Wallace Kuhl & ASSOCIATES

Geotechnical Engineering Report

L STREET MIXED USE

WKA No. 9955.01

January 27, 2014

Prepared For:

LVP Revocable Trust

2020 L Street, 5th Floor

Sacramento, California 95811

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INTRODUCTION

We have completed a geotechnical engineering investigation for the proposed mixed use development on the north side of L Street between 20th and 21st Streets in Sacramento, California. The purposes of our work have been to explore the existing site, soil and groundwater conditions beneath the proposed improvement areas and to provide geotechnical engineering conclusions and recommendations for the design and construction of the proposed improvements. This report represents the results of our work.

Work Scope

Our scope of work has included the following tasks:

1. site reconnaissance;
2. review of aerial photographs and available historical groundwater contour maps;
3. subsurface exploration, including the drilling and sampling of two (2) test borings to maximum depths of approximately 50 to 51 feet below the existing ground surface;
4. bulk sampling of near-surface soils;
5. laboratory testing of selected soil samples;
6. engineering analyses, and;
7. preparation of this report.

Our evaluation was performed in general accordance with our *Geotechnical Engineering Services Proposal* dated September 10, 2013.

Figures and Attachments

A Vicinity Map showing the location of the site is included as Figure 1. Figure 2 shows the approximate locations of the borings relative to existing site features. The Logs of Soil Borings are presented as Figures 3 and 4. An explanation of the symbols and classification system used on the logs appears on Figure 5. Appendix A contains general information regarding the field investigation, descriptions of the field exploration and laboratory testing programs, and the results of laboratory tests that do not appear on the Logs of Soil Borings.

Proposed Development

We understand the project will consist of the demolition of the existing parking structure and the design and construction of a new mixed-use development consisting of one- to two-stories of below-grade parking and seven stories of mixed-use development. The above-grade portion of the development will include additional parking levels, retail space, and apartments. Associated development is anticipated to consist of exterior concrete flatwork and underground utilities.

FINDINGS

Site Description

The project site is located on the north side of L Street between 20th and 21st Streets in Sacramento, California. The site is currently occupied by a two-story, at-grade parking structure with an adjoining office building and asphalt concrete surface parking. Associated development includes concrete flatwork and landscaping.

Our review of historic aerial photographs obtained for the *Phase 1 Environmental Site Assessment* (ESA) dated September 23, 2013 prepared by our firm indicates that the site was developed with residences from at least 1895 to at least 1965 and has been developed with a parking structure since at least 1965. Several structures are noted east and west of the existing parking structure on aerial photographs from until at least 1981. The structures are not seen in an aerial photograph from 1993 and were presumably demolished prior to 1993. Our review of the Phase 1 ESA indicates the demolished buildings were residential from at least 1915 to at least 1980 and were used for commercial purposes since at least 1991.

Subsurface Soil Conditions

Two (2) exploratory borings were performed on November 23, 2013 at the approximate locations indicated on Figure 2. The soil conditions at the borings generally consist of about 26 to 28 feet of interbedded sand and silt layers overlying relative dense gravels extending about 42 to 44 feet below the existing ground surface. The gravels are underlain by relatively dense silts extending to the explored 50 to 51 foot depths of the borings.

At the completion of our drilling activities, the test borings were grouted to the surface with a slurry of neat cement and water, as required by the permit issued by the County of Sacramento Environmental Management Department.

For soil conditions at the specific boring locations, please refer to the boring logs contained on Figures 3 and 4.

Groundwater

Groundwater was encountered about 18 feet below the ground surface at the boring locations during and immediately after the drilling operations. Based on our experience in the area, groundwater is anticipated to be as high as about 15 feet below the existing ground surface at the site.

CONCLUSIONS

Seismic Code Parameters – 2013 CBC/ASCE 7-10

We understand the design of the structures will be performed using the 2013 CBC. The 2013 edition of the CBC references American Society of Civil Engineers (ASCE) Standard 7-10 for seismic design. The following seismic parameters were determined based on the site latitude and longitude using the public domain computer program developed by the United States Geological Survey (USGS).

2013 CBC/ASCE 7-10 Seismic Design Parameters

Latitude: 38.5746° N Longitude: 121.4801° W	ASCE 7-10 Table/Figure	2013 CBC Table/Figure	Factor/ Coefficient	Value
Short-Period MCE at 0.2s	Figure 22-1	Figure 1613.3.1(1)	S_S	0.667 g
1.0s Period MCE	Figure 22-2	Figure 1613.3.1(2)	S_1	0.291 g
Soil Class	Table 20.3-1	Section 1613.3.2	Site Class	D*
Site Coefficient	Table 11.4-1	Table 1613.3.3(1)	F_a	1.266
Site Coefficient	Table 11.4-2	Table 1613.3.3(2)	F_v	1.817
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equation 16-37	S_{MS}	0.845 g
	Equation 11.4-2	Equation 16-38	S_{M1}	0.529 g
Design Spectral Acceleration Parameters	Equation 11.4-3	Equation 16-39	S_{DS}	0.563 g
	Equation 11.4-4	Equation 16-40	S_{D1}	0.3531 g

Latitude: 38.5746° N Longitude: 121.4801° W	ASCE 7-10 Table/Figure	2013 CBC Table/Figure	Factor/ Coefficient	Value
Seismic Design Category	Table 11.6-1	Section 1613.3.5(1)	Occupancy I to IV	D
	Table 11.6-2	Section 1613.3.5(2)	Occupancy I to IV	D

* Assumes the structure is supported on foundation system situated on or extending to the relatively dense gravels below the site.

Liquefaction Potential

Liquefaction is a soil strength loss phenomenon that typically occurs in loose, saturated cohesionless sands as a result of strong ground shaking during earthquakes. The potential for liquefaction at a site is usually determined based on the results of a subsurface soil investigation and the groundwater conditions beneath the site. Hazards to structures associated with liquefaction include shallow and deep foundation bearing capacity failure, lateral spreading of soil, and differential settlement of soils below foundations, all of which can contribute to structural damage or collapse.

The results of our subsurface soil exploration at the site indicate the underlying soils generally consist of about 26 to 28 feet of relatively loose sandy silts overlying relatively dense gravels extending to depths of about 42 to 44 feet below the existing ground surface. Historical high groundwater is indicated to be about 15 feet below the existing ground surface. Based on the soil and groundwater conditions at the site, a liquefaction analysis to determine factors of safety against liquefaction was performed.

Liquefaction Analysis and Results

We performed a liquefaction analysis of data obtained from the blow counts measured in the borings performed for this investigation. The borings were analyzed using LiqIT (version 4.7) and the liquefaction analyses were performed utilizing the NCERR methodology. A design static groundwater level of approximately 15 feet below existing ground surface was used in our analysis based on our review of historic groundwater levels at the site. A peak ground acceleration (PGA) of 0.227 g was used in the liquefaction analysis based on Equation 11.8-1 of ASCE 7-10. A mode magnitude earthquake of 6.6 was used for this analysis using the 2008 USGS National Seismic Hazard Mapping Project (NSHMP) Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation web site.

The results of the liquefaction analyses indicate factors of safety against liquefaction below 1.3 and that the majority of the soil profile may be susceptible to liquefaction.

Seismically Induced Settlement

Post-liquefaction settlement calculations within LiqIT are performed using the methodology of Ishihara and Yoshimine (1992).

Given the results of our analysis performed for this investigation, the worst-case estimate of total and differential post-liquefaction settlement is calculated to be about six (6) inches total seismic induced settlement. Based on the soil conditions encountered at the borings and our previous work at the site, we anticipate about three (3) inches of differential settlement across 50 feet, or the shortest dimension of the structure, whichever is less. These estimates of post-liquefaction seismic settlements represent free-field ground settlement, not settlement of the structures.

Bearing Capacity

Based upon our field and laboratory testing and the results of our liquefaction analysis, it is our opinion that undisturbed native soils overlying the gravel layer are not capable of supporting the planned structures and associated improvements unless the structures are supported on an alternative foundation system, such as shallow foundations supported on an improved subgrade (i.e., Geopier® rammed aggregate piers [RAPs]) or a deep foundation system consisting of driven, precast concrete piles (driven piles); drilled, auger cast-in-place piles; or drilled cast-in-place reinforced concrete piers. However, we anticipate noise and vibrations associated with the construction of driven piles at the site will exceed those typically tolerated for projects within close proximity to existing structures such as those adjacent to the site. Therefore, driven piles will not be considered for this project at this time due to noise pollution, disturbances due to vibrations, and other factors associated with construction of driven piles.

If the proposed structure will extend about 25 feet below the ground surface, consideration may be given to supporting the structure on shallow foundations supported on the relatively dense gravel layer. The gravel layer was encountered about 24 to 26 feet below the existing ground surface at the boring locations.

The selection of the most appropriate foundation system or systems will depend on the actual loads and configurations (i.e., above grade, below-grade, partially below-grade, etc.) of the structures, the acceptable amount of settlement for the structure, and the construction constraints (i.e., vibrations, noise, equipment access, etc.). A discussion of each foundation type is provided as follows.

