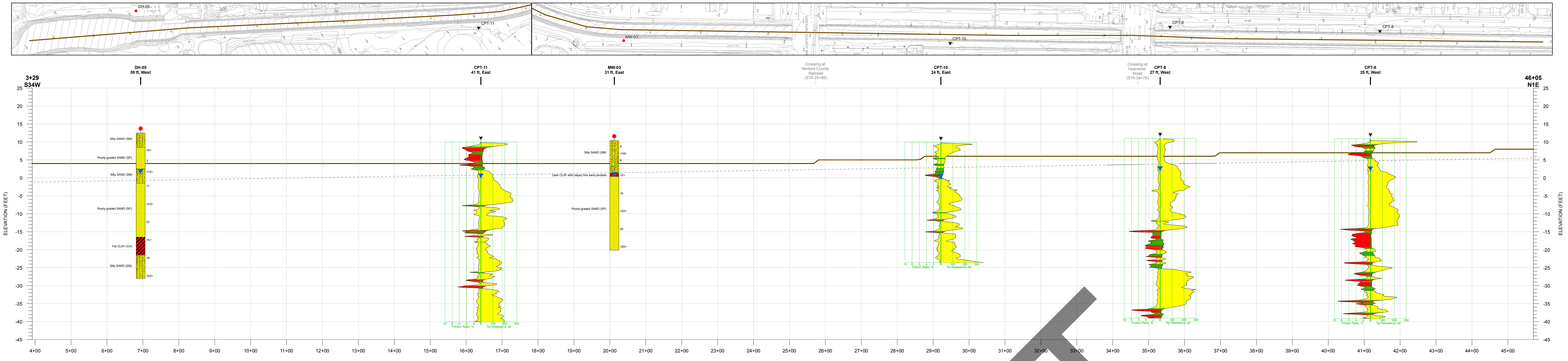
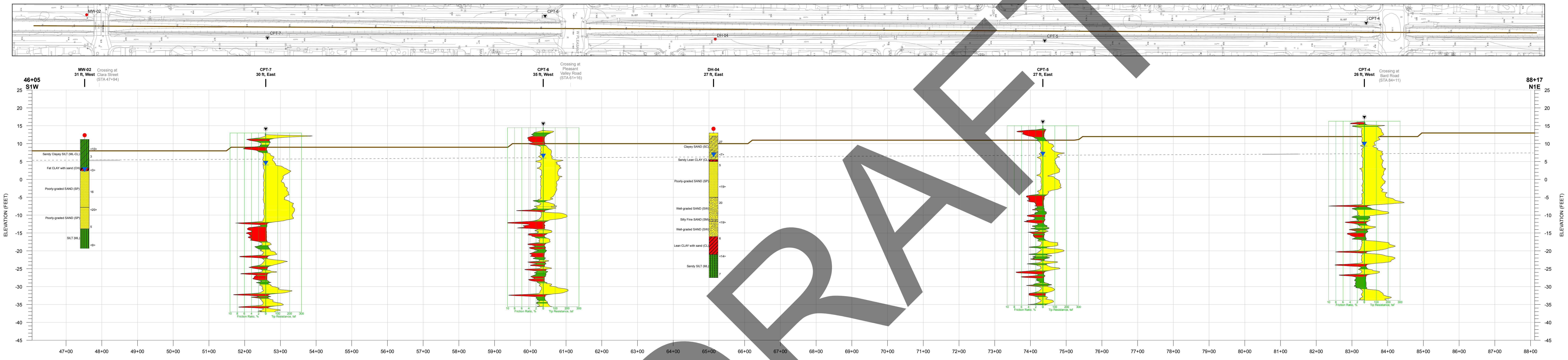
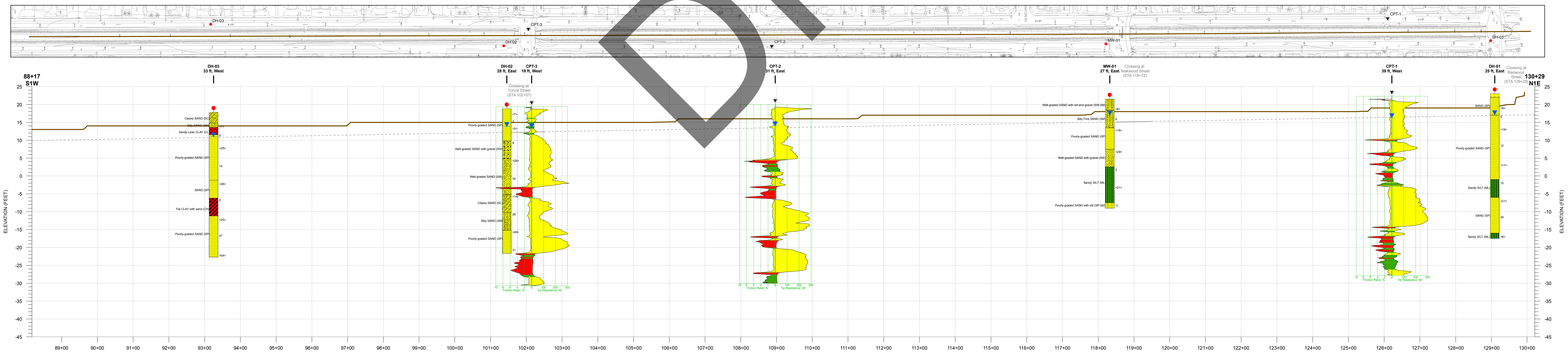
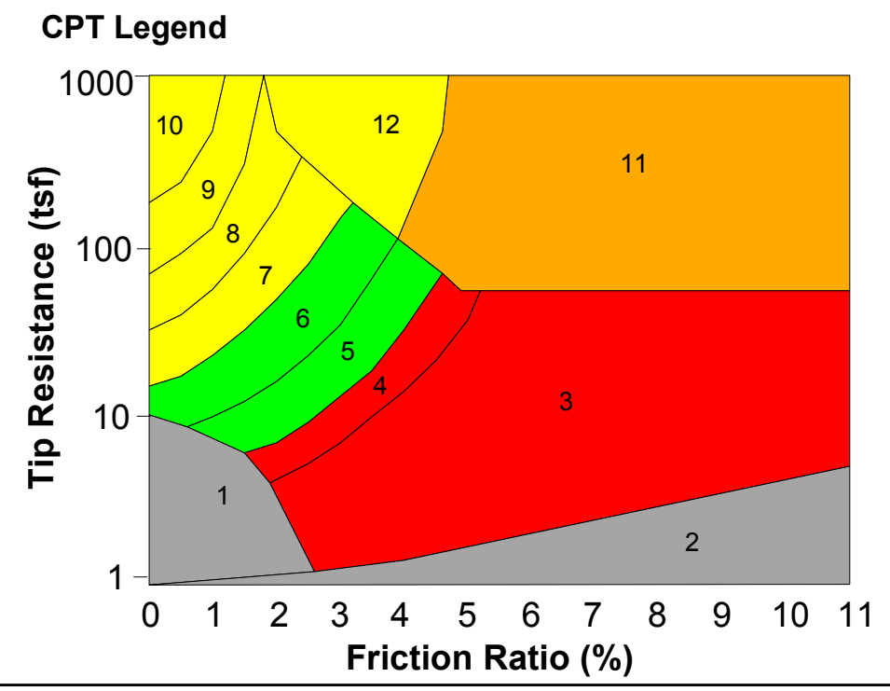






CPT Legend



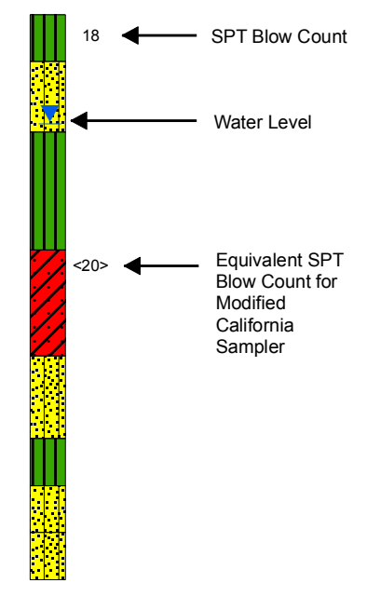
Zone	Soil Behavior Type	U.S.C.S.
1	Sensitive Fine-grained	OL-CH
2	Organic Material	OL-OH
3	Clay	CH
4	Silty Clay to Clay	CL-CH
5	Clayey Silt to Silty Clay	MH-CL
6	Sandy Silt to Clayey Silt	ML-MH
7	Silty Sand to Sandy Silt	SM-ML
8	Sand to Silty Sand	SM-SP
9	Sand	SW-SP
10	Gravelly Sand to Sand	SW-GW
11	Very Stiff Fine-grained *	CH-CL
12	Sand to Clayey Sand *	SC-SM

*overconsolidated or cemented

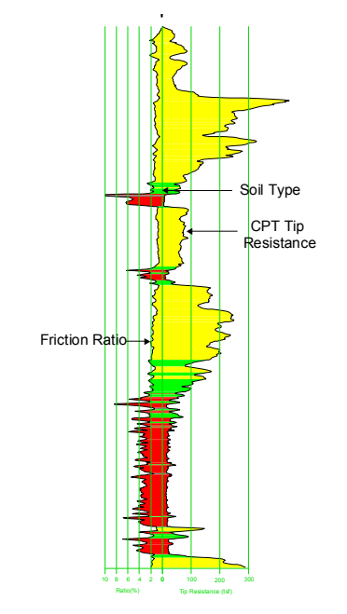
CPT CORRELATION CHART
(Robertson and Campanella, 1984)

Soils Legend

- ▨ Poorly-graded SAND (SP)
- ▨ Well -graded SAND (SW)
- ▨ Silty SAND (SM)
- ▨ Poorly-graded SAND with silt (SPSM)
- ▨ Well-graded SAND with silt (SW-SM)
- ▨ Clayey SAND (SC)
- ▨ SILT (ML)
- ▨ Clayey SILT (ML-CL)
- ▨ Fat CLAY (CH)
- ▨ Lean CLAY (CL)



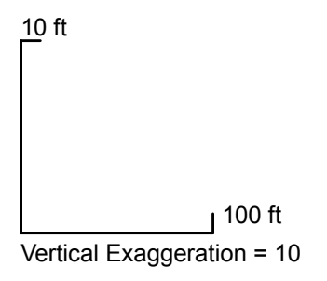
SOIL BORING LITHOLOGY



CPT SOUNDING WITH INTERPRETED LITHOLOGY

- - - Approximate elevation of proposed flow line, dashed where interpolated between known points provided by HDR, Inc.

— Approximate elevation of center of existing flood control channel

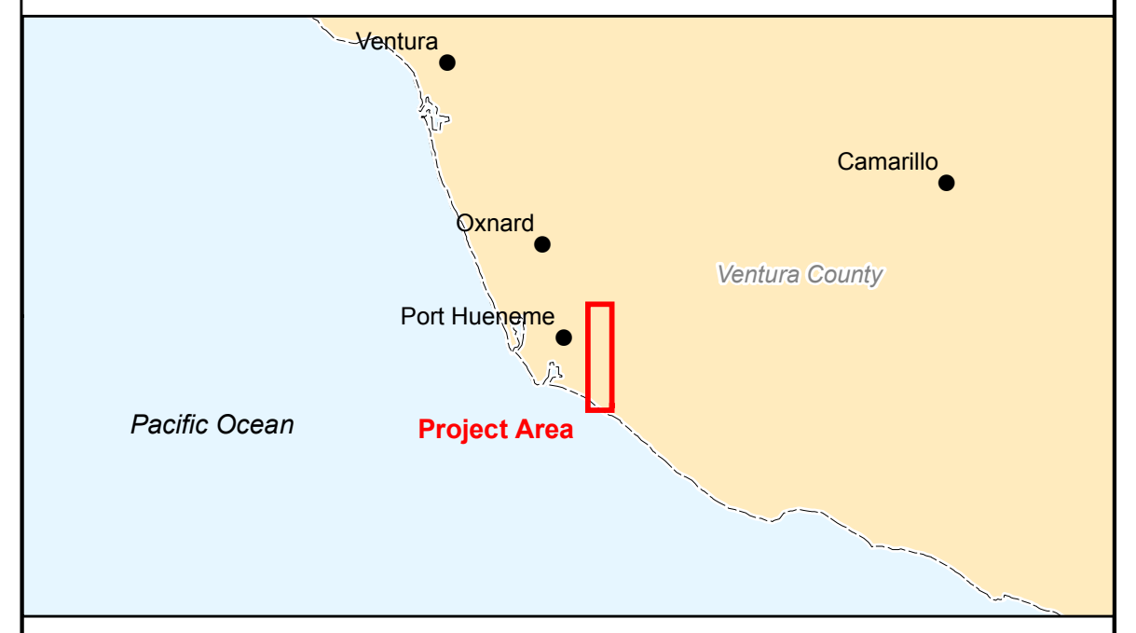


Vertical Exaggeration = 10

DRAFT
PRINTED : 11-14-08

GEODETIC INFORMATION

SPHEROID:	GRS 1980
SEMI-MAJOR AXIS:	6,378,137.000
SEMI-MINOR AXIS:	6,356,752.314
INVERSE FLATTENING:	298.25722101
ECCENTRICITY^2:	0.006694380
PROJECTION:	LAMBERT CONFORMAL CONIC
ZONE:	5
LONGITUDE OF ORIGIN:	33.5°
FALSE EASTING:	6,561,666
FALSE NORTHING:	1,640,416



FUGRO WEST, INC.
4820 McGrath St., Suite 100,
Ventura, California 93003
Tel: (805) 650-7000,
Fax: (805) 650-7010
www.fugrowest.com

SITE EXPLORATION PLAN AND PROFILES
J Street Drain Improvements
Oxnard, California

NO:	DATE:	DESCRIPTION:	DRAWN:	CHKD:	APPR:
1	Aug 2008	Preliminary Cross Sections	CBD	LB	LB

JOB NUMBER: 3161.014

PLATE 2

APPENDIX A
FIELD EXPLORATION

APPENDIX A
FIELD EXPLORATION

INTRODUCTION

The contents of this appendix shall be integrated with the geotechnical engineering study of which it is a part. They shall not be used in whole or in part as a sole source for information or recommendations regarding the subject site.

GENERAL

The field exploration for this geotechnical study consisted of eleven cone penetrometer test soundings (CPTs) and eight hollow-stem-auger drill holes performed over three continuous days beginning April 28, 2008. Three of the hollow-stem-auger drill holes were converted to groundwater monitoring wells and are referred to and labeled as such throughout this report. The approximate locations of the explorations are shown on Plate 1 - Site Location Map and on Plate 2 - Site Exploration Plan and Profiles. The field exploration program was conducted in general accordance with our revised proposal dated May 7, 2007.

CONE PENETRATION TEST SOUNDINGS

CPT soundings were performed by Fugro Consultants, Inc., of Santa Fe Springs, California, in accordance with ASTM D5778, using a 25-ton rig. The CPT soundings were advanced to depths of about 33 feet to 50 feet below the existing ground surface. Data from the CPT soundings consist of plots of sleeve friction, tip resistance, friction ratio, pore pressure, and equivalent blow count (N60) relative to depth, which are presented on Plates A-1 through A-11 - Log of CPT. A soil classification chart is presented on Plate A-12 - Key to CPT logs.

BORINGS AND MONITORING WELLS

Hollow-stem-auger drilling services were performed by Martini Drilling Corporation of Los Alamitos, California. The hollow-stem-auger borings were advanced using a truck-mounted CME 85 drilling rig equipped with an 8-inch-diameter hollow-stem-auger. Drilling was performed under the observation of a Fugro West, Inc., staff engineer, who prepared a field log of the soil conditions and obtained soil samples for laboratory observation and testing. Soils were classified in the field in general accordance with the Unified Soil Classification System. Hollow-stem-auger borings holes were excavated to a depth of about 30 to 40 feet below the ground surface.

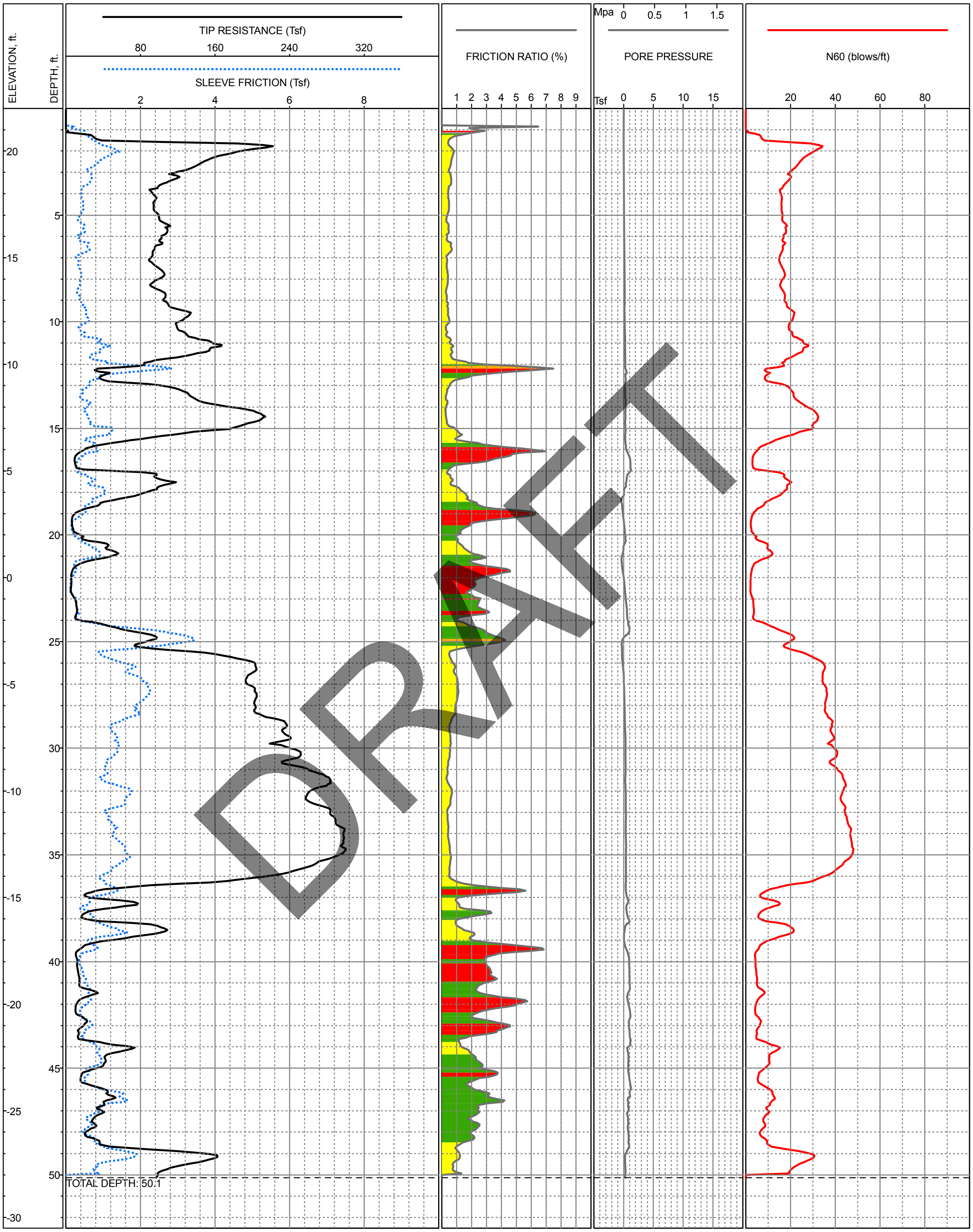
Drive samples were obtained from the hollow-stem-auger drill hole using either modified California or Standard Penetration Test (SPT) samplers. The modified California sampler has a 3-inch-outside-diameter and a 2-3/8-inch-inside-diameter. Samplers were driven into the material at the bottom of the drill hole using a 140-pound CME automatic trip hammer with a 30-inch drop. The number of blows required to drive the California or SPT sampler was recorded on the boring logs in general accordance with ASTM D1586. Recovered samples were placed in transport containers and returned to the laboratory for further classification and testing.

Five of the borings holes were backfilled with a hydrated mixture of bentonite chips and excavated soil cuttings and patched with quick set concrete upon completion. Three of the borings were set with 30-foot deep groundwater monitoring wells upon completion of logging and sampling. The well materials consisted of 15 feet of slotted PVC pipe and 15 feet of solid PVC pipe. The well pipe was backfilled with graded sand to within 5 feet of the ground surface. The upper 5 feet was backfilled with hydrated bentonite chips. A traffic well box was installed at the surface to protect each well.

BORING AND MONITORING WELL LOGS

Boring and monitoring well logs showing the depths and descriptions of soils encountered, geologic structure where applicable, vertical locations of samples, sampler blow counts, and results of density and moisture content tests, are presented as Plates A-13 through A-20 - Log of Boring. A legend of symbols typically used on boring logs hole logs is provided on Plates A-21 and A-22 - Key to Terms and Symbols Used on Logs. The logs represent the interpretation of the visual observation and field tests, interpolation between samples, and laboratory test results. Stratification lines between materials are approximate boundaries between soil types; transitions between soil types can be gradual.

HDR Engineering, Inc.
Project No. 3161.014



LOCATION: See Plate 2
SURFACE EL: 22.0ft +/- (MSL)
COMPLETION DEPTH: 50.1ft
TESTDATE: 4/29/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: L Berry

LOG OF CPT-1

J Street Drain Improvements
Oxnard, California

PLATE A-1

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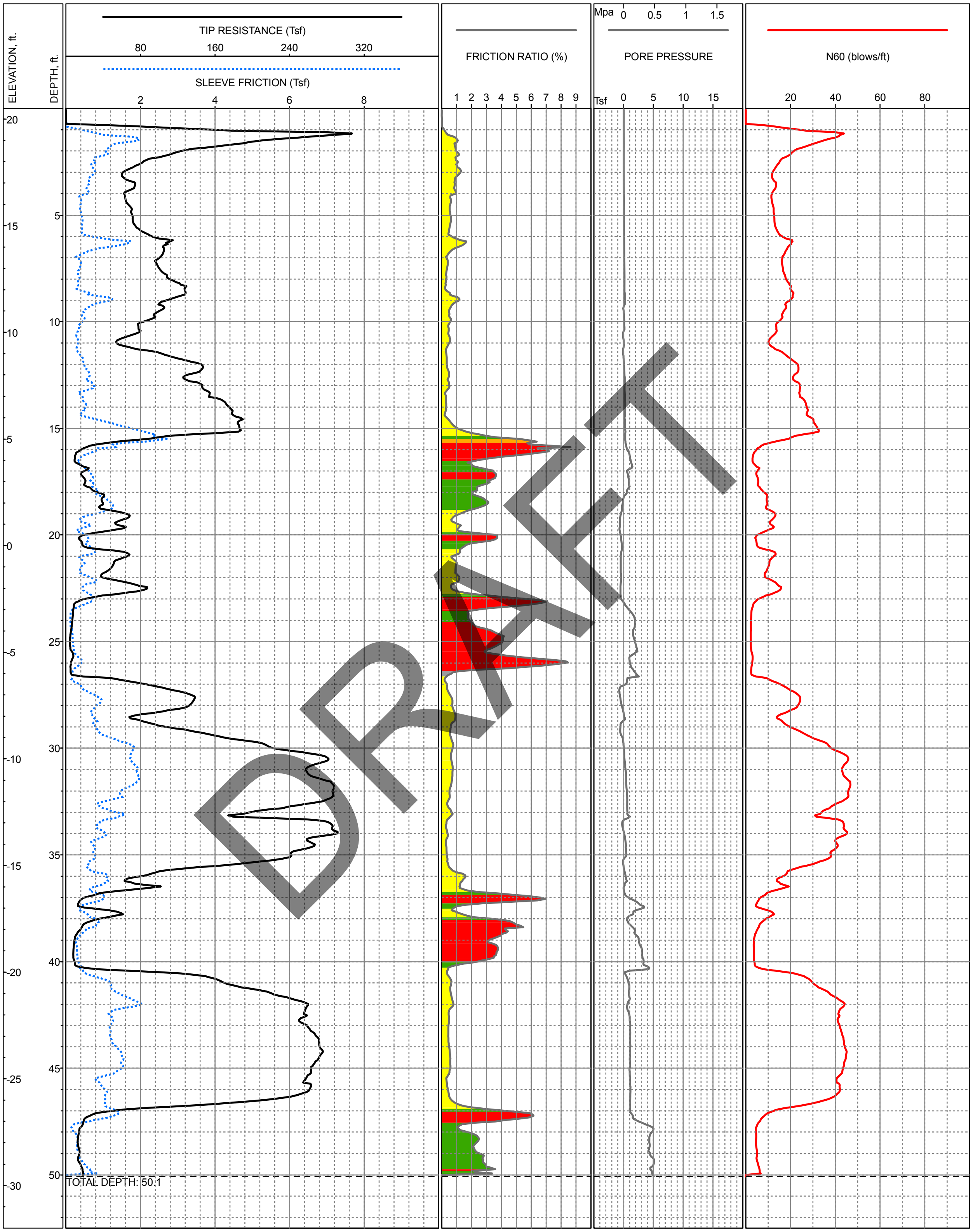
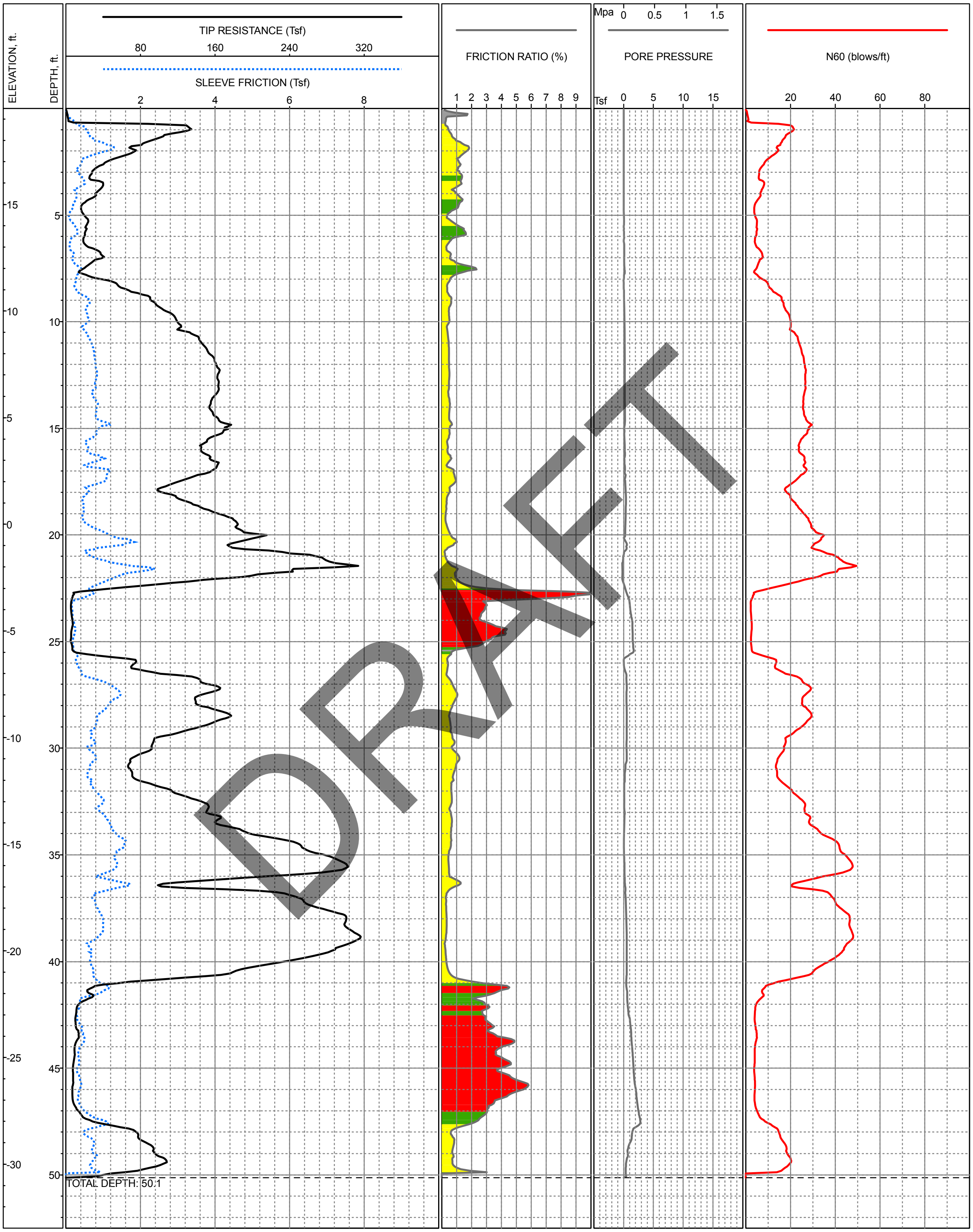


LOCATION: See Plate 2
SURFACE EL: 20.5ft +/- (MSL)
COMPLETION DEPTH: 50.1ft
TESTDATE: 4/30/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: L Berry

LOG OF CPT-2
J Street Drain Improvements
Oxnard, California



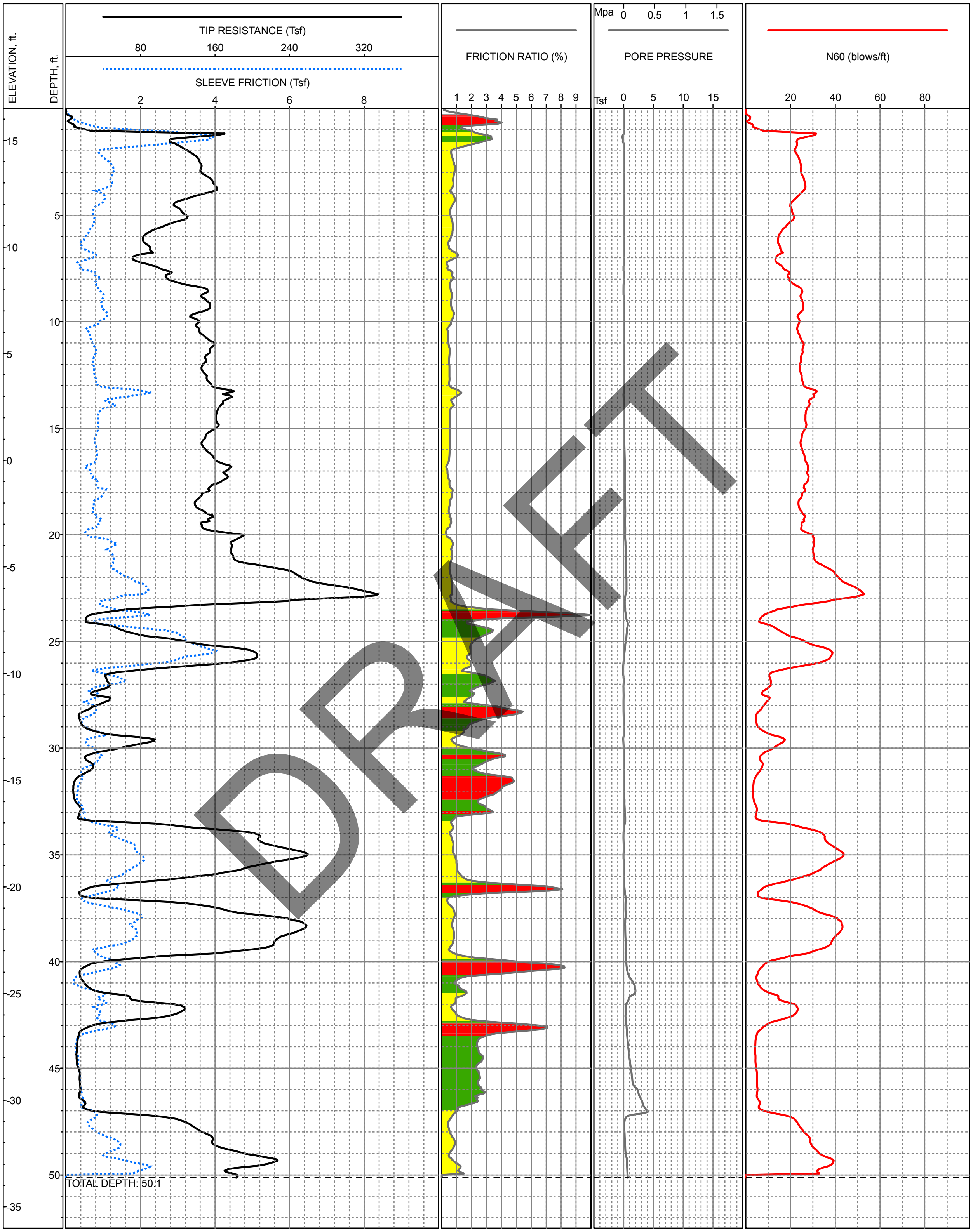
LOCATION: See Plate 2
SURFACE EL: 19.5ft +/- (MSL)
COMPLETION DEPTH: 50.1ft
TESTDATE: 4/29/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: L Berry

LOG OF CPT-3

J Street Drain Improvements
Oxnard, California

HDR Engineering, Inc.
Project No. 3161.014



LOCATION: See Plate 2
SURFACE EL: 16.5ft +/- (MSL)
COMPLETION DEPTH: 50.1ft
TESTDATE: 4/29/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: L Berry

LOG OF CPT-4
J Street Drain Improvements
Oxnard, California

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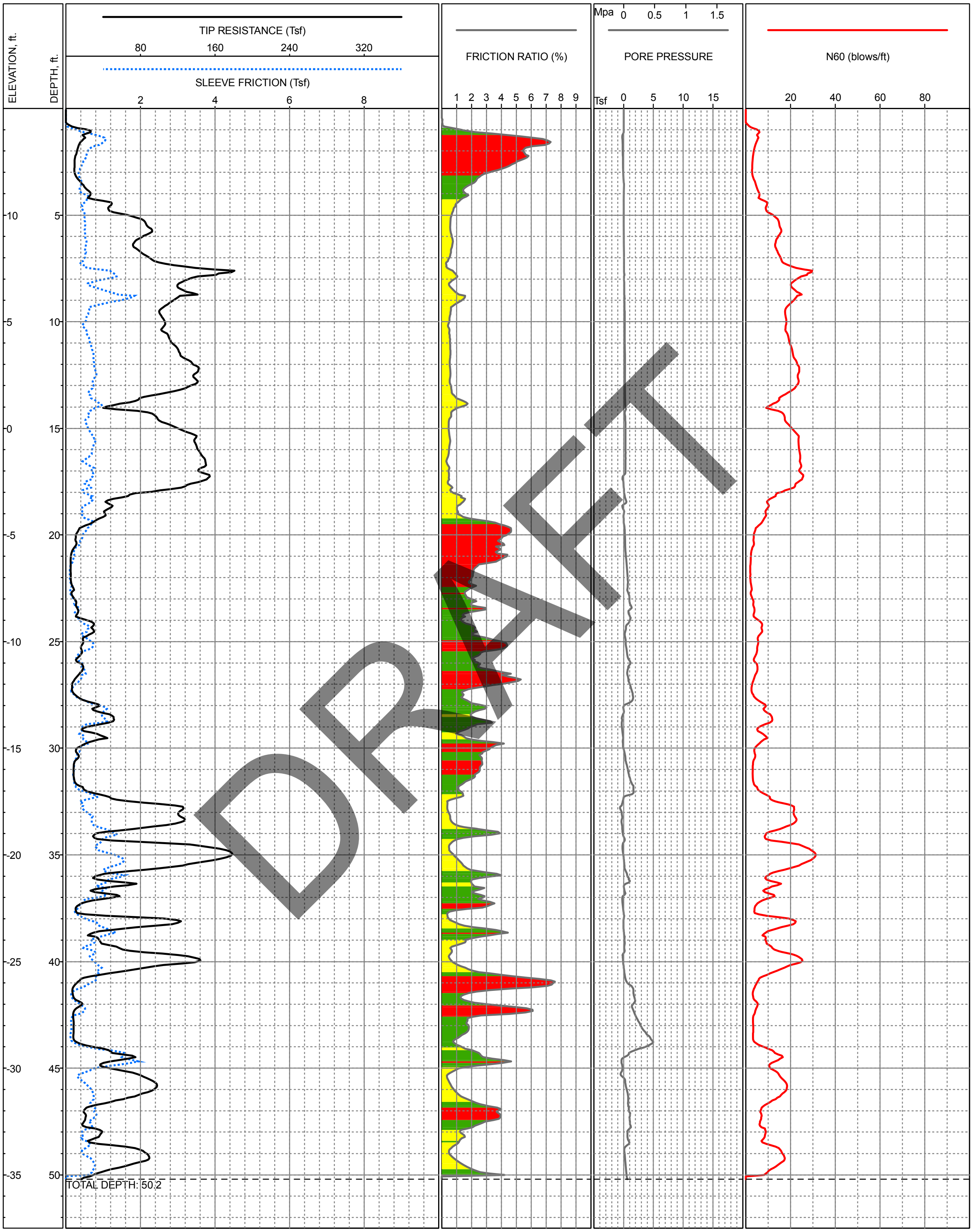
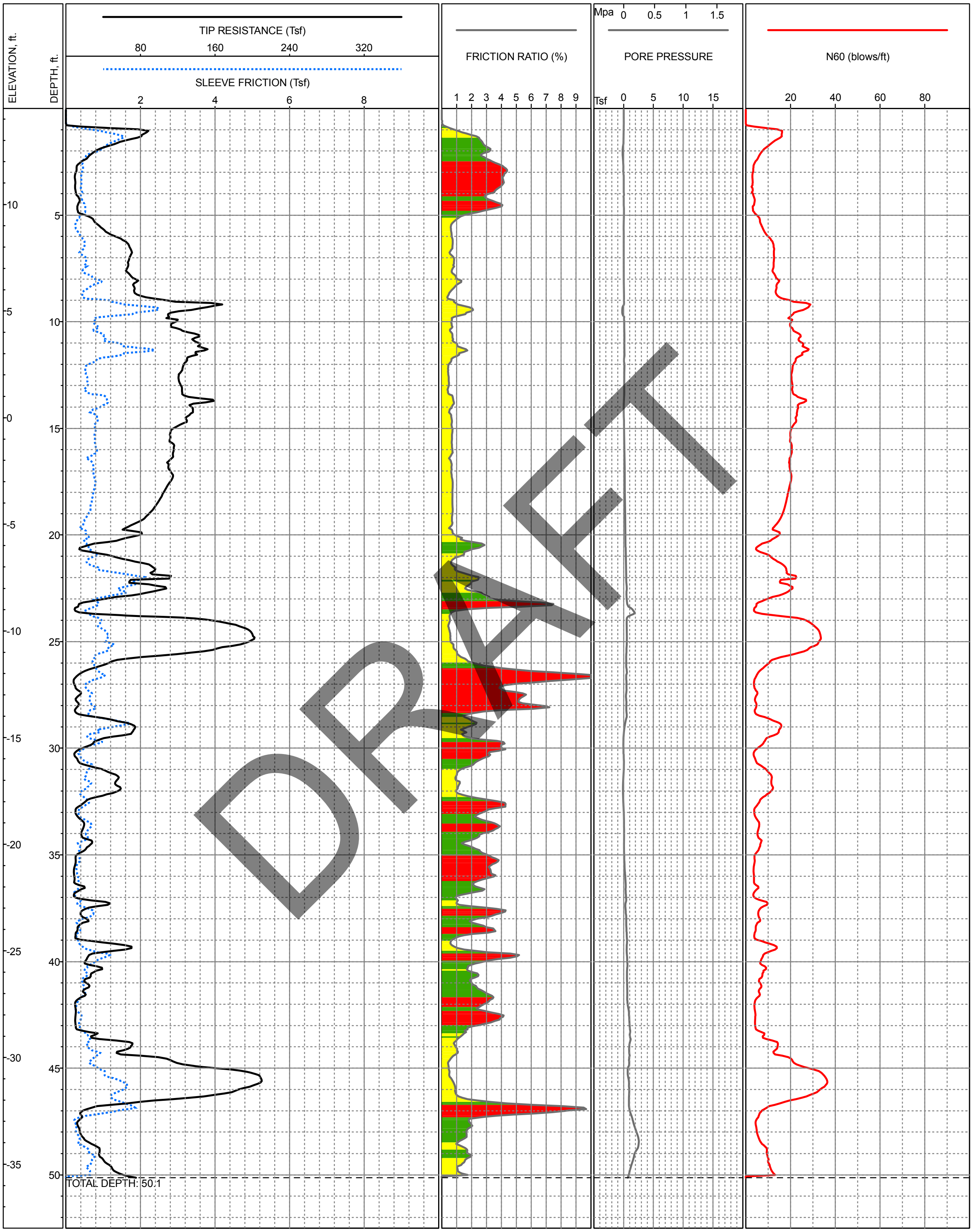