Specific recommendations for the various foundation systems are provided in the Foundations section of this report.

Conventional Shallow Foundations

Conventional foundations extending to the top of the relatively dense gravel layer encountered about 26 to 28 feet below the ground surface at the boring locations could be used to support the parking garage provided the recommendations of this report are carefully followed. However, conventional shallow foundations at the site will require dewatering during construction and should be accounted for in the construction schedule and budget.

We anticipate total settlements on the order of one inch and differential settlements on the order of ½-inch for conventional foundations. Minimizing settlement between the below grade portions and at-grade portions of the proposed structure will be a significant concern. Deepening foundations beneath the at-grade portions of the proposed structure will help mitigate differential settlements of the proposed structure. Foundations should be at or near the same elevation.

Shallow Foundations Supported on Geopier® RAPs

Based on the available information, we conclude that shallow foundations supported on an improved subgrade consisting of Geopier® RAPs would be appropriate for support of the proposed improvements. The Geopier® system uses a drilled shaft backfilled with compacted aggregate base to improve subgrade stability and reduce settlements within the treated area. The Geopier® system should be designed by a professional engineer in the State of California that is qualified and experienced in Geopier® rammed aggregate pier design.

Drilled Auger Cast-in-Place Piles

Based on the soil conditions encountered at the site, deep foundations consisting of auger cast-in-place (ACIP) piles extending into the relatively dense gravel layer are considered feasible at the site. Auger cast-in-place piles have been used as an alternative to driven piling to reduce detrimental vibration, noise, and other problems associated with driving piles, and can achieve similar bearing, uplift, and lateral resistance of the driven piles.

We anticipate total settlements on the order of ½-inch and differential settlements on the order of ¼-inch for ACIP pile foundations. A contingency plan for loading and off-hauling soil cuttings from the ACIP should be considered in the construction plans and schedule.

Drilled, Cast-in-Place, Reinforced Concrete Piers

Drilled, cast-in-place, reinforced concrete piers (drilled piers) could be used to support the structure. Drilled piers will likely extend below the groundwater table during construction and will require wet construction techniques (i.e., casing and/or drilling slurry). We anticipate drilled piers will extend to the top of the gravel layer encountered about 26 to 28 feet below the ground surface at the boring locations.

We anticipate total settlements on the order of ½-inch and differential settlements of ¼-inch. The use of drilled piers also would provide increased uplift and lateral resistance for the structure.

The construction costs, plan, and schedule should include loading and off-hauling soil cuttings from the drilled pier construction.

<u>Soil Expansion Potential</u>

The near-surface soils encountered at the borings generally consist of granular silts and sands, that are not considered expansive. Therefore, special reinforcement of foundations and floor slabs, or special moisture conditioning during site grading to resist or control soil expansion pressures, are <u>not</u> considered necessary on this project.

<u>Pavement Subgrade Quality</u>

Laboratory testing of bulk samples obtained at the site indicates the near-surface soils are relatively good quality materials for support of asphalt concrete and concrete pavements. A Resistance value (R-value) of 61 was obtained on a composite bulk soil samples obtained from the upper three feet of soil at boring location D2. The results of the R-value testing are included on Figure A3 attached.

<u>Material Suitability</u>

The existing on-site materials are considered suitable for use as engineered fill, provided they are free of significant quantities of organics, rubble and deleterious debris, and at a suitable moisture content to achieve the recommended compaction.

Soils beneath existing pavement and slab areas and irrigated areas will likely be at an elevated moisture content regardless of the time of construction and will require drying before compaction or use as fill.

Existing pavements and flatwork (asphalt concrete and concrete) within areas to be demolished, if any, may be broken up and pulverized for use as fill. Asphalt and Portland cement concrete rubble may be used as fill provided it is processed into fragments less than three inches in largest dimension, is mixed with soil to form a compactable mixture, and is approved by the Owner.

The existing aggregate base encountered below the asphalt concrete and concrete surfaces is considered suitable for reuse as engineered fill. Consideration may also be given to reusing the existing aggregate base as aggregate base or subbase. However, additional laboratory testing would be required to verify the material meets the requirements for Caltrans Class 2 aggregate base or subbase.

Excavation Conditions

Based on the information obtained during the field exploration and our local experience, we anticipate the soils at the site will be readily excavatable with conventional earthmoving and trenching equipment. However, larger equipment may be required to remove existing below-grade structures at the site from previous developments and the existing structures (e.g., previous foundations, concrete slabs, etc.). Based on the results of our subsurface exploration, the soils across the site may be classified as Type B soils in accordance with the Occupational Safety and Health (OSHA) classification system.

In general, we anticipate the on-site soils will likely remain stable at near-vertical inclinations without significant caving for relatively short periods (i.e., less than one day) during utility and foundation construction. However, excavations extending into saturated and/or disturbed soils will likely require excavation bracing or shoring to control sloughing and caving for utilities and casing will be required for RAP and/or drilled pier excavations. Excavations deeper than five feet should be sloped or braced in accordance with current OSHA regulations.

Temporarily sloped excavations should be constructed no steeper than a one horizontal to one vertical (1:1) inclination. Temporary slopes likely will stand at this inclination for the short-term duration of construction, provided significant pockets of loose and/or saturated granular soils are not encountered that could slough into excavations.

The contractor must provide a safely sloped excavation or an adequately constructed and braced shoring system in accordance with federal, state and local safety regulations for individuals working in an excavation that may expose them to the danger of moving ground. If material is stored or heavy equipment is operated near an excavation, stronger shoring must be used to resist the extra pressure due to the superimposed loads.

Groundwater

Based on our subsurface exploration and review of groundwater information in the vicinity of the site, a permanent groundwater level of about 15 feet should be used in design of the proposed structure. The permanent groundwater table should not be a significant factor in site development for excavations less than about 15 feet below the existing ground surface. However, it is likely that perched groundwater may be encountered in excavations from rainfall, surface run-off, irrigation, or seepage from perched groundwater sources, especially if construction begins in the winter and early spring months.

For excavations extending less than about 15 feet below the existing ground surface standard sump pit and pumping procedures should be adequate to control localized groundwater. If Geopier® RAPs, ACIP piles, or drilled piers are used for foundation support, the RAP or pile/pier contractor should provide proper equipment and materials to handle the anticipated groundwater depths.

Dewatering of excavations deeper than about 15 feet below the existing ground surface should be anticipated, although the groundwater elevation will vary depending on seasonal rainfall. Temporary dewatering will be necessary to maintain a relatively dry excavation and to limit disturbances to the subgrade at the bottom of the excavation. The groundwater should be temporarily lowered to at least two feet below the bottom of excavations. The spacing interval(s) and depth for dewatering operations will depend on the rate and volume of groundwater flow experienced and should be determined in the field by the dewatering contractor. Note that the dewatering design should take into account the effect dewatering operations will have on the adjacent improvements.

Groundwater levels should be expected to fluctuate throughout the year based on variations in precipitation, temperature, evaporation, run-off, and other factors. The groundwater levels discussed herein, and indicated on the boring logs, represent the conditions at the time the measurements were obtained. The actual groundwater levels at the time of construction may vary.

Seasonal Water

Infiltrating surface run-off water from seasonal moisture during the winter and spring months will create saturated surface soil conditions. It is probable that grading operations attempted following the onset of winter rains and prior to prolonged drying periods will be hampered by high soil moisture contents. Such soils, intended for use as engineered fill, will require a prolonged period of dry weather and aeration or chemical treatment to reach a moisture content suitable for proper compaction.

In addition, soils located beneath existing pavements, slabs, and flatwork, will likely be at elevated moisture contents regardless of the time of year of construction and also require drying. Wet soils should be anticipated and considered in the construction schedule for this project.

Preliminary Soil Corrosion Potential

A sample of near-surface soil was submitted to Sunland Analytical Lab for testing to determine pH, chloride and sulfate concentrations, and resistivity to help evaluate the potential for corrosive attack upon buried structures. Results of the soil corrosivity tests are summarized below; copies of the test results are attached as Figure A4.

SUMMARY OF CORROSION TEST RESULTS						
Sample Location	Test Depth (feet)	USCS Soil Type	pH	Chloride Content (ppm)	Sulfate Content (ppm)	Resistivity (ohm-cm)
D1	1 to 3	SM	8.02	219.3	323.8	860

The California Department of Transportation Corrosion Technology Section, Office of Materials and Foundations, *Corrosion Guidelines Version 1.0, September 2003*, considers a site to be corrosive to foundation elements if one or more of the following conditions exists for the representative soil and/or water samples taken: has a chloride concentration greater than or equal to 500 ppm, sulfate concentration greater than or equal to 2000 ppm, or the pH is 5.5 or less. The corrosivity test results suggest that the site soils are not highly corrosive to exposed reinforced concrete. The low resistivity may indicate an increased potential for corrosion of buried metal. Table 4.3.1 – *Requirement for Concrete Exposed to Sulfate-Containing Solutions*, American Concrete Institute (ACI) 318, Section 4.3, as referenced in section 1904A.3 of the 2007 CBC, indicates the sulfate exposure for the samples tested is *Negligible*. Ordinary Type I-II Portland cement is considered suitable for use on this project, assuming a minimum concrete cover is maintained over the reinforcement.