LOCATION: See Plate 2
SURFACE EL: 15.0ft +/- (MSL)
COMPLETION DEPTH: 50.2ft
TESTDATE: 4/30/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: L Berry

LOG OF CPT-5

J Street Drain Improvements
Oxnard, California



LOCATION: See Plate 2
SURFACE EL: 14.5ft +/- (MSL)
COMPLETION DEPTH: 50.1ft
TESTDATE: 4/29/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: L Berry

LOG OF CPT-6

J Street Drain Improvements
Oxnard, California

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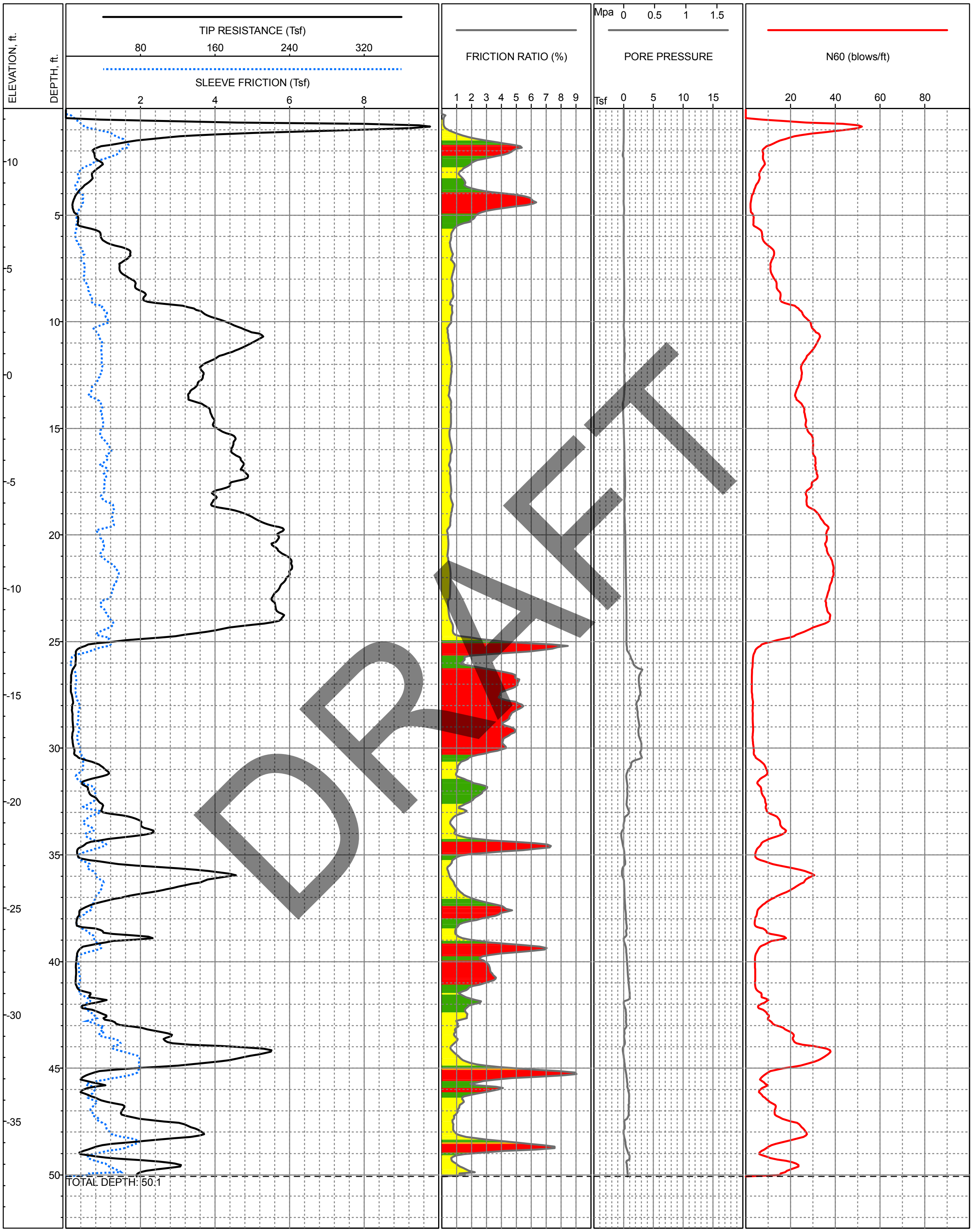


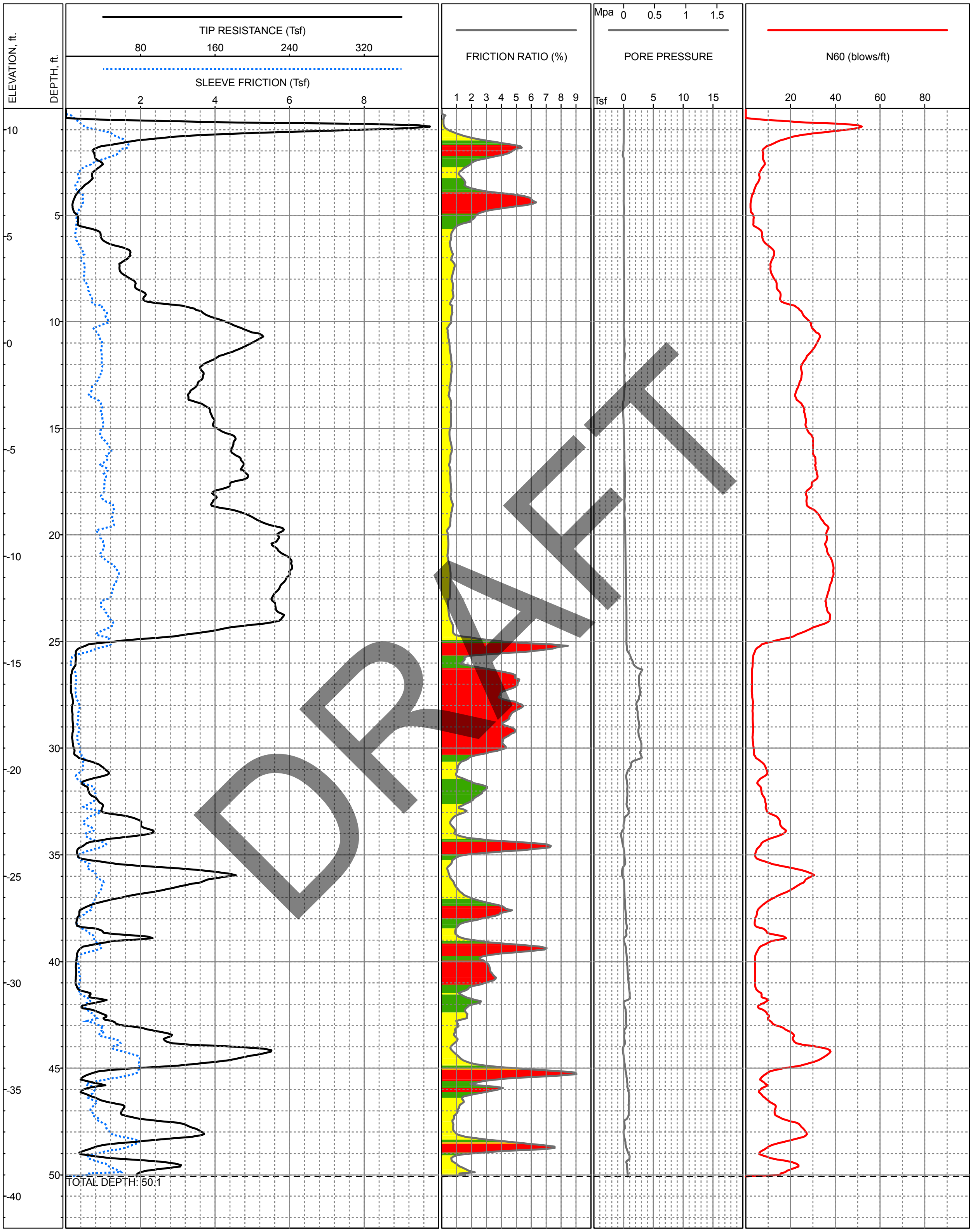
LOCATION: See Plate 2
SURFACE EL: 12.5ft +/- (MSL)
COMPLETION DEPTH: 50.1ft
TESTDATE: 4/30/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: L Berry

LOG OF CPT-7

J Street Drain Improvements
Oxnard, California

N:\Projects\3161_HDREngineering\3161-014_JStreetDrain\Explorations\CPT\2008\Logs\Logs_2008_05_27\MXD\CPTLogs.mxd,05/27/2008,CDean

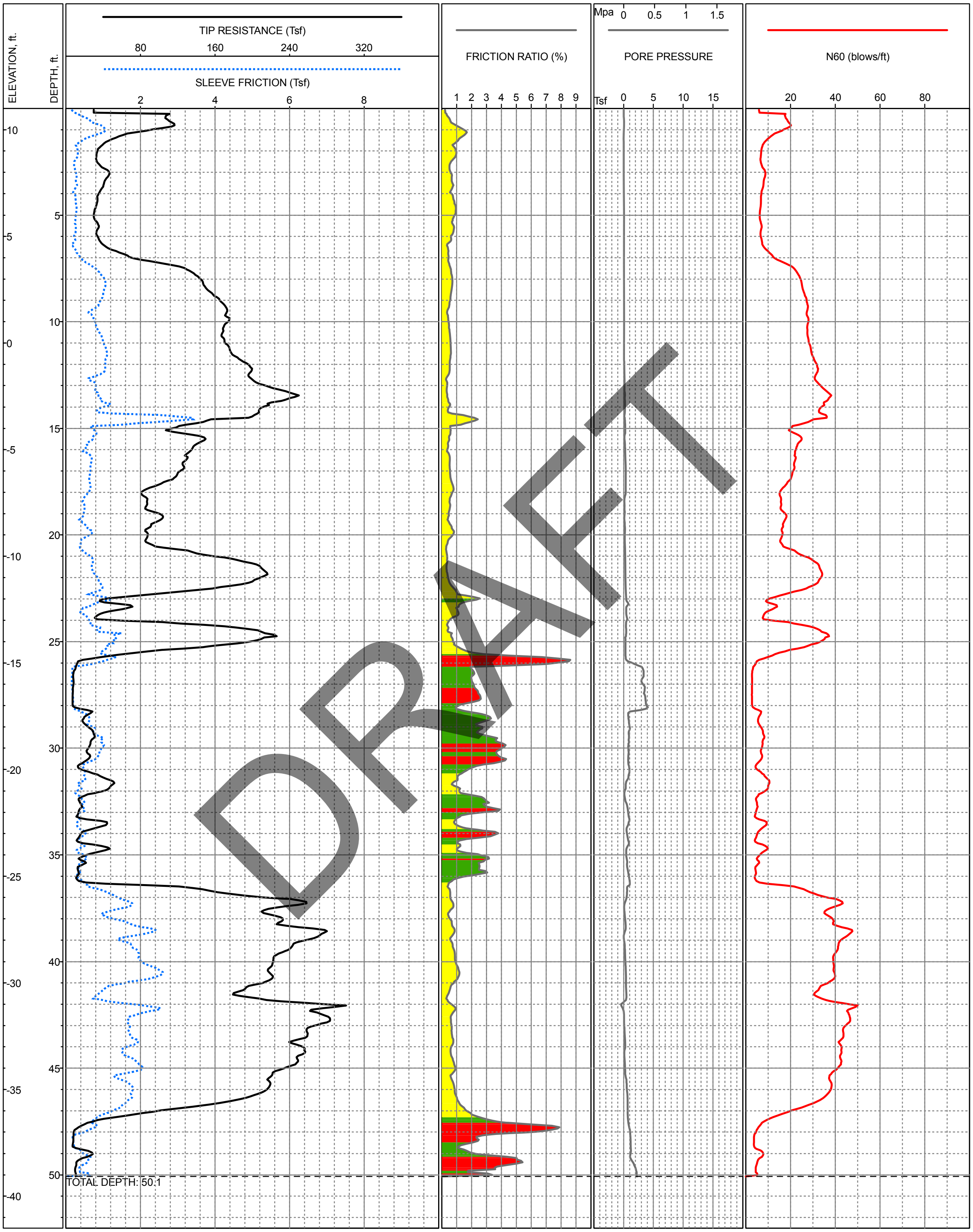


LOCATION: See Plate 2
SURFACE EL: 11.0ft +/- (MSL)
COMPLETION DEPTH: 50.1ft
TESTDATE: 4/30/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: L Berry

LOG OF CPT-8

J Street Drain Improvements
Oxnard, California

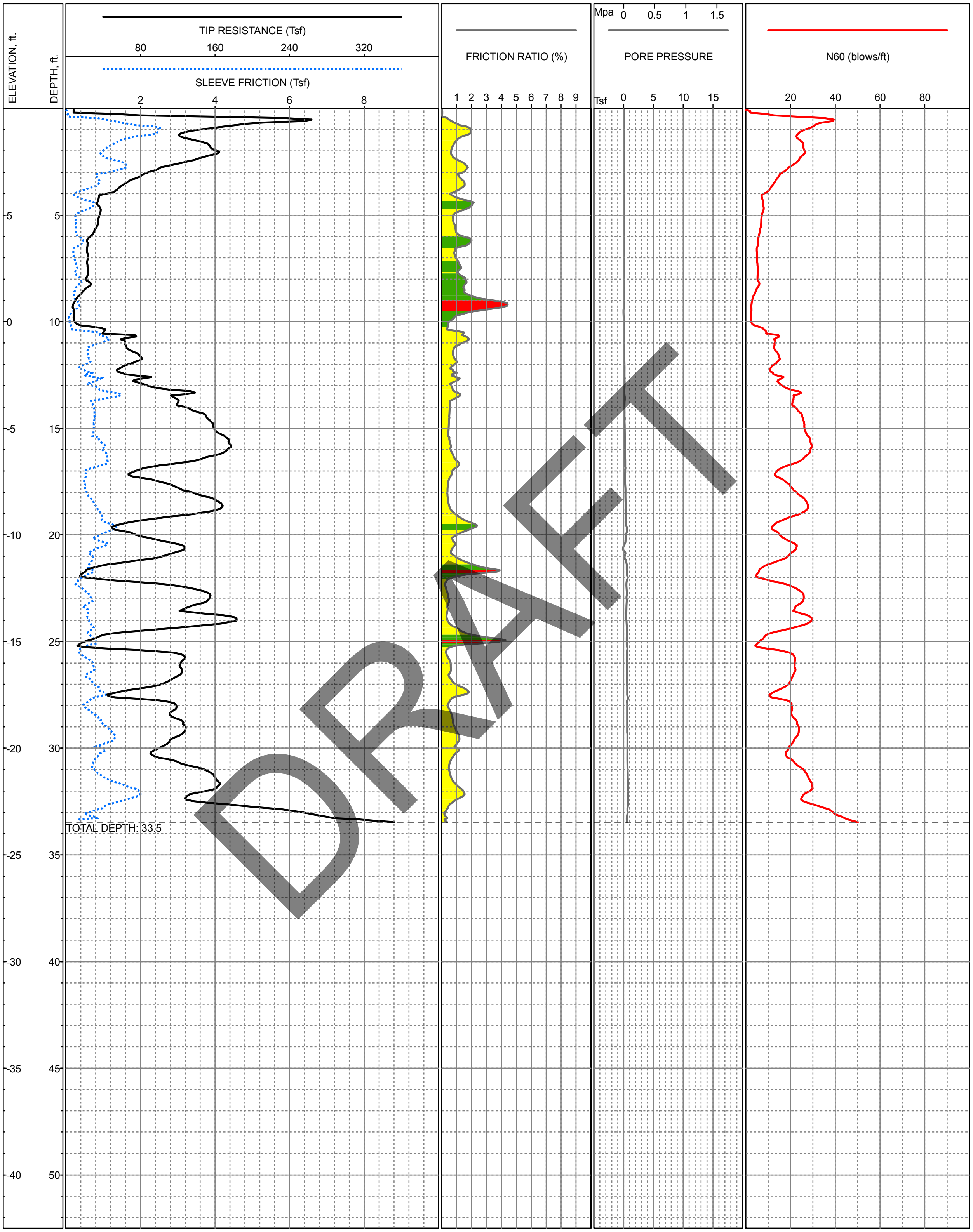


LOCATION: See Plate 2
SURFACE EL: 11.0ft +/- (MSL)
COMPLETION DEPTH: 50.1ft
TESTDATE: 4/30/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: L Berry

LOG OF CPT-9

J Street Drain Improvements
Oxnard, California



LOCATION: See Plate 2
SURFACE EL: 10.0ft +/- (MSL)
COMPLETION DEPTH: 33.5ft
TESTDATE: 4/30/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: L Berry

LOG OF CPT-10
J Street Drain Improvements
Oxnard, California

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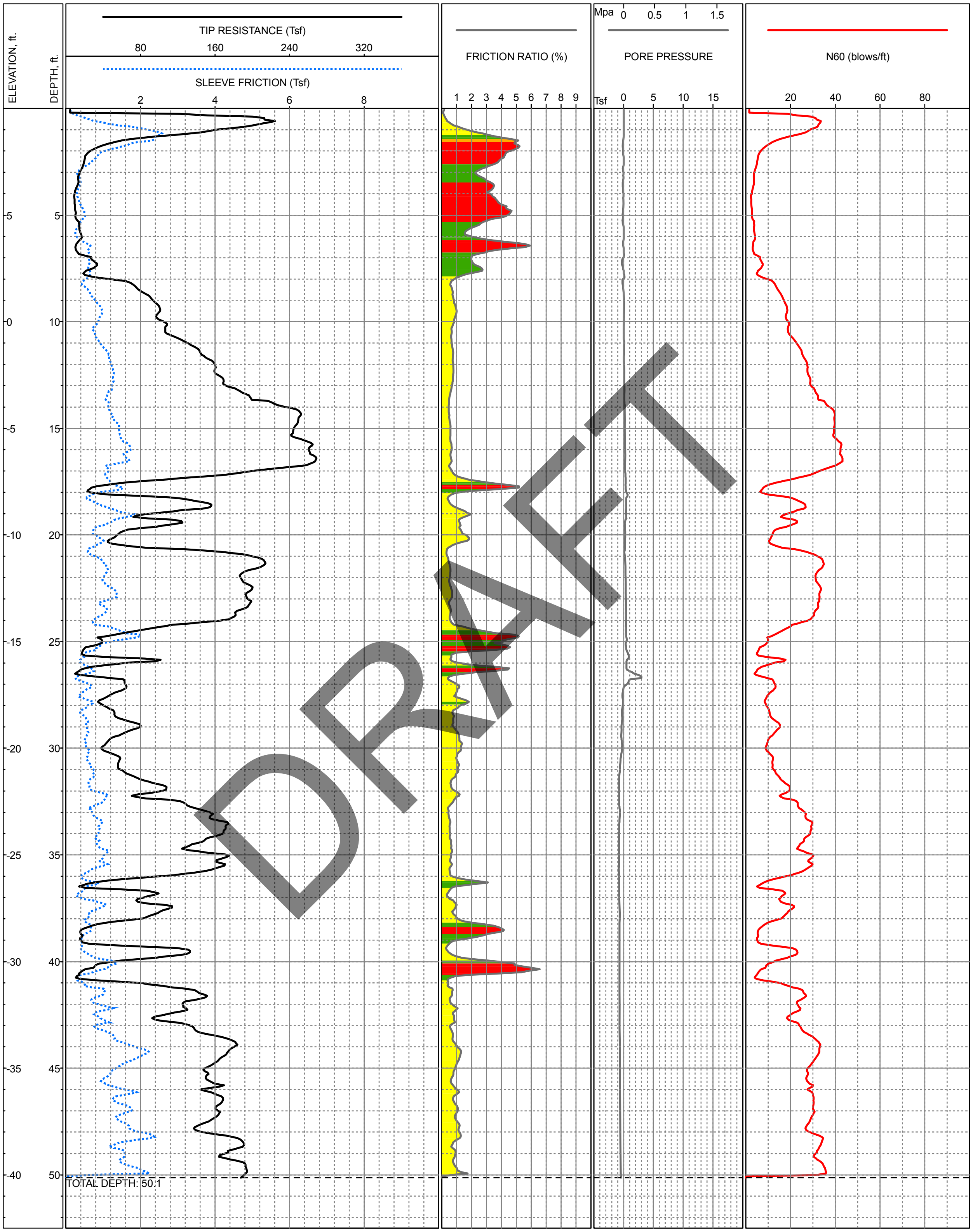

LOCATION: See Plate 2
SURFACE EL: 10.0ft +/- (MSL)
COMPLETION DEPTH: 50.1ft
TESTDATE: 4/30/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: L Berry

LOG OF CPT-11
J Street Drain Improvements
Oxnard, California

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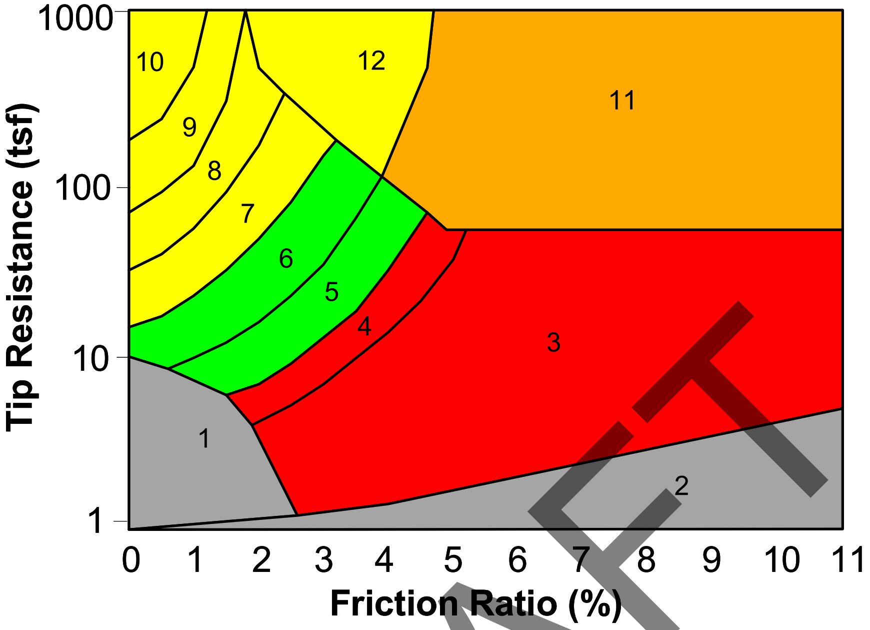
COLOR LEGEND FOR FRICTION RATIO TRACES



Zone	Soil Behavior Type	U.S.C.S.
1	Sensitive Fine-grained	OL-CH
2	Organic Material	OL-OH
3	Clay	CH
4	Silty Clay to Clay	CL-CH
5	Clayey Silt to Silty Clay	MH-CL
6	Sandy Silt to Clayey Silt	ML-MH
7	Silty Sand to Sandy Silt	SM-ML
8	Sand to Silty Sand	SM-SP
9	Sand	SW-SP
10	Gravelly Sand to Sand	SW-GW
11	Very Stiff Fine-grained *	CH-CL
12	Sand to Clayey Sand *	SC-SM

*overconsolidated or cemented

CPT CORRELATION CHART
(Robertson and Campanella, 1984)

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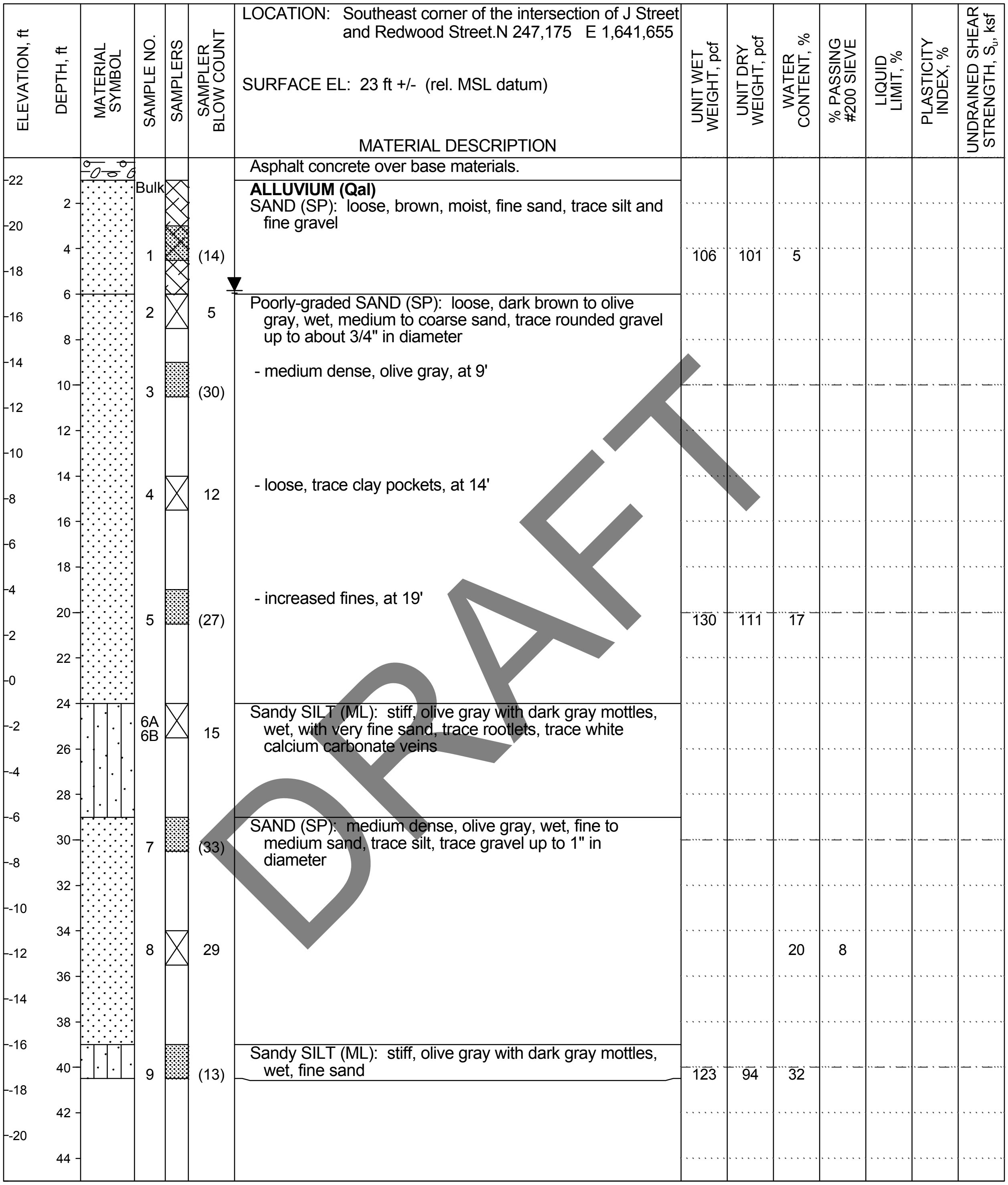
KEY TO CPT LOGS

J Street Drain Improvements
Oxnard, California



LOCATION: Southeast corner of the intersection of J Street and Redwood Street.N 247,175 E 1,641,655

SURFACE EL: 23 ft +/- (rel. MSL datum)

SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
		Asphalt concrete over base materials.							
Bulk		**ALLUVIUM (Qal)** SAND (SP): loose, brown, moist, fine sand, trace silt and fine gravel							
1	(14)		106	101	5				
2	5	Poorly-graded SAND (SP): loose, dark brown to olive gray, wet, medium to coarse sand, trace rounded gravel up to about 3/4" in diameter							
3	(30)	- medium dense, olive gray, at 9'							
4	12	- loose, trace clay pockets, at 14'							
5	(27)	- increased fines, at 19'	130	111	17				
6A 6B	15	Sandy SILT (ML): stiff, olive gray with dark gray mottles, wet, with very fine sand, trace rootlets, trace white calcium carbonate veins							
7	(33)	SAND (SP): medium dense, olive gray, wet, fine to medium sand, trace silt, trace gravel up to 1" in diameter							
8	29				20	8			
9	(13)	Sandy SILT (ML): stiff, olive gray with dark gray mottles, wet, fine sand	123	94	32				

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 40.5 ft
DEPTH TO WATER: 5.9 ft
BACKFILLED WITH: Cuttings and bentonite, topped with concrete.
DRILLING DATE: April 28, 2008

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: Martini Drilling Corporation
LOGGED BY: K. Nelson
CHECKED BY: LE Prentice R.G.

LOG OF BORING NO. DH-01

J Street Drain Improvements
Oxnard, California

PLATE A-13

LOCATION: Southeast corner of the intesection of J Street and Yucca Street.N 244,411 E 1,641,596

SURFACE EL: 18.8 ft +/- (rel. MSL datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
18						**ALLUVIUM (Qal)**							
	2		1	▨	(11)	Poorly-graded SAND (SP): loose, brown, slightly moist, medium to coarse sand, with fine to coarse subangular gravel	107	103	4				
	4		2	☒	3	- trace subangular gravel, at 3'							
	6		3	▨	(5) ▼ ▽	- wet, increased subangular fine gravel, at 5'	122	107	14				
	10		4	☒	8	Well-graded SAND with gravel (SW): loose, brown, wet, fine to coarse sand, fine to coarse subrounded gravel							
-4			5	▨	(42)	Well-graded SAND (SW): medium dense, gray, wet, fine to medium sand, trace clay and fine gravel	135	117	15				
	20		6	☒	34	- dense, at 19'							
-6			7	▨	(22)	Clayey SAND (SC): medium dense, gray, moist, fine sand	135	117	16	23			
	30		8	☒	29	Silty SAND (SM): meidum dense, gray, wet, fine sand							
-16			9	▨	(79)	Poorly-graded SAND (SP): dense, gray, moist, medium sand	133	114	17				
	40		10	☒	31	- wet, at 39'							

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 40.5 ft
DEPTH TO WATER: 5.0 ft
BACKFILLED WITH: Cuttings and bentonite, topped with concrete.
DRILLING DATE: April 29, 2008

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: Martini Drilling Corporation
LOGGED BY: J. Hutchins
CHECKED BY: LE Prentice R.G.

LOG OF BORING NO. DH-02

J Street Drain Improvements
Oxnard, California

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: House No. 3935 on J Street, between Glacier Avenue and Bard Road.N 243,592 E 1,641,515 SURFACE EL: 17.8 ft +/- (rel. MSL datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
			Bulk	☒		**ARTIFICIAL FILL (af)** Clayey SAND (SC): dark brown, moist							
16	2												
14	4		1	▨	(11)	**ALLUVIUM (Qal)** Silty SAND (SM): loose, dark brown, wet, very fine sand Sandy Lean CLAY (CL): dark brown / reddish brown, moist, trace rootlets	123	100	23	58			
12	6		2A 2B	☒	6	▼ Poorly-graded SAND (SP): medium dense, gray, wet, medium to coarse sand, trace subrounded fine gravel			18				
10	8												
8	10		3	▨	(40)								
6	12												
4	14		4	☒	14	- fine to medium sand, olive gray, at 14'			16				
2	16												
0	18												
-2	20		5	▨	(42)	SAND (SP): medium dense, olive gray, wet, fine sand, trace silt	128	106	20	8			
-4	22												
-6	24		6	☒	4	Fat CLAY with sand (CH): medium stiff, olive gray, moist, fine sand - grades to sandy fat clay			30		60	35	
-8	26												
-10	28												
-12	30		7	▨	(40)	Poorly-graded SAND (SP): medium dense, olive gray, wet, fine to medium sand	130	110	18	5			
-14	32												
-16	34		8	☒	47	- dense, trace subrounded gravel up to 1" in diameter, at 34'							
-18	36												
-20	38					- very dense, increased fines, at 39'							
-22	40		9	▨	(87)		130	111	17				
-24	42												
-26	44												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 40.5 ft
DEPTH TO WATER: 6.9 ft
BACKFILLED WITH: Cuttings and bentonite, topped with concrete.
DRILLING DATE: April 28, 2008

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: Martini Drilling Corporation
LOGGED BY: K. Nelson
CHECKED BY: LE Prentice R.G.

LOG OF BORING NO. DH-03
J Street Drain Improvements
Oxnard, California

PLATE A-15

LOCATION: House No. 4920 on J Street, just north of Sonoma Way.N 240,783 E 1,641,514

SURFACE EL: 13 ft +/- (rel. MSL datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						5" Asphalt Concrete over 5" Base Materials							
12													
	2												
10			1	☒	27	**ALLUVIUM (Qal)** Clayey SAND (SC): medium dense, brown, slightly moist, fine to coarse sand			10	30			
	4												
8													
	6		2	▨	(11) ▼	- loose, with clay pockets, below 6'	124	108	15				
6													
	8				▽	Sandy Lean CLAY (CL): 6" thick							
4			3	☒	5	Poorly-graded SAND (SP): loose, brown, wet, medium to coarse sand, trace clay			18				
	10												
2													
	12												
0													
	14												
-2			4A 4B	▨	(30)	- medium dense, fin to coarse sand, with fine to coarse subrounded gravel, at 14'	127	109	17				
	16												
-4													
	18					Well-graded SAND (SW): medium dense, gray, trace fine to coarse subrounded gravel							
-6													
	20		5	☒	20								
-8													
	22												
-10													
	24					Silty Fine SAND (SM): 6" thick							
-12			6A 6B	▨	(30)	Well-graded SAND (SW): medium dense, gray, trace fine to coarse subrounded gravel	133	110	21				
	26												
-14													
	28												
-16													
	30		7	☒	6	Lean CLAY with sand (CL): medium stiff, gray with white and orange mottles, moist, low plasticity, fine sand			28		30	8	p 1.5
-18													
	32												
-20													
	34												
-22			8	▨	(22)	Sandy SILT (ML): stiff, dark gray, moist, very fine sand, with silty sand seams	125	94	33				p 1.5
	36												
-24													
	38												
-26													
	40		9	☒	7								
-28													
	42												
-30													
	44												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 40.5 ft
DEPTH TO WATER: 6.7 ft
BACKFILLED WITH: Cuttings and bentonite, topped with concrete.
DRILLING DATE: April 29, 2008

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: Martini Drilling Corporation
LOGGED BY: J. Hutchins
CHECKED BY: LE Prentice R.G.

LOG OF BORING NO. DH-04

J Street Drain Improvements
Oxnard, California

PLATE A-16

HDR Engineering, Inc.
Project No. 3161.014

LOCATION: Adjacent to the pump station, north of Ocean View Drive.N 235,240 E 1,640,668

SURFACE EL: 12.5 ft +/- (rel. MSL datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
12			Bulk	☒		**ARTIFICIAL FILL (af)** Silty SAND (SM): brown to dark brown, damp, fine to medium sand, trace clay, trace gravel up to 1" in diameter							
10	2			☒									
8	4		1	▨	(14)	**ALLUVIUM (Qal)** Poorly-graded SAND (SP): loose, brown, damp, medium to coarse sand, trace silt	102	97	6				
6	6												
4	8		2	☒	3	- fine to medium sand, at 7' - 1" thick clay lense, at 7.5'							
2	10		3	▨	(20) ▼	Silty SAND (SM): medium dense, brown, wet, fine sand	120	101	18				
0	12												
-2	14		4	☒	17	Poorly-graded SAND (SP): medium dense, brown to olive gray, wet, fine sand, trace silt - dark gray, below 15'							
-4	16												
-6	18												
-8	20		5	▨	(53)	- dense, fine to medium sand, at 19'	132	108	21				
-10	22												
-12	24		6	☒	23	- medium dense, at 24'							
-14	26												
-16	28												
-18	30		7	▨	(5)	Fat CLAY (CH): soft, olive gray, wet, trace fine sand	115	80	44		52	27	
-20	32												
-22	34		8	☒	18	Silty SAND (SM): medium dense, dark gray, wet, fine to coarse sand							
-24	36												
-26	38												
-28	40		9	▨	(36)	- fine sand below 39'	125	97	29	25			
-30	42												
-32	44												

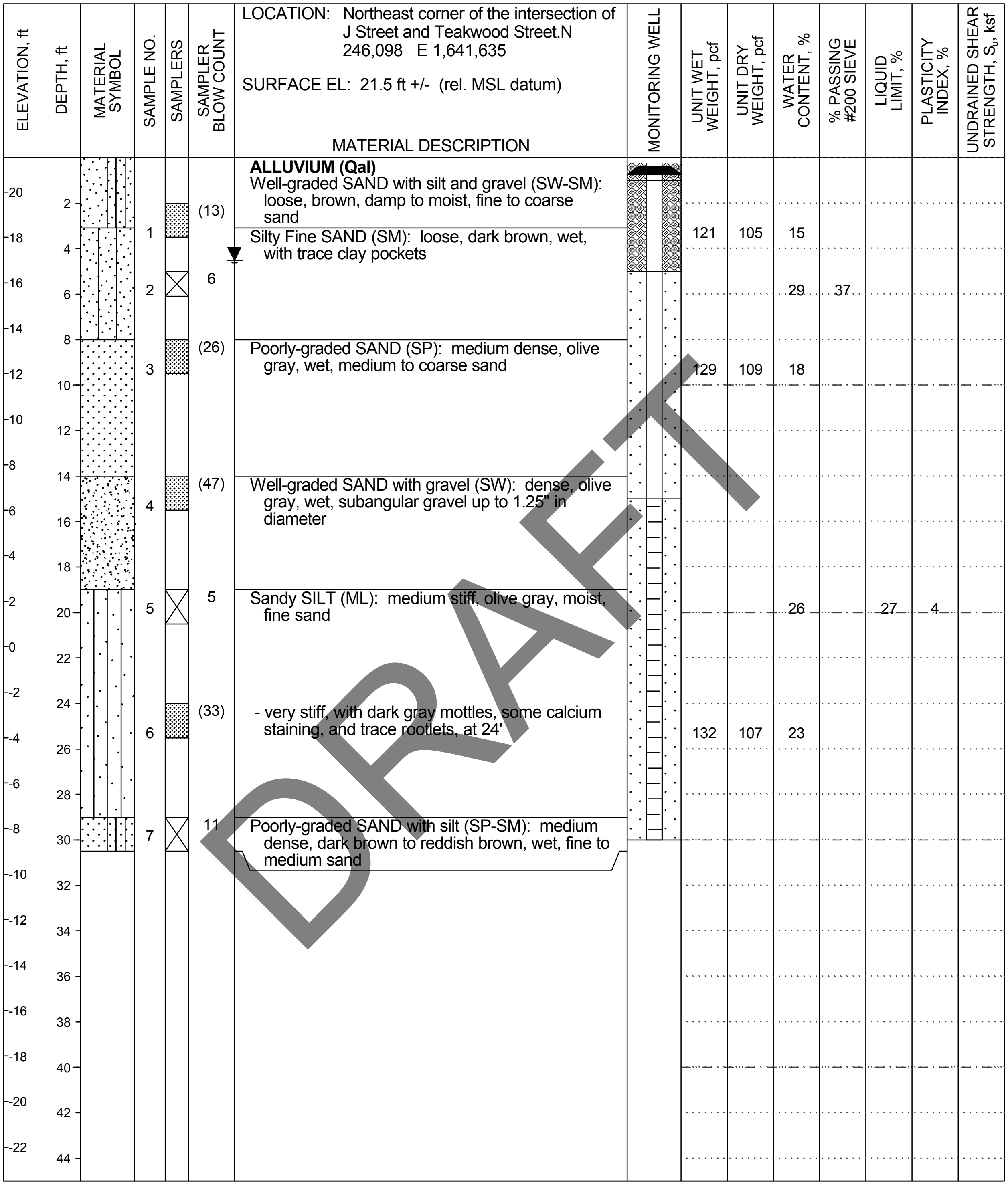
The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 40.5 ft
DEPTH TO WATER: 11.3 ft
BACKFILLED WITH: Cuttings and bentonite, topped with concrete.
DRILLING DATE: April 28, 2008

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: Martini Drilling Corporation
LOGGED BY: K. Nelson
CHECKED BY: LE Prentice R.G.