Wallace-Kuhl & Associates are not corrosion engineers. Therefore, if it is desired to further define the soil corrosion potential at the site a corrosion engineer should be consulted.

RECOMMENDATIONS

General

The recommendations presented below are appropriate for typical construction in the late spring through fall months. The on-site soils likely will be saturated by rainfall in the winter and early spring months, and will not be compactable without drying by aeration or chemical treatment to dry the soils. Should the construction schedule require work during wet conditions, additional recommendations can be provided, as conditions dictate.

Soils under existing pavements or slabs and irrigated areas will be wet regardless of the time of year of construction.

Site preparation should be accomplished in accordance with the provisions of this report and the appended guide specifications. A representative of the Geotechnical Engineer should be present during site grading to evaluate compliance with our recommendations and the guide specifications. The Geotechnical Engineer of Record referenced herein should be considered the Geotechnical Engineer that is retained to provide geotechnical engineering observation and testing services during construction.

Site Preparation

Proposed structural areas of the site should be cleared of existing structures, pavements, flatwork, below-grade structures, vegetation, debris, and other deleterious materials to expose undisturbed native soils or relatively dense existing fill as determined by our representative. Where practical, the clearing should extend a minimum of five feet beyond the limits of the structural areas of the site.

Existing underground utilities within the proposed structural areas should be completely removed and/or relocated as necessary. Utilities to be abandoned outside the structural areas should be removed or properly plugged (i.e., fully grouted provided the abandoned utility is situated at least 2½ feet below the final subgrade level to reduce the potential for localized "hard spots"). All trees/large brush designated for removal should include the entire rootball and roots ½-inch or larger in size. Depressions resulting from removal of underground structures (e.g. foundations, utilities, etc.) should be cleaned of loose soil and properly backfilled in accordance with the recommendations of this report.

The existing pavements and flatwork (asphalt concrete and concrete) that are not incorporated into the new design should be broken up and removed from the site. Pulverized asphalt and Portland cement concrete rubble may be used as fill below the structures and pavements provided they are processed into fragments less than three inches in largest dimension and mixed with soil to form a compactable mixture.

Surface vegetation and organic soils should be removed from the construction areas by stripping. Strippings should be hauled off-site or placed in landscape areas a minimum of five feet from proposed structural areas of the site (e.g., buildings, pavements, sidewalks, etc.).

Subgrade Preparation

Following the site clearing operations, surfaces to receive fill and at-grade areas should be scarified to a depth of at least 12 inches, moisture conditioned to at least t the optimum moisture content, and be compacted to at least 90 percent relative compaction. Relative compaction should be based on the maximum dry density as determined in accordance with the American Society of Testing and Materials (ASTM) D1557 Test Method.

Soils beneath existing pavement and slab areas and irrigated areas will likely be at an elevated moisture content regardless of the time of construction and will require drying before compaction or use as fill.

Compaction operations should be performed in the presence of the Geotechnical Engineer's representative who will evaluate the performance of the subgrade under compactive load and identify loose or unstable soils that could require additional subgrade preparation.

Engineered Fill Construction

Any fill placed within the construction area should be an approved material, free of significant quantities of organics, oversized rubble, or other deleterious materials. The fill should be spread in level layers not exceeding nine inches in loose thickness and compacted to a minimum of 90 percent of the maximum dry density. Maximum dry densities shall be determined in accordance with ASTM D1557.

Engineered fill should be moisture conditioned to at least the optimum moisture content and maintained in that condition.

The on-site soils encountered at the boring locations are considered suitable for use as engineered fill provided they are free of rubble and organic concentrations and are at a compactable moisture content. Imported fill should be an approved compactable granular material, have an Expansion Index of 20 or less, a Resistance value of at least 30 when used within the upper three feet of pavement subgrades, and be free of particles larger than three inches in maximum dimension. The contractor also should supply appropriate documentation for imported fill materials indicating the materials are free of known contamination and have corrosion characteristics within acceptable limits. Our firm must approve import material <u>before</u> being transported to the project site.

The upper six inches of pavement subgrade should be moisture conditioned to at least the optimum moisture content and compacted to no less than 95 percent relative compaction, regardless of whether final subgrade is achieved by excavation, filling or left at existing grade. Final pavement subgrade processing and compaction should be performed after completion of underground utilities and must be stable under construction traffic prior to aggregate base placement.

Permanent excavation and fill slopes should be constructed no steeper than two horizontal to one vertical (2:1), and should be vegetated as soon as practical following grading to minimize erosion. Slopes should be over-built and cutback to design grades and inclinations.

Site preparation should be accomplished in accordance with the recommendations of this report. We recommend the Geotechnical Engineer's representative be present during site preparation and all grading operations to observe and test the fill to verify compliance with the recommendations of this report and the job specifications.

<u>Utility Trench Backfill</u>

Bedding and initial backfill for utility construction should conform with the pipe manufacturer's recommendations and applicable sections of the governing agency standards. General trench backfill should consist of engineered fill backfilled in maximum nine-inch thick loose lifts with each liftcompacted to at least 90 percent of the maximum dry density as determined by ASTM D1557. Utility trench backfill within the upper six inches of the final subgrade within pavement areas should be compacted to at least 95 percent of the maximum dry density.

We recommend that all underground utility trenches aligned nearly parallel with existing or new foundations be at least five feet from the foundations, wherever possible. If this is not practical, the trenches should not encroach on a zone extending at a one horizontal to one vertical (1:1) inclination below the foundations.

It is likely that materials excavated from trenches will be at elevated moisture contents and will require significant aeration or a period of drying to reach a compactable moisture content. We recommend bid documents contain a unit price for the removal and drying of saturated soils, or replacement with approved import soils.

Foundation Design Alternatives

We recommend that our office be given the opportunity to review final grading plans, foundation plans and specifications to determine if the intent of our recommendations has been properly implemented into those documents.

The proposed structure may be supported upon continuous and/or isolated spread foundations bearing on the relatively dense gravels, on a Geopier® RAP improved subgrade, or a deep foundation system consisting of drilled ACIP piles or drilled piers. Alternative foundations may be considered at the site and will be evaluated on a case-by-case basis.

Recommendations for each type of foundation system have been provided. Combination foundation systems (i.e. shallow foundations used with deep foundations) may be acceptable; however, the structure must be designed to accommodate some differential settlement due to the varying support characteristics of the foundations and elastic properties of various bearing strata. The intent of this recommendation is to minimize differential settlement between the two foundation types.

Our recommendations for shallow spread foundations, drilled ACIP piles, and driven piles are provided in the following sections. Preliminary recommendations for shallow foundations supported on a Geopier® RAP improved subgrade are also provided.

Shallow Spread Foundations (If Structure Contains Basement)

If the proposed structure will contain a basement, the structure may be supported upon continuous and/or isolated spread foundations extending to and the relatively dense gravel layer which was encountered about 26 to 28 feet below the ground surface at the boring locations. Shallow foundations should be embedded at least two (2) feet below lowest adjacent soil grade. Lowest adjacent soil grade is defined as the surface upon which the first floor slab is placed, or the exterior grade, whichever is lower. Continuous foundations should maintain a minimum width of 18 inches and isolated spread foundations should be at least 36 inches in plan dimension. Foundations so established may be sized for maximum allowable soil bearing pressures of 4000 pounds per square foot (psf) for dead plus live loads, with a 1/3 increase for total loads including the short-term effects of wind or seismic forces. The weight of the foundation concrete extending below lowest adjacent soil grade may be disregarded in sizing computations.

Increased bearing capacity can be achieved by increasing the embedment depth of the foundations into the relatively dense gravels. For every additional foot of embedment below the lowest adjacent soil grade into the relatively dense gravels, the allowable bearing capacity may be increased by 1500 psf for dead plus live loads with a 1/3 increase for short-term effects of wind or seismic forces. The allowable dead plus live load capacity may be increased to a maximum of 7500 psf at an embedment depth of five feet below soil grade.

Continuous foundations should be reinforced with a <u>minimum</u> of two No. 4 reinforcing bars, placed one each near the top and bottom, to provide structural continuity and to allow the foundations the ability to span isolated soil irregularities. The structural engineer should evaluate the need for additional reinforcement based on anticipated structural loads.

Lateral resistance of foundations may be computed using an allowable friction factor of 0.30, which may be multiplied by the vertical load on the foundation. Additional lateral resistance may be assumed to develop against the vertical face of the foundations and may be computed using a "passive" lateral earth pressure equal to an equivalent fluid pressure of 350 psf per foot of depth. These two modes of resistance should not be added unless the frictional component is reduced by 50 percent, since full mobilization of the passive resistance requires some horizontal movement, which significantly diminishes the frictional resistance.