LOG OF BORING NO. DH-05

J Street Drain Improvements
Oxnard, California



LOCATION: Northeast corner of the intersection of J Street and Teakwood Street.N 246,098 E 1,641,635

SURFACE EL: 21.5 ft +/- (rel. MSL datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	MONITORING WELL	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
20	2					**ALLUVIUM (Qal)** Well-graded SAND with silt and gravel (SW-SM): loose, brown, damp to moist, fine to coarse sand								
18	4		1	▨	(13) ▼	Silty Fine SAND (SM): loose, dark brown, wet, with trace clay pockets		121	105	15				
16	6		2	☒	6					29	37			
14	8					Poorly-graded SAND (SP): medium dense, olive gray, wet, medium to coarse sand								
12	10		3	▨	(26)			129	109	18				
10	12													
8	14					Well-graded SAND with gravel (SW): dense, olive gray, wet, subangular gravel up to 1.25" in diameter								
6	16		4	▨	(47)									
4	18													
2	20		5	☒	5	Sandy SILT (ML): medium stiff, olive gray, moist, fine sand				26		27	4	
0	22													
-2	24					- very stiff, with dark gray mottles, some calcium staining, and trace rootlets, at 24'								
-4	26		6	▨	(33)			132	107	23				
-6	28													
-8	30		7	☒	11	Poorly-graded SAND with silt (SP-SM): medium dense, dark brown to reddish brown, wet, fine to medium sand								
-10	32													
-12	34													
-14	36													
-16	38													
-18	40													
-20	42													
-22	44													

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 30.5 ft
DEPTH TO WATER: 4.5 ft
BACKFILLED WITH: Well Materials
DRILLING DATE: April 28, 2008

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: Martini Drilling Corporation
LOGGED BY: K. Nelson
CHECKED BY: LE Prentice R.G.

LOG OF BORING NO. MW-01

J Street Drain Improvements
Oxnard, California

PLATE A-18

LOCATION: Southwest corner of the intersection of J Street and Clara Street.N 239,025 E 1,641,416

SURFACE EL: 11.2 ft +/- (rel. MSL datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	MONITORING WELL	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
10						**ALLUVIUM (Qal)** Sandy Clayey SILT (ML-CL): stiff, dark brown, moist, fine sand, trace fine gravel								
	2		1		(16)			127	111	14		25	7	p 4.0
8														
	4		2		3									p 2.0
6														
	6													
4														
	8													
2			3		(5) ▼	Fat CLAY with sand (CH): medium stiff, orange and gray mottled, moist, trace decomposing wood pieces		102	68	51		75	44	p 1.0
	10					Poorly-graded SAND (SP): medium dense, gray, wet, fine to medium sand, trace silt								
0														
	12													
-2														
	14		4		16					23				
-4														
	16													
-6														
	18													
-8			5		(32)	Poorly-graded SAND (SP): medium dense, gray, wet, medium to coarse sand, trace rounded fine gravel		132	113	17				
	20													
-10														
	22													
-12														
	24		6		5									
-14						SILT (ML): medium stiff, gray, wet, trace fine sand								
	26													
-16														
	28													
-18			7		(14)	- stiff, brown, at 29'		125	97	29				p 2.5
	30													
-20														
	32													
-22														
	34													
-24														
	36													
-26														
	38													
-28														
	40													
-30														
	42													
-32														
	44													

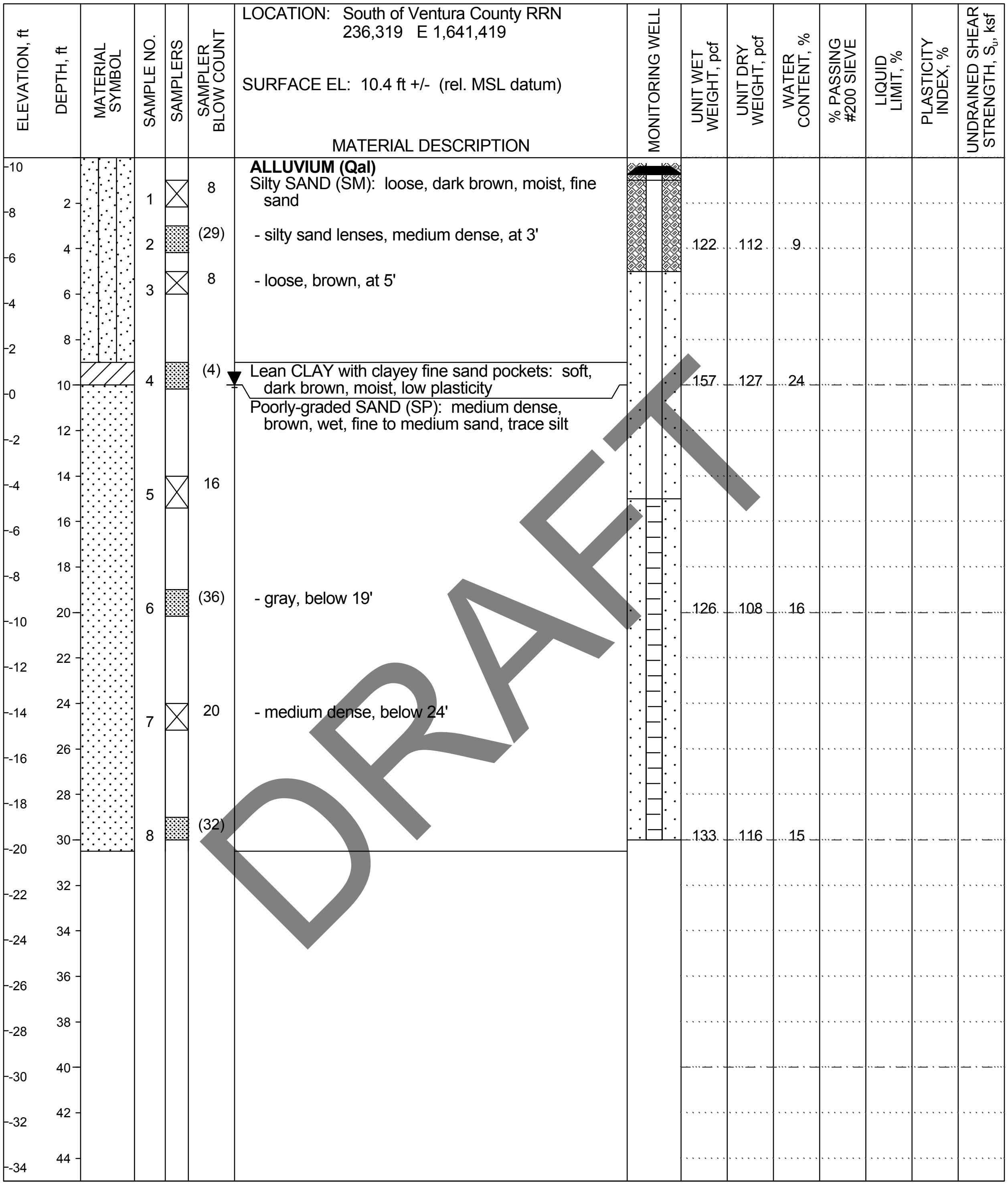
The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 30.5 ft
DEPTH TO WATER: 9.0 ft
BACKFILLED WITH: Well Materials
DRILLING DATE: April 29, 2008

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: Martini Drilling Corporation
LOGGED BY: J. Hutchins
CHECKED BY: LE Prentice R.G.

LOG OF BORING NO. MW-02

J Street Drain Improvements
Oxnard, California



LOCATION: South of Ventura County RRN
236,319 E 1,641,419

SURFACE EL: 10.4 ft +/- (rel. MSL datum)

SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
1	8	**ALLUVIUM (Qal)** Silty SAND (SM): loose, dark brown, moist, fine sand							
2	(29)	- silty sand lenses, medium dense, at 3'	122	112	9				
3	8	- loose, brown, at 5'							
4	(4)	Lean CLAY with clayey fine sand pockets: soft, dark brown, moist, low plasticity Poorly-graded SAND (SP): medium dense, brown, wet, fine to medium sand, trace silt	157	127	24				
5	16								
6	(36)	- gray, below 19'	126	108	16				
7	20	- medium dense, below 24'							
8	(32)		133	116	15				

DRAFT

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 30.5 ft
DEPTH TO WATER: 10.0 ft
BACKFILLED WITH: Well Materials.
DRILLING DATE: April 29, 2008

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: Martini Drilling Corporation
LOGGED BY: J. Hutchins
CHECKED BY: LE Prentice R.G.

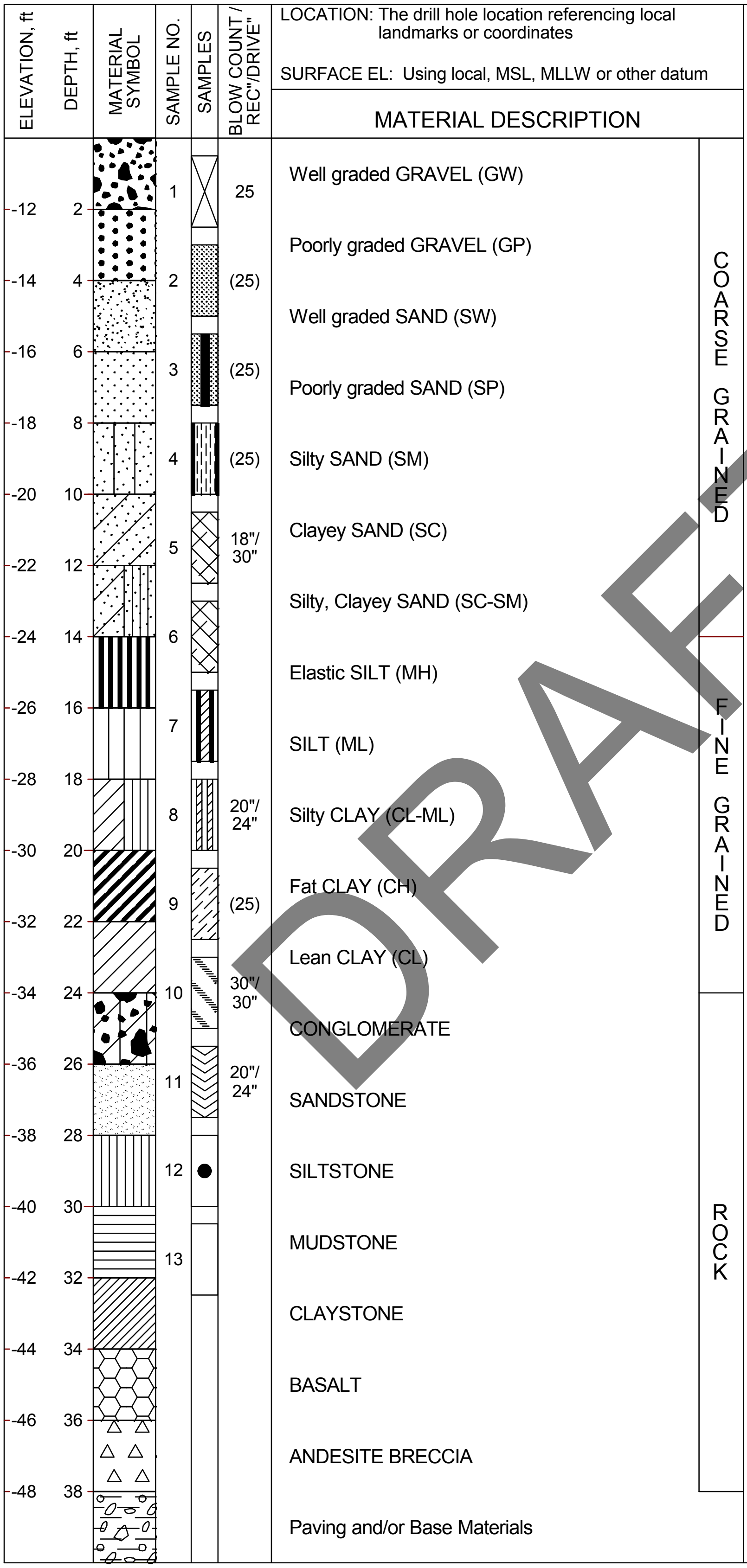
LOG OF BORING NO. MW-03

J Street Drain Improvements
Oxnard, California

PLATE A-20

LOCATION: The drill hole location referencing local landmarks or coordinates

SURFACE EL: Using local, MSL, MLLW or other datum



General Notes

Soil Texture Symbol

Sloped line in symbol column indicates transitional boundary

Samplers and sampler dimensions
(unless otherwise noted in report text) are as follows:

Symbol for:

1	SPT Sampler, driven 1-3/8" ID, 2" OD
2	CA Liner Sampler, driven 2-3/8" ID, 3" OD
3	CA Liner Sampler, disturbed 2-3/8" ID, 3" OD
4	Thin-walled Tube, pushed 2-7/8" ID, 3" OD
5	Bulk Bag Sample (from cuttings)
6	CA Liner Sampler, Bagged
7	Hand Auger Sample
8	CME Core Sample
9	Pitcher Sample
10	Lexan Sample
11	Vibracore Sample
12	No Sample Recovered
13	Sonic Soil Core Sample

Sampler Driving Resistance

Number of blows with 140 lb. hammer, falling 30" to drive sampler 1 ft. after seating sampler 6"; for example,

Blows/ft	Description
25	25 blows drove sampler 12" after initial 6" of seating
86/11"	After driving sampler the initial 6" of seating, 36 blows drove sampler through the second 6" interval, and 50 blows drove the sampler 5" into the third interval
50/6"	50 blows drove sampler 6" after initial 6" of seating
Ref/3"	50 blows drove sampler 3" during initial 6" seating interval

Blow counts for California Liner Sampler shown in ( )

Length of sample symbol approximates recovery length

Classification of Soils per ASTM D2487 or D2488

Geologic Formation noted in bold font at the top of interpreted interval

Strength Legend

Q = Unconfined Compression
u = Unconsolidated Undrained Triaxial
t = Torvane
p = Pocket Penetrometer
m = Miniature Vane

Water Level Symbols

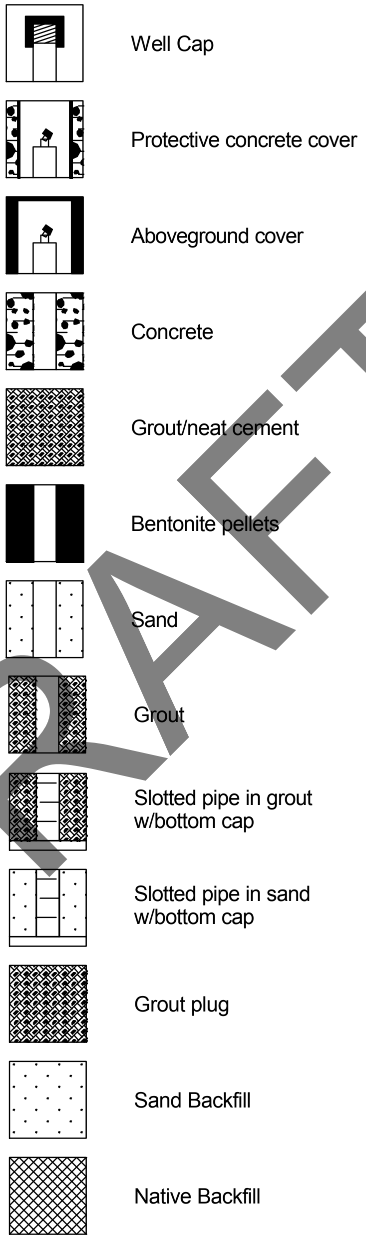
▽	Initial or perched water level
▼	Final ground water level
∿	Seepages encountered

Rock Quality Designation (RQD) is the sum of recovered core pieces greater than 4 inches divided by the length of the cored interval.

KEY TO TERMS & SYMBOLS USED ON LOGS

PLATE A-21

Well Construction Diagram



A. The different types of well constructed include but are not limited to monitoring, vapor extraction, and piezometer.

B. Types and sizes of the materials used are as described in report text.

KEY TO TERMS & SYMBOLS USED ON LOGS (con't)

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

GENERAL

This appendix provides a discussion of the laboratory test program performed for this geotechnical study. Laboratory tests were performed on selected samples obtained from the field to help classify the soils and estimate some of their engineering properties. Laboratory tests were performed in general accordance with American Society for Testing and Materials (ASTM) test procedures.

Driven-ring and bulk samples used in the laboratory-testing program were obtained from various depths during the field exploration, as discussed in Appendix A. Each sample is identified by sample number and depth. Various laboratory tests that were performed are described below.

INDEX PROPERTIES TESTING

Classification

The method of identifying and classifying soils according to their engineering properties used in this study is ASTM Test Method D2487, which is based on the Unified Soil Classification System. Index properties tests discussed in this report include moisture content and dry density measurements, grain size distribution, and plasticity.

Moisture Content and Dry Density

Tests for moisture content of the soils were performed generally according to ASTM Test Method D2216, often in conjunction with other tests. The dry density of selected driven ring samples was obtained by trimming the end of the sample to obtain a smooth, flat face. The trimmed sample was measured to obtain volume and wet weight, extruded, and visually classified. The samples were dried in an oven maintained at approximately 110 degrees Celsius. After drying, each sample was weighed, and the moisture content and dry density were calculated. The moisture content and dry density results are summarized on Plates B-1a and B-1b - Summary of Laboratory Test Results, and also are presented on the drill-hole logs.

Grain Size Distribution

Gradation tests were performed on selected samples in general accordance with ASTM D422. In addition, tests were performed to determine the amount of material in soils finer than the No. 200 sieve in general accordance with ASTM test method D1140-71. These tests were performed to assist in the classification of the soil and to determine its grain size distribution. Results of these tests are presented on Plates B-2a and B-2b - Grain Size Curves. The fines content results are also summarized on Plates B-1a and B-1b - Summary of Laboratory Test Results and presented on the drill hole logs.

Plasticity Index

Atterberg limits (ASTM D4318) were performed on selected fine grained soil samples to measure the range of water contents over which the tested material exhibits plasticity. The limits were used to classify the soil in accordance with the United Soil Classification System and to evaluate the soil expansion potential. Results of the testing are presented on Plate B-3 - Plasticity Chart. The results are also summarized on Plates B-1a and B-1b - Summary of Laboratory Test Results and presented on the drill hole logs.

ENGINEERING PROPERTIES TESTING

Direct Shear

Direct shear tests were performed on two samples to evaluate the strength characteristics of the subsurface soil in accordance with ASTM D3080. The tests were performed at a constant rate of strain based on t_{50} and failure was taken as ultimate normal stress. The results of the direct shear tests are presented on Plates B-4a and 4b - Direct Shear Test Results and are also summarized on Plates B-1a and B-1b - Summary of Laboratory Test Results.

Consolidation

A consolidation test (ASTM D2453) was performed on a ring sample of highly plastic clay to assist in evaluating the compressibility properties of this unit. Results of the consolidation test are presented on Plate B-5 - Consolidation Test Results.

Soil Chemistry Tests

Three suites of soil chemical tests were performed selected samples of the near-surface soils to assess corrosion potential. Chemical tests consisted of pH, sulfate, chloride, and resistivity. Tests were performed by Cooper Testing Laboratory of Palo Alto, California. Results of the chemical tests are presented on Plate 1 - Summary of Laboratory Test Results, Plate B-6 - Corrosivity Test Summary, and summarized in the report text.

DRILL HOLE	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC %	FINES %	ATTERBERG LIMITS		COMPACTION TEST		DIRECT SHEAR		COMPRESSIVE STRENGTH TESTS		CORROSIVITY TESTS				R-VALUE	EXPANSION INDEX	SAND EQUIVALENT (SE)	SPECIFIC GRAVITY
								LL	PI	MAX DD pcf	OPT MC %	C ksf	PHI deg	Qu, ksf	S_u (Cell Prs.) ksf	R	pH	Cl	So_4 (%)				
DH-01	1.0	Bulk	SAND (SP)													7893	9.00	<2	<0.0005				
DH-01	4.0	1	SAND (SP)	106	101	5	7					0.0	39										
DH-01	20.0	5	Poorly-graded SAND (SP)	130	111	17	5																
DH-01	34.5	8	SAND (SP)			20	8																
DH-01	40.0	9	Sandy SILT (ML)	123	94	32																	
DH-02	1.5	1	Poorly-graded SAND (SP)	107	103	4																	
DH-02	5.5	3	Poorly-graded SAND (SP)	122	107	14	3																
DH-02	9.5	4	Well-graded SAND with gravel (SW)				1																
DH-02	14.5	5	Well-graded SAND (SW)	135	117	15	8																
DH-02	24.5	7	Clayey SAND (SC)	135	117	16	23																
DH-02	34.5	9	Poorly-graded SAND (SP)	133	114	17	6																
DH-03	0.3	Bulk	Clayey SAND (SC)													1100	8.10	10	0.05				
DH-03	4.0	1	Sandy Lean CLAY (CL)	123	100	23	58																
DH-03	6.5	2B	Poorly-graded SAND (SP)			18	3																
DH-03	15.0	4	Poorly-graded SAND (SP)			16	8																
DH-03	20.0	5	SAND (SP)	128	106	20	8																
DH-03	24.5	6	Fat CLAY with sand (CH)			30	75	60	35														
DH-03	30.0	7	Poorly-graded SAND (SP)	130	110	18	5																
DH-03	40.0	9	Poorly-graded SAND (SP)	130	111	17																	
DH-04	2.5	1	Clayey SAND (SC)			10	30																
DH-04	5.5	2	Clayey SAND (SC)	124	108	15																	
DH-04	9.0	3	Poorly-graded SAND (SP)			18	4																
DH-04	15.0	4B	Poorly-graded SAND (SP)	127	109	17																	
DH-04	19.5	5	Well-graded SAND (SW)				5																
DH-04	25.0	6B	Well-graded SAND (SW)	133	110	21																	
DH-04	29.5	7	Lean CLAY with sand (CL)			28	73	30	8														
DH-04	34.5	8	Lean CLAY with sand (CL)	125	94	33																	
DH-05	0.0	Bulk	Silty SAND (SM)													1188	8.20	56	0.10				
DH-05	5.0	1	Poorly-graded SAND (SP)	102	97	6	9																
DH-05	11.0	3	Silty SAND (SM)	120	101	18																	

SUMMARY OF LABORATORY TEST RESULTS

J Street Drain Improvements
Oxnard, California

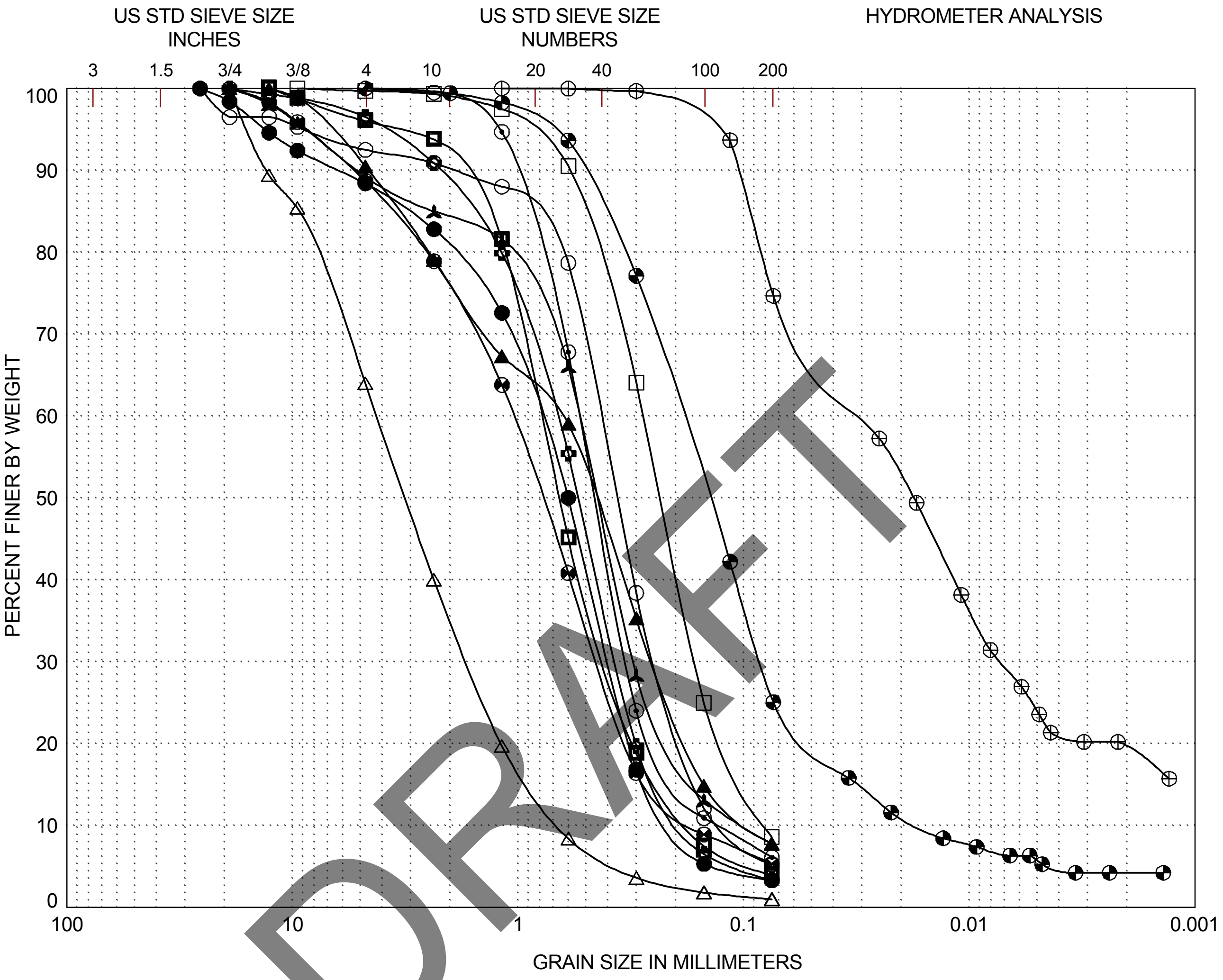
HDR Engineering, Inc.
Project No. 3161.014

DRILL HOLE	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC %	FINES %	ATTERBERG LIMITS		COMPACTION TEST		DIRECT SHEAR		COMPRESSIVE STRENGTH TESTS		CORROSIVITY TESTS				R-VALUE	EXPANSION INDEX	SAND EQUIVALENT (SE)	SPECIFIC GRAVITY
								LL	PI	MAX DD pcf	OPT MC %	C ksf	PHI deg	Qu, ksf	S_u (Cell Prs.) ksf	R	pH	Cl	So_4 (%)				
DH-05	20.0	5	Poorly-graded SAND (SP)	132	108	21	7																
DH-05	30.0	7	Fat CLAY (CH)	115	80	44	94	52	27														
DH-05	40.0	9	Silty Fine SAND (SM)	125	97	29	25																
MW-01	3.0	1	Silty Fine SAND (SM)	121	105	15																	
MW-01	5.5	2	Silty Fine SAND (SM)			29	37																
MW-01	9.0	3	Poorly-graded SAND (SP)	129	109	18	5																
MW-01	19.5	5	Sandy SILT (ML)			26	69	27	4														
MW-01	25.0	6	Sandy SILT (ML)	132	107	23																	
MW-02	2.5	1	Sandy Clayey SILT (ML-CL)	127	111	14	54	25	7														
MW-02	4.5	2	Sandy Clayey SILT (ML-CL)																				
MW-02	8.5	3	Fat CLAY with sand (CH)	102	68	51	82	75	44														
MW-02	14.5	4	Poorly-graded SAND (SP)			23	8																
MW-02	19.5	5	Poorly-graded SAND (SP)	132	113	17																	
MW-02	29.5	7	SILT (ML)	125	97	29																	
MW-03	3.5	2	Silty SAND (SM)	122	112	9	52					0.1	35										
MW-03	9.5	4	Lean CLAY (CL)	157	127	24																	
MW-03	19.5	6	Poorly-graded SAND (SP)	126	108	16	10																
MW-03	29.5	8	Poorly-graded SAND (SP)	133	116	15																	

SUMMARY OF LABORATORY TEST RESULTS
J Street Drain Improvements
Oxnard, California

PLATE B-1b

HDR Engineering, Inc.
Project No. 3161.014



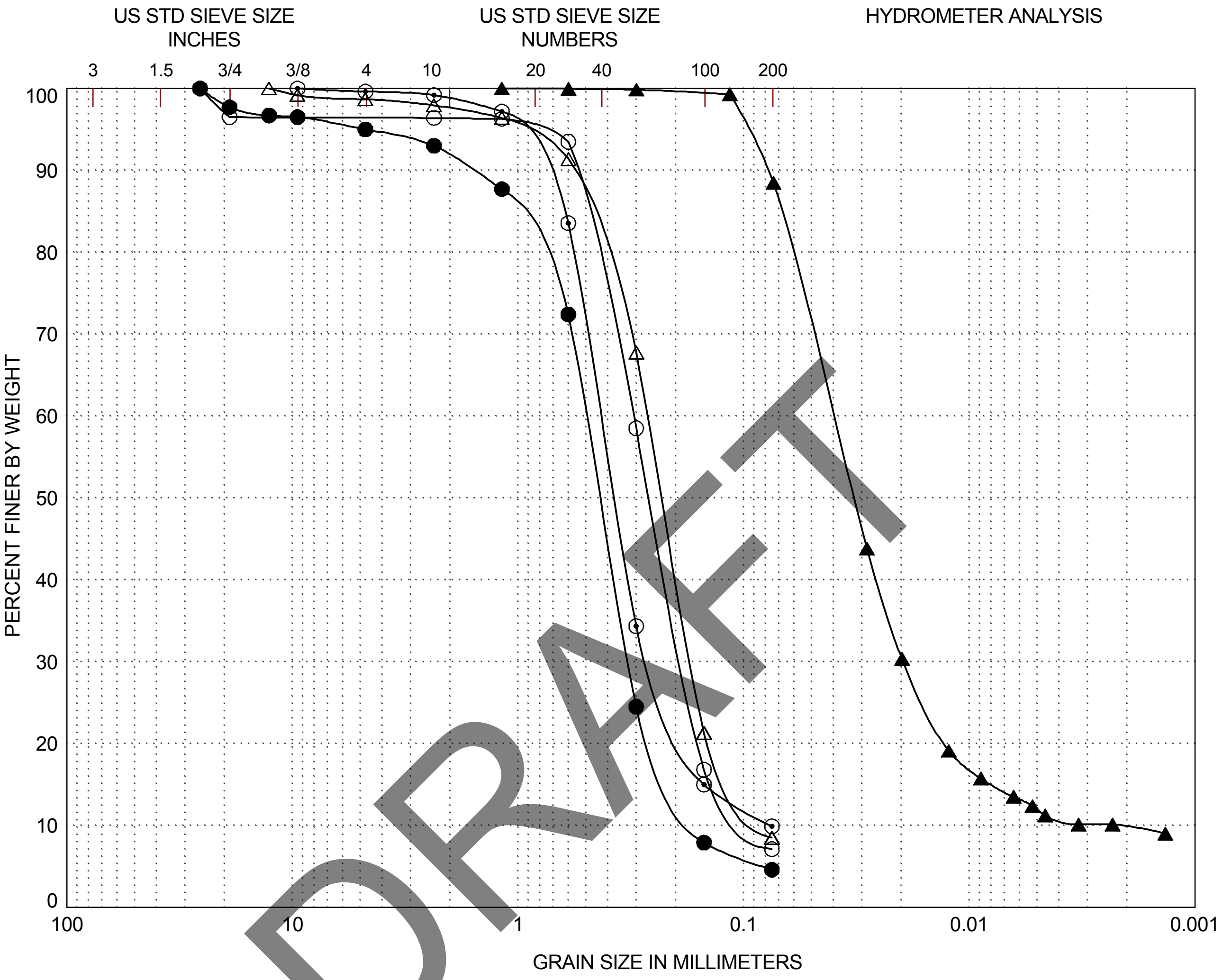
GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

LEGEND			CLASSIFICATION	Cc	Cu
	(location)	(depth,ft)			
○	DH-01	20.0	Poorly-graded SAND (SP)	1.1	3.6
●	DH-02	5.5	Poorly-graded SAND (SP)	1.0	4.1
△	DH-02	9.5	Well-graded SAND with gravel (SW)	1.0	6.4
▲	DH-02	14.5	Well-graded SAND (SW)	1.0	7.0
⊙	DH-02	34.5	Poorly-graded SAND (SP)	1.6	4.0
✣	DH-03	6.5	Poorly-graded SAND (SP)	1.1	3.8
⅄	DH-03	15.0	Poorly-graded SAND (SP)	1.8	5.3
▣	DH-04	9.0	Poorly-graded SAND (SP)	1.2	4.5
⊗	DH-04	19.5	Well-graded SAND (SW)	1.1	6.4
⊕	DH-04	34.5	Lean CLAY with sand (CL)		
□	DH-05	5.0	Poorly-graded SAND (SP)	1.2	3.5
◐	DH-05	11.0	Silty SAND (SM)	2.2	11.0

GRAIN SIZE CURVES
J Street Drain Improvements
Oxnard, California

PLATE B-2a

HDR Engineering, Inc.
Project No. 3161.014

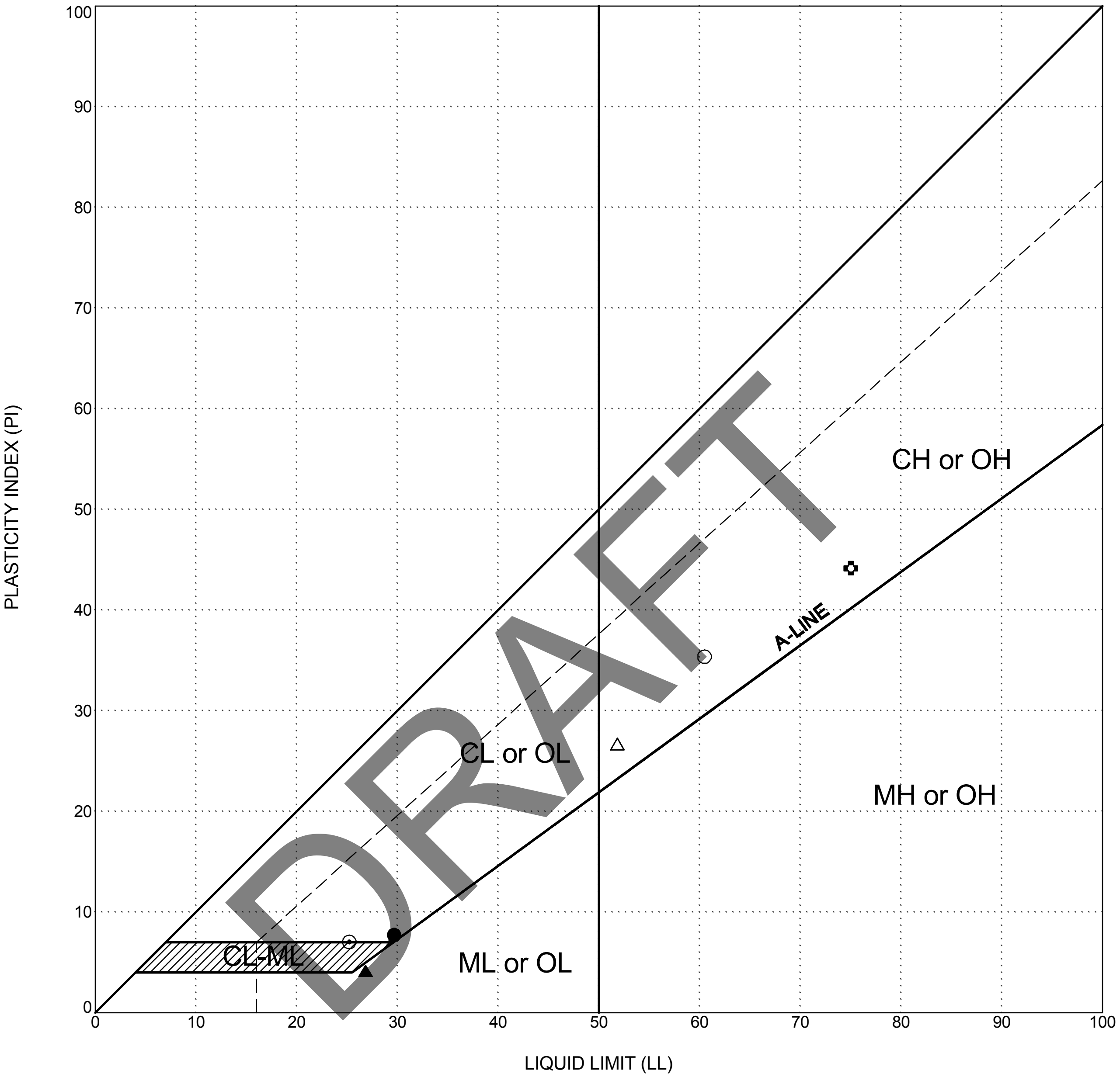


GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

LEGEND			CLASSIFICATION	Cc	Cu
	(location)	(depth,ft)			
○	DH-05	20.0	Poorly-graded SAND (SP)	1.2	3.3
●	MW-01	9.0	Poorly-graded SAND (SP)	1.3	3.1
△	MW-02	14.5	Poorly-graded SAND (SP)	1.3	3.3
▲	MW-02	29.5	SILT (ML)	4.4	18.3
⊙	MW-03	19.5	Poorly-graded SAND (SP)	2.0	5.7

GRAIN SIZE CURVES
J Street Drain Improvements
Oxnard, California

PLATE B-2b

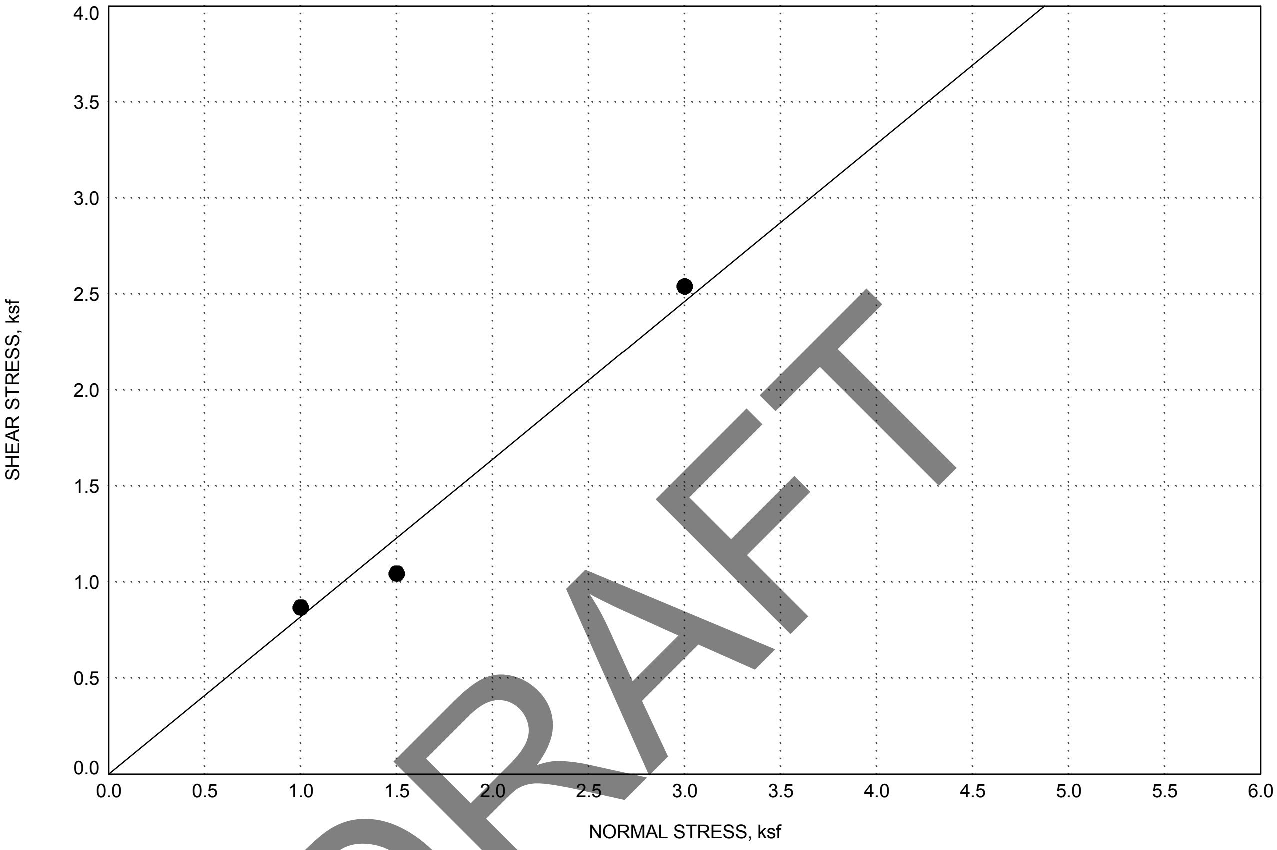


LEGEND			CLASSIFICATION	ATTERBERG LIMITS TEST RESULTS		
	location	depth, ft		LIQUID LIMIT(LL)	PLASTIC LIMIT(PL)	PLASTICITY INDEX (PI)
○	DH-03	24.5	Fat CLAY with sand (CH)	60	25	35
●	DH-04	29.5	Lean CLAY with sand (CL)	30	22	8
△	DH-05	30.0	Fat CLAY (CH)	52	25	27
▲	MW-01	19.5	Sandy SILT (ML)	27	23	4
⊙	MW-02	2.5	Sandy Clayey SILT (ML-CL)	25	18	7
✤	MW-02	8.5	Fat CLAY with sand (CH)	75	31	44