We recommend that all foundation excavations be observed by our representative prior to placement of reinforcement and concrete to verify firm bearing materials are exposed.

Shallow Foundations on Geopier® Rammed Aggregate Piers

We anticipate a Geopier® RAP system could provide adequate support for the proposed structure supported on continuous and/or isolated spread foundations or a mat foundation. A qualified RAP contractor licensed in the State of California should be contacted directly to provide final recommendations for the Geopier® RAP system, including allowable capacities and settlements.

Continuous and/or isolated spread foundations bearing on a Geopier® RAP improved subgrade should extend at least 18 inches below the lowest adjacent soil grade of the structure pad. For this project, the pad subgrade is the surface on which aggregate materials (i.e., aggregate base below slab areas of the structures or capillary break materials within proposed building areas) are placed. Isolated spread foundations should be at least 18 inches wide.

Preliminary design information indicates allowable rammed aggregate pier capacities of 85 kips and a bearing capacity of 6000 psf for dead plus live load can be achieved on Geopier® RAPs. The RAP layout and final bearing pressures and cell capacities will depend on the actual loading conditions for each structure and should be determined by the RAP designer and should include an appropriate factor of safety. The weight of foundation concrete extending below adjacent soil grade may be disregarded in sizing computations.

Uplift resistance can be provided using ground improvement equipped with a steel uplift anchor and can provide about 35 kips of allowable uplift.

We recommend that all foundations be reinforced to provide structural continuity, reduce cracking and permit spanning of local soil irregularities. The project structural engineer should determine final foundation reinforcement. However, as a minimum, we recommend continuous foundations contain at least four No. 4 reinforcing bars, placed two each near the top and bottom of the foundation.

Preliminary resistance to lateral foundation displacement for conventional foundations supported on RAPs may be computed using an allowable friction factor of 0.45, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an allowable passive earth pressure of 350 psf per foot of depth. These two modes of resistance should not be added unless the frictional value is reduced by 50 percent since full mobilization of these resistances typically occurs at different degrees of horizontal movement.

Auger Cast-in-Place (ACIP) Concrete Piles

The proposed structure also may be supported upon ACIP concrete piles. ACIP concrete piles are installed using special equipment equipped with hollow-stem augers. Once the pile hole has been drilled, grout/concrete is injected under pressure through the auger to displace the soil and provide positive contact with the surrounding soils. Reinforcement is placed into the grouted shaft after withdrawal of the auger.

Piles for the structure should extend to a minimum of approximately two feet into the relatively dense gravels, which were encountered at depths of about 26 to 28 feet below the ground surface at the boring locations. Drilled ACIP concrete piles may be designed utilizing the following maximum allowable loads per pile with appropriate factor of safety (F.S.) as summarized in the table below as follows:

ALLOWABLE ACIP PILE CAPACITIES					
Loading Conditions		18-inch Diameter		24-inch Diameter	
		Allowable Pile Capacity (tons)	Ultimate Pile Capacity (tons)	Allowable Pile Capacity (tons)	Ultimate Pile Capacity (tons)
Axial Compression	DL (F.S. = 3)	80	240	140	420
	DL + LL (F.S. = 2)	120	240	210	420
	Total Load (F.S. = 1.5)	160	240	280	420
Axial Uplift (Tension)	Total Load (F.S. = 1.5)	40	60	70	105

Reductions in pile capacity for consideration of group action are unnecessary, provided piles are spaced no closer (center-to-center) than three times the diameter of the pile.

The indicated uplift pile capacity is based upon the assumption that the piles will be properly reinforced to transfer pullout forces to the pile tip.

Lateral loading information was not available at the time this report was prepared. The lateral resistance of individual piles and the passive resistance of the pile cap against the soil can be combined to provide lateral resistance. For preliminary design purposes, 18-inch ACIP piles can be assumed to provide an allowable lateral resistance of five (5) tons and 24-inch ACIP piles can be assumed to provide an allowable lateral resistance of 10 tons. Both lateral resistance values are based on a pile deflection of one-inch. Resistance to lateral loads for ACIP piles can be determined and presented in a supplemental report using a lateral pile analysis program when final size design information is known and if required to further aid in the structural design.

The weight of pile cap concrete extending below grade and the weight of each pile may be disregarded in determinations of the net compressive load transmitted to the supporting soil.

Concurrent lateral resistance derived in friction between the slab and the supporting subgrade layer may be computed using an allowable friction factor of 0.30 at the interface between the slab and the subgrade.

A pile load test program will be necessary to determine the correct length of the ACIP piles to achieve the specified capacities. Additional load testing could be performed during construction, where as-built pile dimensions differ from the recommended dimensions, which could result from refusal to auger penetration in denser/stiffer soils beneath this site.

Drilled Cast-in-Place Concrete Piers

Drilled, cast-in-place piers (drilled piers) may be used to support the proposed structure. Drilled piers should be at least 24 inches in diameter and extend to at least two (2) feet into the relatively dense gravels encountered about 26 to 28 feet below the ground surface at the boring locations. Piers so established may be designed based on an allowable end bearing capacity of 6000 psf for dead plus live loads. We recommend that adjacent piers be constructed no closer than three pier diameters apart, as measured between centers of the piers. Drilled pier foundations should be structurally isolated from any adjacent concrete flatwork by a felt strip or similar material.

Due to the anticipated depth of groundwater and the required drilled pier depths, the contractor should be prepared to construct the drilled piers using wet drilling methods (i.e., casing, slurry, etc.).

Uplift resistance of the pier foundations may be computed assuming the following resisting forces, where applicable: 1) the unit weight of foundation concrete (150 pound per cubic foot); and, 2) shearing resistance of 350 psf applied over the shaft area of the pier. Increased uplift resistance can be achieved by increasing the diameter of the pier or increasing the depth of the embedment depth.

It will be essential that our representative be present during pier drilling operations to verify compliance with our recommendations and the job specifications.

Lateral resistance of drilled piers can also be evaluated by determining the shear, moment and deflection of the pier using a computer model of the pier and soil (i.e. LPILE). Such an analysis is beyond the current scope of this evaluation and can be accomplished after the dimensions of the piers and loading conditions are known, if desired.

The bottom of the pier excavations should be free of loose or disturbed soils prior to placement of the concrete. Cleaning of the bearing surface may be done mechanically with the belling bucket, but should be verified by the geotechnical engineer prior to concrete placement.

Reinforcement and concrete should be placed in the pier excavations as soon as possible after excavation is completed to reduce the potential of sidewall caving into the excavations. Excessive sloughing of the sidewalls during pier construction is anticipated for piers extending deeper than about 10 feet below the existing ground surface. Therefore, we recommend that the pier contractor be prepared to case the pier holes or use drilling fluid (slurry) if conditions require.

To reduce lateral movement of the drilled shafts, it is necessary to place the concrete for the drilled shafts in intimate contact with the surrounding soil. Any voids or enlargements in the shafts due to over-excavation or temporary casing installation shall be filled with concrete at the time the shaft concrete is placed.

If the drilled piers are constructed in the "dry" (with dry being less than two inches of water at the base of the excavation), the concrete may be placed by the free-fall method, using a short hopper or back-chute to direct the concrete flow out of the truck into a vertical stream of flowing concrete with a relatively small diameter. The stream is directed to avoid hitting the sides of the excavation or any reinforcing cages. For the free-fall method of concrete placement, we recommend the concrete mix be designed with a slump of five to seven inches.

In general, we anticipate the drilled pier excavations will be relatively dry for pier excavations extending less than about 15 feet below the existing ground surface. For excavations extending deeper than about 15 feet below the existing ground surface we anticipate groundwater will be encountered which cannot be controlled such that more than six (6) inches of water accumulates at the bottom of the pier excavation. After it is confirmed that the excess water cannot be removed from the caisson excavation by bailing or with pumps, concrete should be placed using a tremie. For concrete placed using the tremie method, a slump of six to eight (8) inches, and a maximum aggregate size of ¾-inch is recommended. The required slump should be obtained by using plasticizers or water-reducing agents. Addition of water on-site to establish the recommended slump should not be allowed.

When extracting temporary casings or tremie methods from the excavation, care should be taken to maintain a head of concrete to prevent infiltration of water and soil into the shaft area. The head of concrete should always be greater than the head of water trapped outside the pier or tremie, taking into account the differences in unit weights of concrete and water.

We estimate total settlement for drilled pier foundations using the recommended maximum net allowable bearing pressure and allowable capacities presented above, will be less than one (1) inch. Differential settlements may be as much as the total settlement between individual pier elements. The settlement estimates are based on the available soil information, our experience with similar structures and soil conditions, and field verification of suitable bearing soils during foundation construction.

Pile Load Testing Program

If ACIP are used for support of the structure, a pile loading testing program conducted prior to installation of production piles will be necessary to determine and verify the appropriate length of pile to achieve the **ultimate capacity** of the piles. The pile load test program should include both static load tests and pile driving analyzer (PDA) tests. The purpose of the PDA testing for the pre-construction piles would be to develop a correlation between the static load test results and the PDA testing that would be used during the construction of production piles in lieu of "quick" load tests. The advantage of PDA testing over the "quick" load pile testing is the savings in time to set up the load test frame that typically takes three to five days, and a "quick" load test program often takes about eight hours per pile to complete

Static Load Testing

The pile load test frame and supply of the personnel and equipment necessary to conduct the load tests should be constructed in accordance with the latest version of ASTM Test Method D1143 for compressive loads, ASTM Test Method D3689 for tensile loads, and ASTM Test Method D3966 for lateral loads as delineated in the *Guide Specifications for Auger Cast Piles* provided as Appendix B.

One test pile should be cast-in-place to reach minimum tip elevations of at least 30 feet below the ground surface and at least two (2) feet into the gravel stratum. Additional test piles will be required if multiple pile sizes are used in the design or if alternate pile capacities are being considered. The reaction system should be capable of resisting forces from tests on the test piles in axial compression and tension as specified in the previous Allowable Pile Capacities table. We intend to test the test pile in compression and tension, and to perform a lateral load test between adjacent piles. The pile may be loaded to failure in any of the test configurations.

Submittals for the load testing frame, hydraulic pumps, hydraulic jacks, dial indicators, and calibration documentation must be provided by the pile contractor in accordance with the project plans and specifications.

Prior to beginning load tests, the pile concrete should achieve a minimum compressive strength of 4000 pounds per square inch when tested in accordance with ASTM C109. Construction activities must be restricted during the load-testing program. Construction activities may proceed during the set up of the load frame and installation of the test piles. Excessive vibration of the ground near the load test can cause movement of the test frame and the sensitive pile deflection measurement devices.

Final pile construction criteria will be determined from the results of the load-testing program. It is intended that the pile load test setup will be located outside the location of any permanent pile caps or grade beams, and that the test piles and reaction piles will be abandoned upon completion of the testing.

Pile Driving Analyzer Testing

Following the static load testing program, the test pile will be subjected to PDA testing, provided the pile is not damaged during the static load testing. PDA testing involves instrumenting piles and recording the response of the pile during dynamic loading. PDA testing consists of dropping a heavy weight from a certain height on to the pile head and monitoring the response of the pile. The capacity of the piles can be computed from the analyses of the PDA test.

Additional PDA testing will be performed during construction of production piles, in the event that as-built pile dimensions differ from the recommended dimensions, which could result from refusal to auger penetration or in random areas across the site to verify that the earth materials are supporting the piles as indicated by the load test program.

<u>Surveillance/Protection</u>

We recommend that photographic and written records be kept of both the pre-existing condition and new damage (if any) sustained by improvements in and around the site. The elevation of sidewalks and buildings adjacent to the construction site should be measured prior to construction activities. The elevations of selected survey points should be measured on a weekly basis during the initial stages of construction. Elevation of improvements and photographs should include basic data for determining the validity of claims lodged by nearby property owners or tenants.

Below-Grade Walls and Drainage

Foundations for below-grade walls may be designed and constructed as noted in the Foundation Design section of this report. The walls may be designed for an "active" earth pressure of 50 psf per foot of wall height, assuming the wall is free to rotate. If the wall is restrained at the top, or is rigid enough so that it does not rotate sufficiently to reach the active earth pressure condition, a higher lateral "at rest" earth pressure of 70 psf per foot of wall height should be used for design of rigid walls. These values do not include the effect of hydrostatic forces and assume the wall backfill is fully drained or that free water cannot collect behind the walls. Lateral resistance may be computed using an allowable passive earth pressure of 250 psf per foot of depth.

If the walls are designed to include the effects of hydrostatic forces, active and at rest pressures would increase to 90 pcf and 100 pcf, respectively, to include the effect of hydrostatic pressures. Passive pressures below the groundwater table can be evaluated using 185 pcf.

Retaining walls could experience additional surcharge loading if equipment is stored within a 1:1 projection from the bottom of the excavation. Surcharge loading under these circumstances will need to be evaluated on a case-by-case basis.

Based on recent research (Lew, et al. 2010), the seismic increment of earth pressure may be neglected if the maximum ground acceleration is 0.4 g or less. Our analysis indicates the maximum ground acceleration will be about 0.23 g; therefore, the seismic increment of earth pressure may be neglected. Earth pressures due to seismic loading may be evaluated using a total active earth pressure of 50 psf per foot of wall height and a total passive earth pressure of 200 psf per foot of wall height. The resultant active force should be applied at 1/3 times the height of the retaining wall, measured from the bottom of the wall.

Wall drainage should consist of a drainage blanket of Class 2 permeable material (Caltrans Specification Section 68-1.025) at least one foot wide extending from the base of wall to within one foot of the top of the wall. The top foot above the drainage layer should consist of engineered fill placed in accordance with the recommendations of this report. Perforated pipe should be provided at the base of the wall to collect accumulated water. Drain pipes, if used, should slope to discharge at no less than a one percent fall to a suitable sump system or drainage facilities. Open-graded ½- to ¾-inch crushed rock may be used in lieu of the Class 2 permeable material, if the rock and drain pipe are completely enveloped in an approved non-woven geotextile filter fabric. Alternatively, geotextile drainage composites such as

MiraDRAIN® may be used in lieu of the drain rock layer. If used, geocomposite drain panels should be installed in accordance with the manufacturer's recommendations.

If efflorescence (discoloration of the wall face) or moisture penetration of the wall is not acceptable, waterproofing measures should be applied to the back face of the wall. A specialist in protection against moisture penetration should be consulted to determine specific waterproofing measures.

Structural backfill materials for retaining walls should be placed and compacted as noted in the <u>Engineered Fill Construction</u> section of this report. Pea gravel and crushed rock are not considered suitable backfill materials for retaining walls.

<u>Interior Grade Slab Support</u>

The interior concrete slabs-on-grade can be supported upon the soil subgrade prepared in accordance with the recommendations in this report and maintained in that condition. Slabs-on-grade that will be used for vehicle support should be designed in accordance with the recommendations provided in the <u>Pavement Design</u> section of this report.

Interior slab-on-grade concrete slabs that will <u>not</u> be used for vehicle support should be at least four inches thick and, as a minimum, contain chaired No. 3 reinforcing bars on 18-inch center-on-center spacing, located at mid-slab depths. All reinforcing should be located at mid-slab depth. This slab reinforcement is suggested as a guide "minimum" only for crack control; final reinforcement and joint spacing should be determined by the structural engineer. Wheel loads from forklifts, storage of palletized materials, cranes, etc., anticipated during construction should be considered in the design of the slab-on-grade floors.

Conventional floor slabs may be underlain by a layer of free-draining gravel serving as a deterrent to migration of capillary moisture. If used, the gravel layer should be at least four inches thick and graded such that 100 percent passes a one-inch sieve and no appreciable amount passes a No. 4 sieve. Additional moisture protection may be provided by placing a water vapor retarder (at least 10-mils thick) directly over the gravel. If used, the water vapor retarder should meet or exceed that standard specification as outlined in ASTM E1745.

Floor slab construction practice over the past 25 years or more has included placement of a thin layer of sand over the vapor retarder membrane. The intent of the sand is to aid in the proper curing of the slab concrete. However, recent debate over excessive moisture vapor emissions from floor slabs includes concern of water trapped within the sand. As a consequence, we consider use of the sand layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

The recommendations presented above should reduce significant soils-related cracking of slab-on-grade floors. Also important to the performance and appearance of a Portland cement concrete slab is the quality of the concrete, the workmanship of the concrete contractor, the curing techniques utilized and spacing of control joints.

Floor Slab Moisture Penetration Resistance

It is likely the floor slab subgrade soils will become saturated at some time during the life of the structure, especially when slabs are constructed during the wet season and when constantly wet ground or poor drainage conditions exist adjacent to structures. For this reason, it should be assumed that all interior slabs, particularly those intended for moisture-sensitive floor coverings or materials, require protection against moisture or moisture vapor penetration. Standard practice includes placing a layer of rock and a vapor retarder membrane (and possibly a layer of sand) as discussed above. Recommendations contained in this report concerning foundation and floor slab design are presented as minimum requirements only from the geotechnical engineering standpoint.

Use of sub-slab gravel and a vapor retarder membrane will not "moisture proof" the slab, nor does it assure that slab moisture vapor transmission levels will be low enough to prevent damage to floor coverings or other building components. It is emphasized that we are not slab moisture proofing or moisture protection experts. The sub-slab gravel and vapor retarder membrane simply offer a first line of defense against soil-related moisture. If increased protection against moisture vapor penetration of the slab is desired, a concrete moisture protection specialist should be consulted. It is commonly accepted that maintaining the lowest practical water-cement ratio in the slab concrete is one of the most effective ways to reduce future moisture vapor penetration of the completed slab.