PLASTICITY CHART

J Street Drain Improvements
Oxnard, California

PLATE B-3



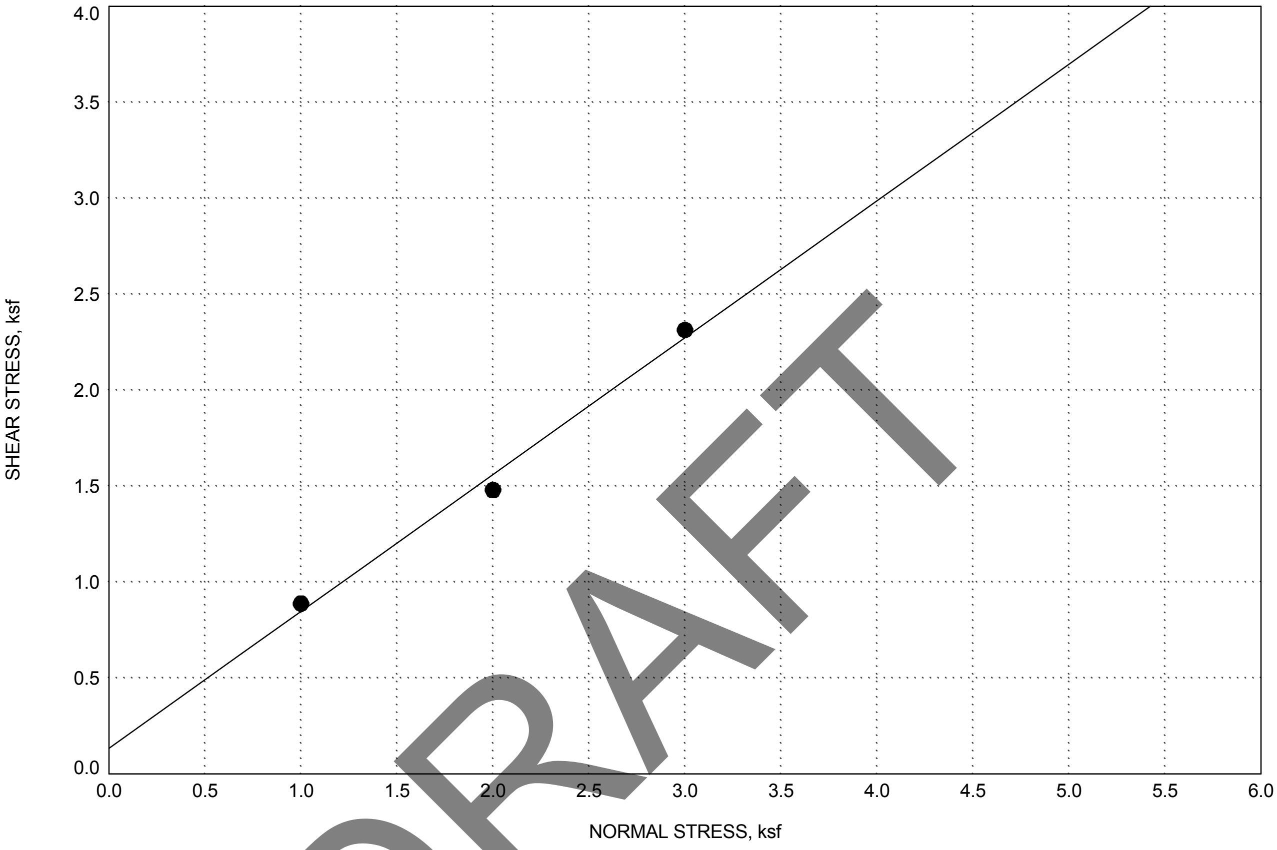
COHESION, ksf	0.0
ANGLE OF INTERNAL FRICTION, deg	39
LOCATION	DH-01
DEPTH, ft	4
MOISTURE CONTENT, %	19
UNIT DRY WEIGHT, pcf	102
MATERIAL DESCRIPTION	SAND (SP)
SAMPLE CONDITION	Ring Sample

DIRECT SHEAR TEST RESULTS

J Street Drain Improvements
Oxnard, California

PLATE B-4a

HDR Engineering, Inc.
Project No. 3161.014

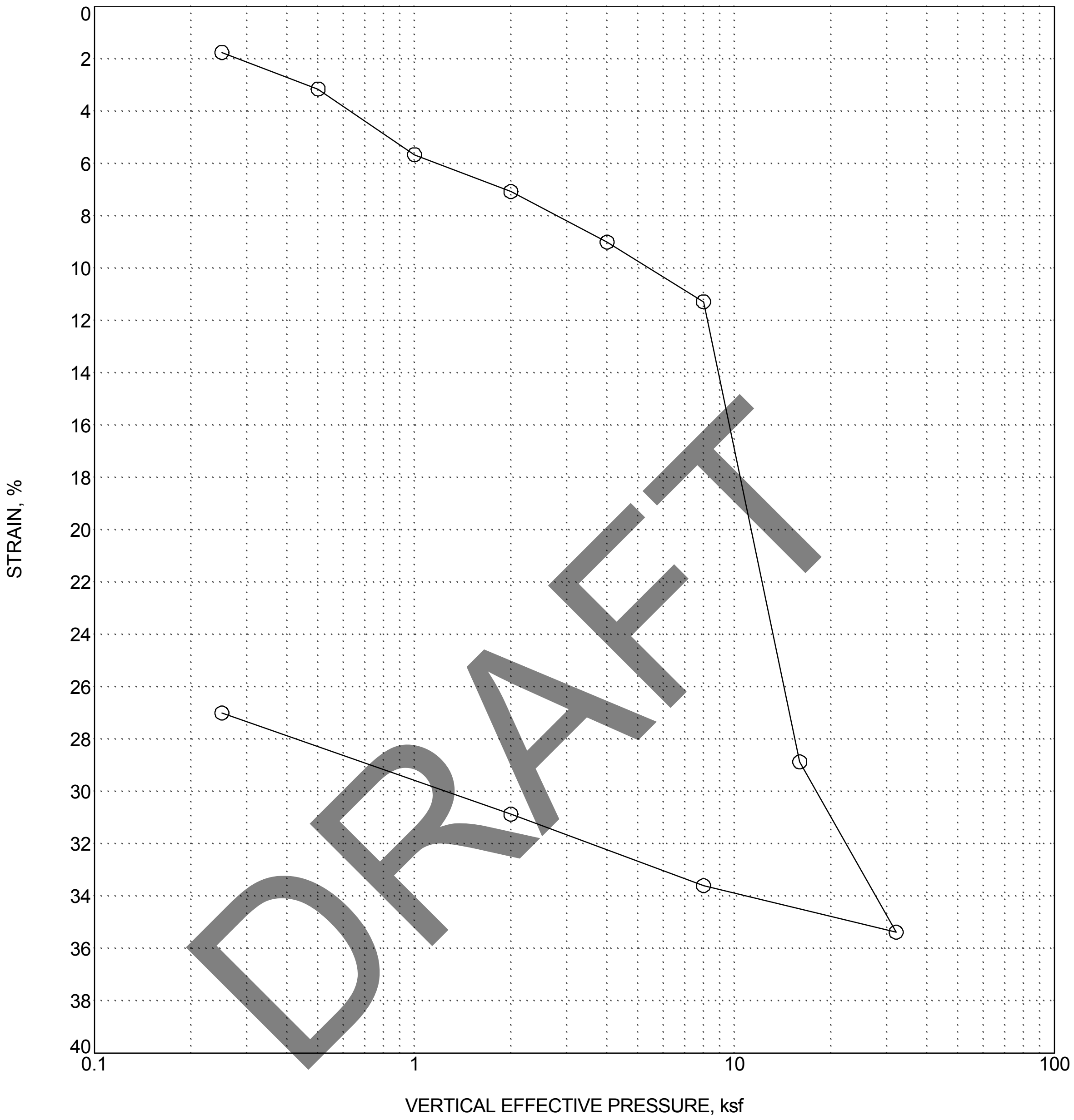


COHESION, ksf	0.1
ANGLE OF INTERNAL FRICTION, deg	35
LOCATION	MW-03
DEPTH, ft	3.5
MOISTURE CONTENT, %	20
UNIT DRY WEIGHT, pcf	108
MATERIAL DESCRIPTION	Silty SAND (SM)
SAMPLE CONDITION	Ring Sample

DIRECT SHEAR TEST RESULTS

J Street Drain Improvements
Oxnard, California

PLATE B-4b



LOCATION	MW-02
DEPTH, ft	8.5
INITIAL MOISTURE CONTENT, %	51
UNIT DRY WEIGHT, pcf	68
MATERIAL DESCRIPTION	Fat CLAY with sand (CH)
SAMPLE CONDITION	

CONSOLIDATION TEST RESULTS

J Street Drain Improvements
Oxnard, California

COOPER TESTING LABORATORY

Corrosivity Test Summary

CTL # 446-051 **Date:** 5/22/2008 **Tested By:** PJ **Checked:** PJ
Client: Fugro **Project:** J Street Drain #3837 **Proj. No:** 3161.014
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	As Received	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
DH-1	-	0-5	-	7893	-	<2	<5	<0.0005	9.0	-	5.2	Brown Silty SAND
DH-3	-	0-5	-	1100	-	10	480	0.0480	8.1	-	12.8	Brown Sandy CLAY w/ Gravel
DH-5	-	0-5	-	1188	-	56	991	0.0991	8.2	-	5.1	Brown Silty SAND

CORROSIVITY TEST SUMMARY

J Street Drain Improvements
Oxnard, California

APPENDIX C
HYDRAULIC CONDUCTIVITY TESTING

APPENDIX C
HYDRAULIC CONDUCTIVITY TESTING

INTRODUCTION

This appendix describes the equipment and test methods employed to evaluate the hydraulic conductivity at two monitoring wells at the project site. The contents of this appendix shall be integrated with the geotechnical engineering study of which it is a part. They shall not be used in whole or in part as a sole source for information or recommendations regarding the subject site.

FIELD STUDY

Approximate hydraulic conductivity along the proposed alignment was evaluated within two of the constructed monitoring wells (piezometers), MW-1 and MW-2, by performing short-term constant rate pumping tests. The methods and materials involved in piezometer installation are summarized in Appendix A with complete as-built diagrams and details. The approximate locations of the piezometers are shown on Plates 1 and 2.

Test Method

ASTM Standards on Ground Water and Vadose Zone Investigations include guidance document D4043 - Standard Guide for Selection of Aquifer-Test Method in Determining of Hydraulic Properties by Well Techniques and is to be used in conjunction with D4050 - Standard Test Method (Field Technique) for Withdrawal and Injection Well Tests for Determining Hydraulic Properties of Aquifer Systems. ASTM D4043 contains a decision tree (flowchart) used to select an appropriate test procedure (attached). Selection of Single-Well Hydraulic Test Methods for Monitoring Wells, also contained in Standards on Ground Water and Vadose Zone Investigations, indicates that when hydraulic conductivity values are above 28 ft/d (209 gpd/ft2), as is the case in the sands and gravels underlying the site, slug testing is not appropriate. Rather, a constant head (injection) test or single-well (constant rate) pumping test is appropriate. Of these two tests, the single-well pumping test was chosen as an appropriate method, mainly because it did not involve introducing water into the aquifer.

Test Setup and Well Development

Short-term constant rate pump tests were performed on piezometers, MW-1 and MW-2, on Thursday, May 8, 2008. First, water levels were measured within each piezometer relative to the top of each traffic-rated vault. Next, each piezometer was instrumented with a submersible electric pump and an In-Situ MiniTroll Professional datalogger, programmed to read and record water level data at regular intervals. After several minutes of recording static water level data with the datalogger, the pump within each well was operated at varied pumping rates for approximately 15 minutes, until the produced water was relatively clear. This was the only development performed within each well.

Pumping Tests

Following a brief period of recovery, the wells were pumped for about 100 minutes at constant rates of approximately 6.4 gallons per minute (pgm) for MW-1 and approximately 3.3 gpm for MW-2. The pumping rates were measured with the use of a calibrated 5-gallon bucket and chronograph. After about 100 minutes of pumping, the pumps were turned off and water level recovery data was collected until no further significant water level recovery was observed.

A summary of the pumping test data is presented below as Table 5 – Summary of Pump Test Data and Hydraulic Conductivity Results. Hydrographs of the entire period of pumping are presented as Plates C-1 and C-2. Hydrographs of the constant rate pumping tests are presented as Plates C-3 and C-4. Hydrographs of the constant rate pumping test recovery are presented as Plates C-5 and C-6.

Table 5 - Summary of Pump Test Data and Hydraulic Conductivity Results

Test Portion	Duration (minutes)	Pumping Rate (gpm)	Drawdown (feet)	Transmissivity (ft^2/d)	Saturated Thickness (ft)	Hydraulic Conductivity (ft/d)
MW-1 Pumping	100	6.4	3.12	1,113	5 (15)	223 (74)
MW-1 Recovery	40	0	3.90	2,271	5 (15)	454 (151)
MW-2 Pumping	85	3.3	0.49	673	10 (15)	67 (45)
MW-2 Recovery	12	0	0.48	4,213	10 (15)	421 (281)

Note: Two values for saturated thickness given; first value is thickness of likely water-bearing layer, value in parentheses is entire saturated thickness. Hydraulic conductivity value in parentheses is an average value.

HYDRAULIC CONDUCTIVITY EVALUATION

Pumping test data were evaluated for transmissivity and hydraulic conductivity in accordance with the Cooper and Jacobs (1946) straight-line approximation method. A summary of the hydraulic conductivity results are presented in Table 5. In both piezometers, the data collected during the pumping tests were preferred over the recovery tests. Therefore, the results of the recovery tests, although presented, are not considered in the calculation of the site hydraulic conductivity. It appears that the pumping test performed within both piezometers resulted in a high enough discharge rate to sufficiently stress the aquifer. Therefore, the calculated values of hydraulic conductivity for the pumping tests presented in Table 5 are generally considered accurate.

Based on the analysis, the approximate value of hydraulic conductivity within MW-1 for the sand layer below 14 feet (total thickness of 5 feet) is approximately 223 feet per day (ft/d).

The average hydraulic conductivity of the entire saturated interval, including a 10 foot thick layer of sandy silt, is approximately 74 ft/d. Within MW-2, the approximate value of hydraulic conductivity for the sand layer (total thickness of 10 feet) is approximately 67 feet per day (ft/d). The average hydraulic conductivity of the entire saturated interval is approximately 45 ft/d.

Based on a literature search, the calculated values of hydraulic conductivity are characteristic of "silty sands" to "clean sands" (Freeze and Cherry, 1979). According to Driscoll (1986), these hydraulic conductivity values are representative of "fine to coarse sand". Fetter (1988) indicates that "well-sorted sands" have hydraulic conductivity values similar to those presented above. The calculated values of hydraulic conductivity reasonably represent the materials described on the drilling logs of the monitoring wells.

M:\Drafting\JOBFILES\2008\3161\3161.014\Drawings\Plate C-1.cdr, 7-1-08

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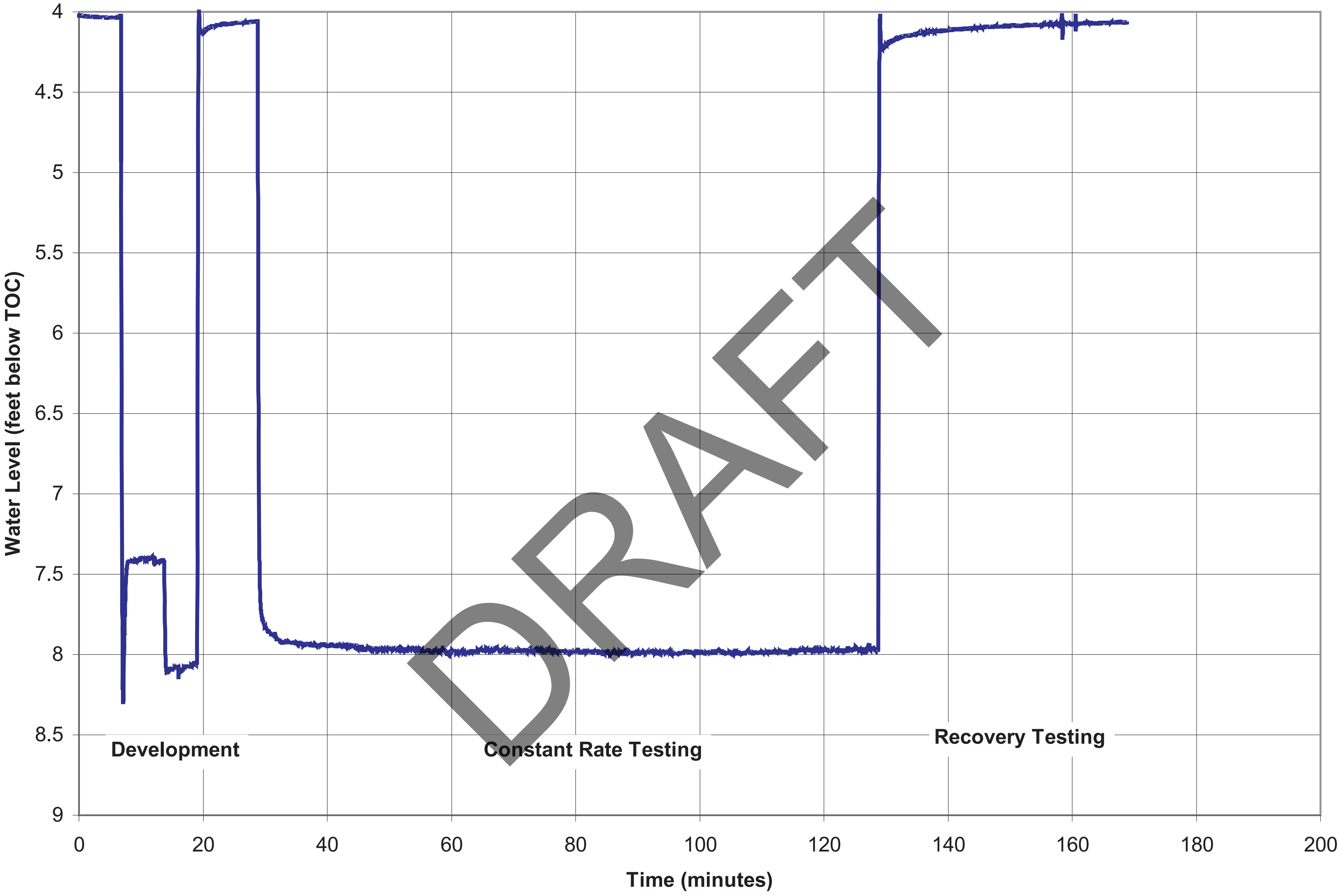

MW-1 PUMPING TEST
(Water Level vs. Time)
J Street Drain Improvements
Oxnard, California

PLATE C-1

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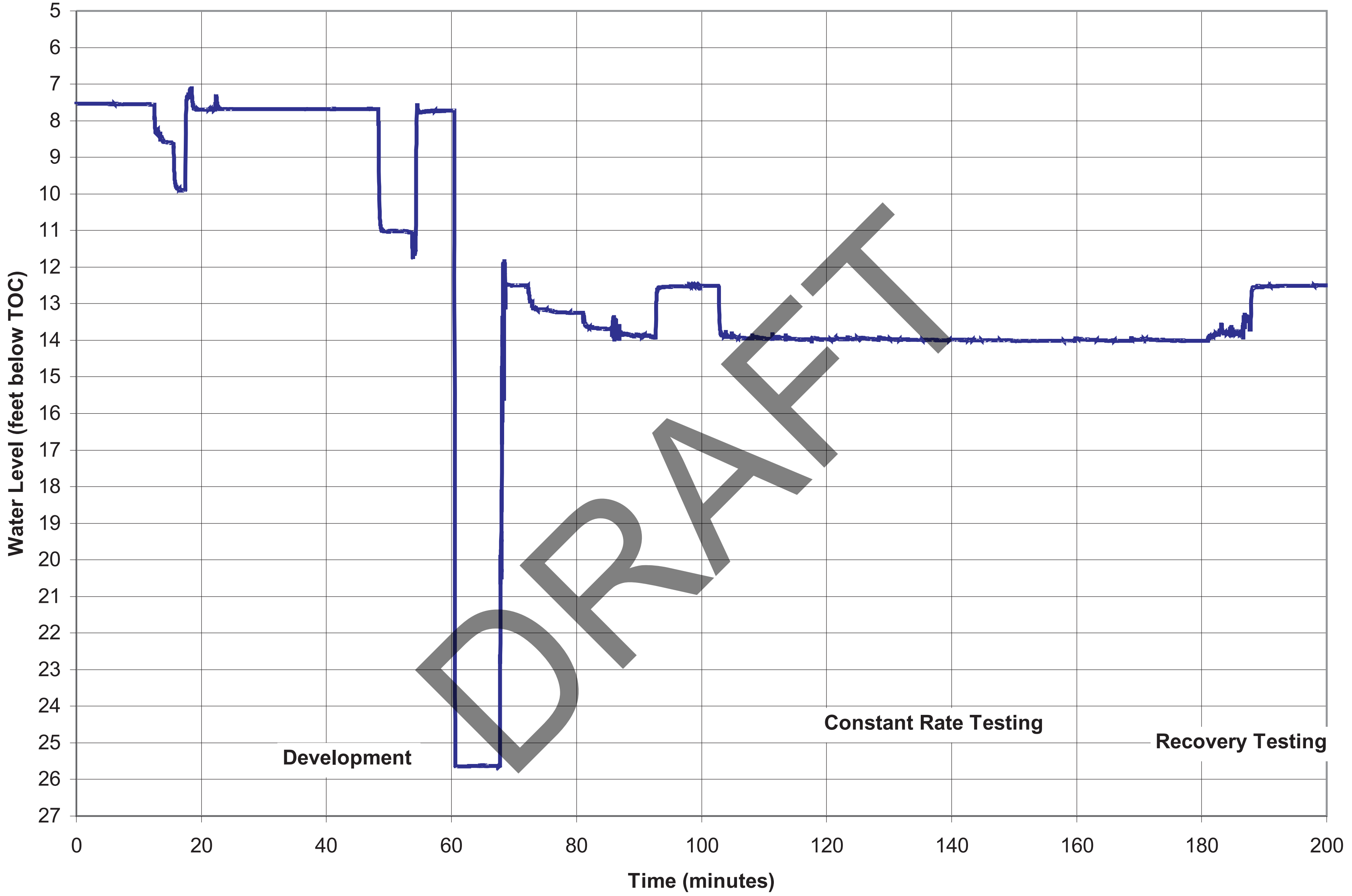

MW-2 PUMPING TEST
(Water Level vs. Time)
J Street Drain Improvements
Oxnard, California

PLATE C-2

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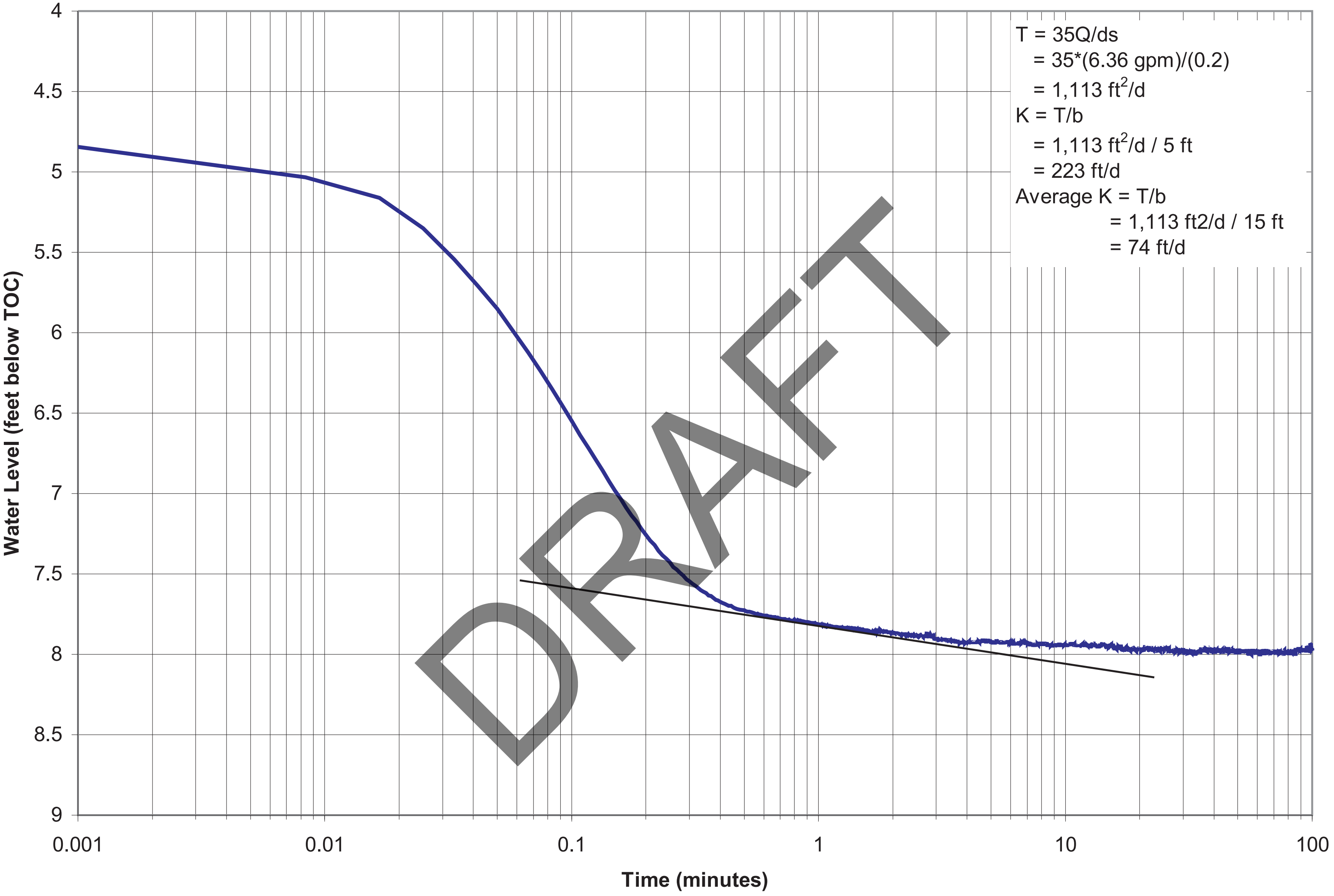
Project No. 3161.014



T = 35Q/ds
= 35*(6.36 gpm)/(0.2)
= 1,113 ft^2/d
K = T/b
= 1,113 ft^2/d / 5 ft
= 223 ft/d
Average K = T/b
= 1,113 ft2/d / 15 ft
= 74 ft/d

PLATE C-3

MW-1 PUMPING TEST
(Water Level vs. Log Time)

J Street Drain Improvements
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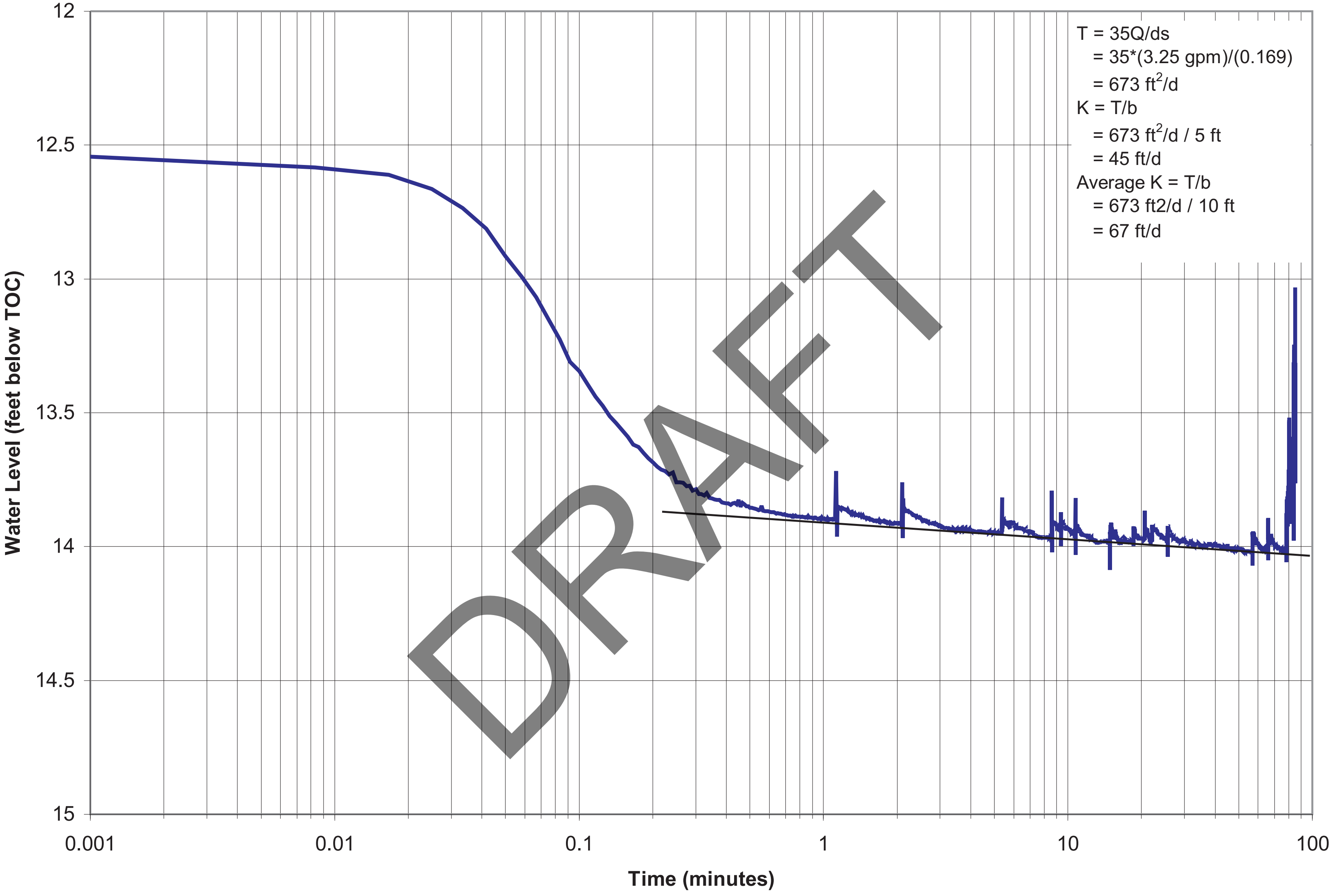

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MW-2 PUMPING TEST
(Water Level vs. Log Time)

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PLATE C-4

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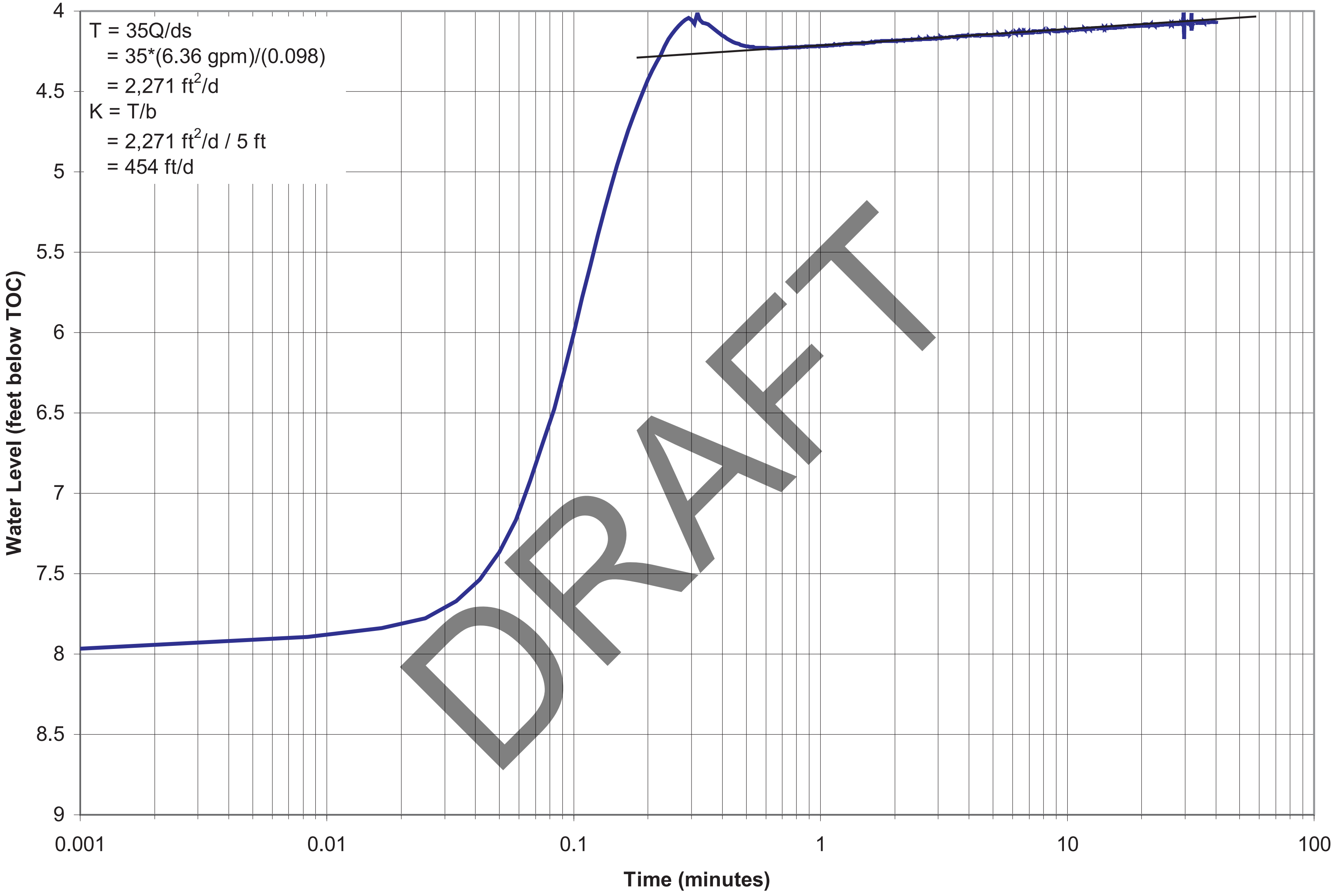

T = 35Q/ds
= 35*(6.36 gpm)/(0.098)
= 2,271 ft^2/d
K = T/b
= 2,271 ft^2/d / 5 ft
= 454 ft/d

MW-1 RECOVERY TEST
(Water Level vs. Log Time)

J Street Drain Improvements
Oxnard, California

PLATE C-5

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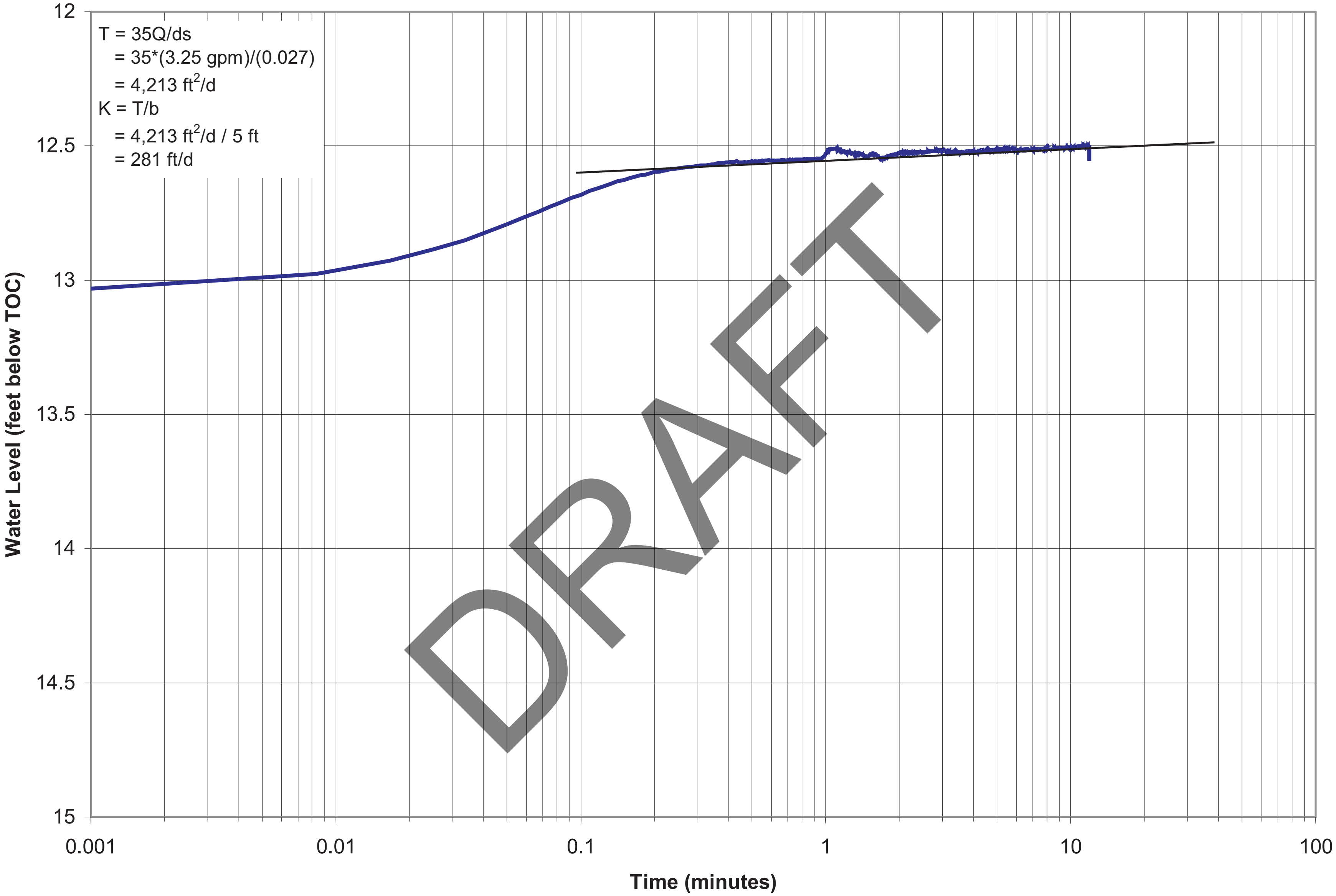

$T = 35Q/ds$
$= 35*(3.25 \text{ gpm})/(0.027)$
$= 4{,}213 \text{ ft}^2/\text{d}$
$K = T/b$
$= 4{,}213 \text{ ft}^2/\text{d} / 5 \text{ ft}$
$= 281 \text{ ft/d}$

MW-2 RECOVERY TEST
(Water Level vs. Log Time)

J Street Drain Improvements
Oxnard, California

PLATE C-6