Exterior Concrete Flatwork

Exterior concrete flatwork may be constructed directly on the prepared soil subgrade prepared and compacted in accordance with the recommendations of this report. A four-inch layer of aggregate base could be used as a leveling course under flatwork if necessary, compacted to not less than 95 percent relative compaction.

Flatwork should be at least four inches thick and reinforced for crack control. Reinforcement should include, as a minimum, chaired No. 3 rebar located on maximum 18-inch centers, both ways, throughout slabs. Accurate and consistent location of the reinforcement at mid-slab is essential to its performance and the risk of uncontrolled drying shrinkage slab cracking is increased if the reinforcement is not properly located within the slab.

Uniform moisture conditioning of subgrade soils is important to reduce the risk of non-uniform moisture withdrawal from the concrete and the possibility of plastic shrinkage cracks. Practices recommended by the Portland Cement Association (PCA) for proper placement and curing of concrete should be followed during exterior concrete flatwork construction. Flatwork should be independent of the building foundations and felt strips should be used to separate concrete slabs from building foundations.

The architect or civil engineer should determine the final thickness, strength, reinforcement, and joint spacing of exterior slab-on-grade concrete. Exterior flatwork next to landscaped areas should be thickened to twice the slab thickness for a width of at least 12 inches to help support lawn mowing equipment and other maintenance equipment.

Exterior flatwork should be constructed independent of the building foundations. Isolated column foundations should be structurally separated from adjacent flatwork by the placement of a layer of felt, or other appropriate material, between the flatwork and foundations. Practices recommended by the Portland Cement Association (PCA) for proper placement and curing of concrete should be followed during exterior concrete flatwork construction.

Exterior flatwork that will be traversed by vehicles or heavy equipment should be designed in accordance with the recommendations provided in the Pavement Design section of this report.

Pavement Design

We are providing several alternative pavement designs based on the soil conditions encountered at the site, the results of laboratory testing previously obtained at the site, and our experience.

The procedures used to design the pavement sections are in general conformance with the "Flexible Pavement Structural Design Guide for California Cities and Counties" dated January 1979, and the *California Highway Design Manual, Sixth Edition*. Laboratory testing of the on-site soils indicates an R-value of 61 was obtained on the near-surface soils at the site. Based on our experience with similar soil conditions and the variability of the near-surface soils, an R-value of 40 is considered appropriate for design of pavements at the site.

PAVEMENT DESIGN ALTERNATIVES R-value = 40				
Traffic Index (TI)	Traffic Condition	Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)
4.5	Automobile Parking Only	2½*	4	--
		--	4	4
7.0	Entrance/Exit Driveways & Traffic Lanes	3	9	--
		4*	7	--
		--	4	6

* = Asphalt thickness includes Caltrans Factor of Safety.

We emphasize that the performance of the pavement is critically dependent upon adequate and uniform compaction of the subgrade soils, including utility trench backfill within the limits of the pavements. The upper six inches of untreated pavement subgrade should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. Aggregate base materials should be compacted to at least 95 percent of the maximum dry density. Class 2 aggregate base should conform to Section 26 of the Caltrans Standard Specifications.

It has been our experience that pavement failures may occur where a non-uniform or disturbed subgrade soil condition is created. Subgrade disturbances can result if pavement subgrade preparation is performed prior to underground utility construction and/or if a significant time period passes between subgrade preparation and placement of aggregate base. Therefore, we recommend that final pavement subgrade preparation (i.e. scarification, moisture conditioning, and compaction) be performed just prior to aggregate base placement.

We suggest that concrete slabs be constructed with thickened edges at least two inches plus the slab thickness and 36 inches wide in accordance with American Concrete Institute (ACI) design standards. Reinforcing for concrete pavement crack control, if desired, should consist of No. 3 reinforcing bars placed on maximum 18-inch centers each way throughout the slab.

Reinforcement must be located at mid-slab depth to be effective. Portland cement concrete should achieve a minimum compressive strength of 3500 psi at 28 days. Concrete curing and joint spacing and details should conform to current PCA and ACI guidelines.

We suggest considering the use of full depth curbs where pavements abut landscaping. The curbs should extend to at least the surface of the soil subgrade. Weep holes also could be provided at storm drain drop inlets, located at the subgrade-base interface, to allow water to drain from beneath the pavements.

Site Drainage

Site drainage should be accomplished to provide positive drainage of surface water away from the proposed structures and prevent ponding of water adjacent to foundations. The subgrade adjacent to the proposed structures should be sloped away from foundations at a minimum two percent gradient for at least 10 feet, where possible. We recommend consideration be given to connecting all roof drains to non-perforated rigid pipes which are connected to available drainage features to convey water away from the structure, or discharging the drains onto paved surfaces that slope away from the foundations. Ponding of surface water should not be allowed adjacent to the proposed structures or pavements.

Observation and Testing of Earthwork Construction

Site preparation should be accomplished in accordance with the recommendations of this report. Representatives of the Geotechnical Engineer should be present during site preparation and all grading operations to observe and test the fill to verify compliance with our recommendations and the job specifications. These services are beyond the scope of work authorized for this investigation.

Additional Services

We recommend that our firm be retained to review the final plans and specifications to determine if the intent of our recommendations has been implemented in those documents.

LIMITATIONS

Our recommendations are based upon the information provided regarding the proposed project, combined with our analysis of site conditions revealed by the field exploration and laboratory testing programs. We have used prudent engineering judgment based upon the information provided and the data generated from our investigation.

This report has been prepared in substantial compliance with generally accepted geotechnical engineering practices that exist in the area of the project at the time the report was prepared. No warranty, either express or implied, is provided.

If the proposed construction is modified or re-sited; or, if it is found during construction that subsurface conditions differ from those we encountered at our boring locations, we should be afforded the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations must be modified.

We emphasize that this report is applicable only to the proposed construction and the investigated site, and should not be utilized for construction on any other site. The conclusions and recommendations of this report are considered valid for a period of two years. If design is not completed and construction has not started within two years of the date of this report, the report must be reviewed and updated if necessary.

Wallace - Kuhl & Associates

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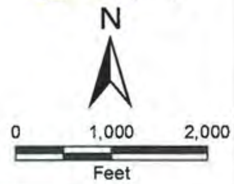
Street data courtesy of Sacramento County.
Hydrography courtesy of the U.S. Geological Survey
acquired from the GIS Data Depot, December, 2007.
Projection: NAD 83, California State Plane, Zone II



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& ASSOCIATES

VICINITY MAP

L STREET MIXED USE

Sacramento, California

FIGURE	1
DRAWN BY	TJC
CHECKED BY	MSM
PROJECT MGR	MSM
DATE	1/14
WKA NO. 9955.01	

Adapted from a Google Earth aerial photograph,
dated August 14, 2013.
Projection: NAD 83, California State Plane, Zone II

Legend

Approximate soil boring location

N

0 35 70
Feet

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SITE PLAN

L STREET MIXED USE

Sacramento, California

FIGURE	2
DRAWN BY	TJC
CHECKED BY	MSM
PROJECT MGR	MSM
DATE	1/14
WKA NO. 9955.01	

Project: L Street Mixed Use
Project Location: Sacramento, California
WKA Number: 9955.01

LOG OF SOIL BORING D1

Date(s) Drilled	11/23/13	Logged By	GJF	Checked By	MSM
Drilling Method	Hollow Stem Auger	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	50.0 feet
Drill Rig Type	CME-75	Diameter(s) of Hole, inches	8"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	18.0	Sampling Method(s)	California Modified	Drill Hole Backfill	Cement Grout
Remarks				Driving Method and Drop	140-lb automatic hammer; 30-inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Asphalt concrete						
			Aggregate base						
			Brown, moist, medium dense, silty fine sand (SM)		D1-1I	21	14.4	86	
			Brown, moist to wet, very loose to medium dense, sandy silt with occasional clayey silt seams and layers (ML)						
	5				D1-2I	11	20.6	89	
	10				D1-3I	10	26.0	96	TR
	15				D1-4I	3	31.5	91	
			▼						
	20				D1-5I	4	33.3	91	

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Project: L Street Mixed Use
Project Location: Sacramento, California
WKA Number: 9955.01

LOG OF SOIL BORING D1

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
					D1-6I	29			
			Gray brown, wet, very dense, silty sandy gravel (GM)						
	30				D1-7I	50/5"			
	35								
	40								
			Brown, wet, dense, sandy silt (ML)						
	45				D1-8I	38			
	50		Boring terminated at 50 feet below existing site grade. Groundwater was encountered about 18 feet below the ground surface during drilling and about 18 feet below the ground surface immediately after drilling.						

Project: L Street Mixed Use Project Location: Sacramento, California WKA Number: 9955.01	# LOG OF SOIL BORING D2

Date(s) Drilled	11/23/13	Logged By	GJF	Checked By	MSM
Drilling Method	Hollow Stem Auger	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	51.0 feet
Drill Rig Type	CME-75	Diameter(s) of Hole, inches	8"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	18.0	Sampling Method(s)	California Modified	Drill Hole Backfill	Cement Grout
Remarks				Driving Method and Drop	140-lb automatic hammer; 30-inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Asphalt concrete						
			Aggregate base						
			Brown, moist to wet, very loose to medium dense, sandy silt with occasional sand and clay seams and layers (ML)		D2-1I	17	23.9	88	
	5				D2-2I	19	21.2	96	
	10				D2-3I	9	25.5	92	
	15				D2-4I	2	28.5	94	
	20				D2-5I	1	34.8	88	

BORING LOG 9955.01 - L STREET MIXED USE.GPJ WKA.GDT 1/21/14 1:16 PM

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FIGURE 4

Project: L Street Mixed Use
Project Location: Sacramento, California
WKA Number: 9955.01

LOG OF SOIL BORING D2

ELEVATION, feet	DEPTH, feet	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		Gray brown, wet, very dense to dense, silty sandy gravel (GM)	D2-6I	55			
	30		D2-7I	50/3"			
	35		D2-8I	48			
	40						
		Light brown, wet, very stiff, clayey silt (ML)					
	45		D2-9I	33	53.0	57	TR
	50		D2-10I	50/4"	24.4	100	

Boring terminated at 51 feet below existing site grade. Groundwater was encountered about 18 feet below the ground surface during drilling and about 18 feet below the ground surface immediately after drilling.

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FIGURE 4

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		SYMBOL	CODE	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of soil > no. 200 sieve size)	GRAVELS (More than 50% of coarse fraction > no. 4 sieve size)	GW		Well graded gravels or gravel - sand mixtures, little or no fines
		GP		Poorly graded gravels or gravel - sand mixtures, little or no fines
		GM		Silty gravels, gravel - sand - silt mixtures
		GC		Clayey gravels, gravel - sand - clay mixtures
	SANDS (50% or more of coarse fraction < no. 4 sieve size)	SW		Well graded sands or gravelly sands, little or no fines
		SP		Poorly graded sands or gravelly sands, little or no fines
		SM		Silty sands, sand - silt mixtures
		SC		Clayey sands, sand - clay mixtures
FINE GRAINED SOILS (50% or more of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL		Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS LL ≥ 50	MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH		Inorganic clays of high plasticity, fat clays
		OH		Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils
ROCK		RX		Rocks, weathered to fresh
FILL		FILL		Artificially placed fill material

OTHER SYMBOLS

= Drive Sample: 2-1/2" O.D. Modified California sampler

= Drive Sampler: no recovery

= SPT Sampler

= Initial Water Level

= Final Water Level

= Estimated or gradational material change line

= Observed material change line

Laboratory Tests

PI = Plasticity Index

EI = Expansion Index

UCC = Unconfined Compression Test

TR = Triaxial Compression Test

GR = Gradational Analysis (Sieve)

K = Permeability Test

GRAIN SIZE CLASSIFICATION

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL	3" to No. 4	76.2 to 4.76
coarse (c)	3" to 3/4"	76.2 to 19.1
fine (f)	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.074
coarse (c)	No. 4 to No. 10	4.76 to 2.00
medium (m)	No. 10 to No. 40	2.00 to 0.420
fine (f)	No. 40 to No. 200	0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

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UNIFIED SOIL CLASSIFICATION SYSTEM

L STREET MIXED USE

Sacramento, California

FIGURE	5
DRAWN BY	TJC
CHECKED BY	MSM
PROJECT MGR	MSM
DATE	1/14
WKA NO. 9955.01	

APPENDICES

APPENDIX A
Field and Laboratory Testing

APPENDIX A

A. GENERAL INFORMATION

The performance of a geotechnical engineering investigation for the proposed L Street Mixed Use project, to be constructed on the north side of L Street between 20th and 21st Streets in Sacramento, California was authorized by Steve Vannatta on November 20, 2013. Authorization was for an investigation as described in our proposal letter dated September 10, 2013, sent to our client LVP Revocable Trust, whose address is 2020 L Street, 5th Floor, Sacramento, California 95811; telephone (916) 447-7100; facsimile (916) 447-7112.

B. FIELD EXPLORATION

Two (2) borings were drilled at the site on November 23, 2013, at the approximate locations indicated on Figure 2 utilizing a CME-75 truck-mounted drill rig. The borings were drilled to maximum depths of approximately 50 to 51 feet below existing site grades using eight-inch (8") diameter, hollow-stem helical augers. At various intervals, relatively undisturbed soil samples were recovered with a 2½-inch O.D., 2-inch I.D., modified California sampler driven by an automatic 140-pound hammer freely falling 30 inches. The number of blows of the hammer required to drive the 18-inch long sampler each six-inch (6") interval was recorded. The sum of the blows required to drive the sampler the lower 12-inch interval, or portion thereof, is designated the penetration resistance or "blow count" for that particular drive.

The samples were retained in two-inch (2") diameter by six-inch (6") long thin-walled brass tubes contained within the sampler. Immediately after recovery, the soils in the tubes were visually classified by the field engineer and the ends of the tubes were sealed to preserve the natural moisture contents. All samples were taken to our laboratory for soil classification and selection of samples for testing.

The Logs of Soil Borings, Figures 3 and 4, contain descriptions of the soils encountered at each boring location. A Boring Legend explaining the Unified Soil Classification System and the symbols used on the logs is contained on Figure 5.

C. LABORATORY TESTING

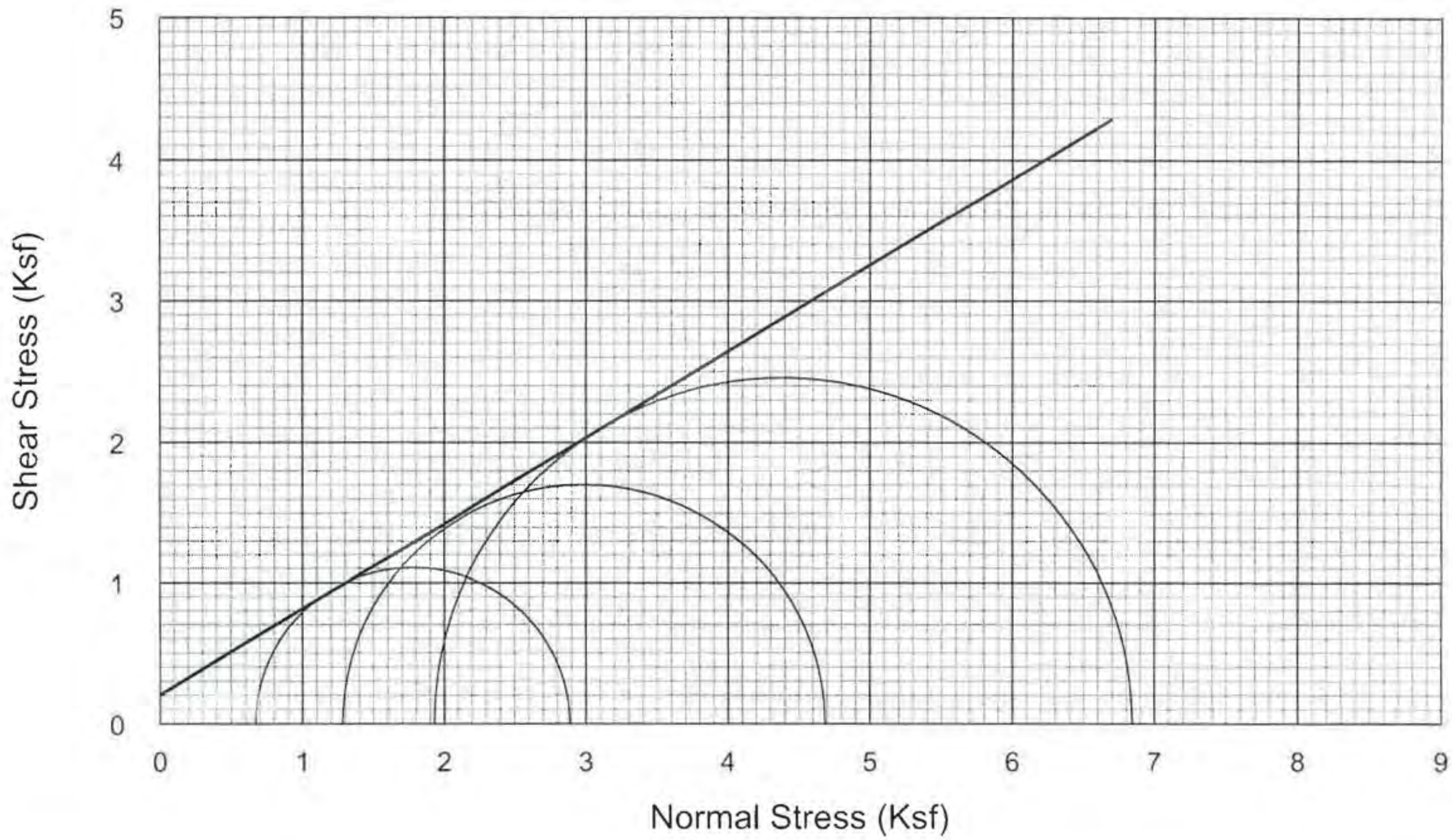
Selected undisturbed samples of the soils were tested to determine dry unit weight (ASTM D2937), natural moisture content (ASTM D2216) and shear strength by triaxial strength testing (ASTM D4767). The results of the moisture/density tests are included on the boring logs at the depth each sample was obtained. The results of the shear strength testing are presented on Figures A1 and A2.

A bulk sample of the anticipated pavement subgrade soil was subjected to Resistance-value ("R-value") testing in accordance with California Test 301. The results of the R-value test, which were used in the pavement design, are presented on Figure A3.

A composite sample of near-surface soil was submitted to Sunland Analytical of Rancho Cordova, California, for corrosivity testing in accordance with California Test (CT) Nos. 643 (Modified Small Cell), CT 422 and CT 417. Copies of the analytical results are presented on Figure A4.

TRIAXIAL COMPRESSION TEST

ASTM D4767



SAMPLE NO. :	D1-3I
SAMPLE CONDITION :	Undisturbed
SAMPLE DESCRIPTION :	Brown, sandy silt

DRY DENSITY (PCF) :	96
INITIAL MOISTURE (%) :	26.0
FINAL MOISTURE (%) :	26.9
ANGLE OF INTERNAL FRICTION (ø) :	31°
COHESION (PSF) :	203

WallaceKuhl & ASSOCIATES

TRIAXIAL COMPRESSION TEST RESULTS

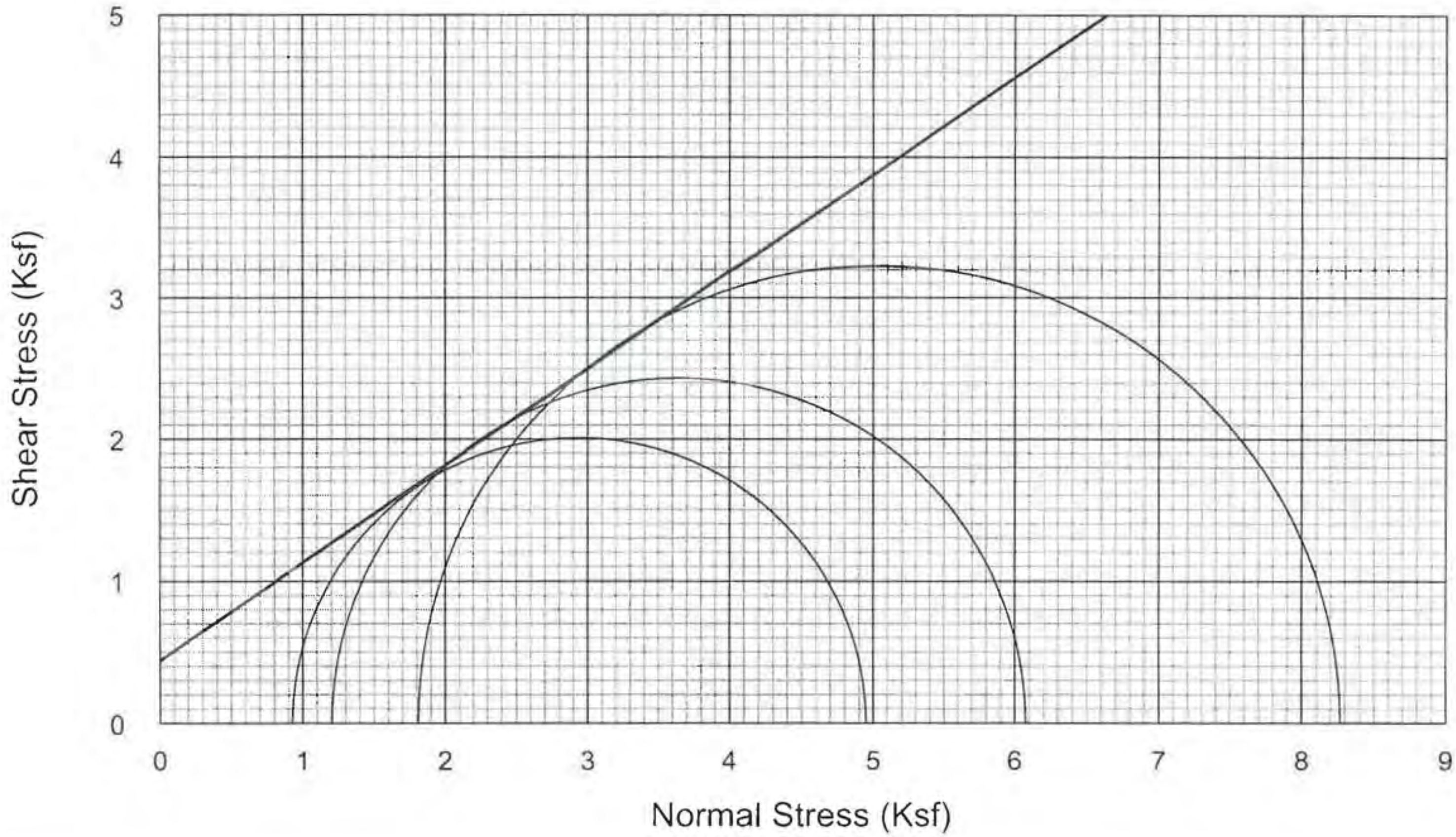
L STREET MIXED USE

Sacramento, California

FIGURE	A1
DRAWN BY	TJC
CHECKED BY	MSM
PROJECT MGR	MSM
DATE	1/14
WKA NO. 9955.01	

TRIAXIAL COMPRESSION TEST

ASTM D4767



SAMPLE NO. :	D2-9I	DRY DENSITY (PCF) :	67
SAMPLE CONDITION :	Undisturbed	INITIAL MOISTURE (%) :	53.0
SAMPLE DESCRIPTION :	Light brown, clayey silt	FINAL MOISTURE (%) :	53.4
		ANGLE OF INTERNAL FRICTION (ø) :	34°
		COHESION (PSF) :	438

WallaceKuhl & ASSOCIATES

TRIAXIAL COMPRESSION TEST RESULTS

L STREET MIXED USE

Sacramento, California

FIGURE	A2
DRAWN BY	TJC
CHECKED BY	MSM
PROJECT MGR	MSM
DATE	1/14

WKA NO. 9955.01

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Brown, sandy silt

LOCATION: D2 (1'-3')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion Pressure (dial)	Expansion Pressure (psf)	R Value
1	114	14.9	185	18	78	29
2	116	13.7	316	52	225	64
3	118	12.7	402	60	260	69

R-Value at 300 psi exudation pressure = 61

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RESISTANCE VALUE TEST RESULTS

L STREET MIXED USE

Sacramento, California

FIGURE	A3
DRAWN BY	TJC
CHECKED BY	MSM
PROJECT MGR	MSM
DATE	1/14
WKA NO.	9955.01

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 12/13/2013
Date Submitted 12/10/2013

To: Matt Moyneur
Wallace-Kuhl & Assoc.
3050 Industrial Blvd.
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 9955.01-L ST MIX USE Site ID : D1-BULK 1-3 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 66074-136782.

EVALUATION FOR SOIL CORROSION

Soil pH	8.02	
Minimum Resistivity	0.86 ohm-cm (x1000)	
Chloride	219.3 ppm	00.02193 %
Sulfate	323.8 ppm	00.03238 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

WallaceKuhl

CORROSION TEST RESULTS

L STREET MIXED USE

Sacramento, California

FIGURE	A4
DRAWN BY	TJC
CHECKED BY	MSM
PROJECT MGR	MSM
DATE	1/14
WKA NO. 9955.01	

APPENDIX B
Guide Specifications for Auger Cast Piles

APPENDIX B

GUIDE SPECIFICATIONS FOR AUGER CAST PILES

L STREET MIXED USE

Sacramento, California

WKA No. 9955.01

PART 1: GENERAL

1.1 SUMMARY

A. This Section includes construction of compression and tension auger cast piles, where shown on contract drawings and specified herein.

B. The Contractor shall furnish all labor, materials, tools, and equipment necessary for designing, furnishing, installing, inspecting and testing augered cast-in-place piles, and shall remove and dispose spoils generated by pile construction.

1.2 WORK NOT INCLUDED UNDER THIS SECTION

A. Concrete pile caps: Section _______.

B. Excavations: Section ______.

C. Shoring and bracing of earth banks: Section ______.

D. Dewatering: Section ______.

1.3 REFERENCE STANDARDS

A. Requirements, abbreviations and acronyms for reference standards are defined in Section _____.

B. American Concrete Institute (ACI)

1. ACI 305 - Hot Weather Concreting.
2. ACI 306 - Cold Weather Concreting.
3. ACI 315 - Details and Detailing of Concrete Reinforcement.

C. American Society for Testing and Materials (ASTM) latest editions

1. ASTM A615 - Deformed and Plain Billet-Steel Bars for Concrete Reinforcement.
2. ASTM C33 - Concrete Aggregates.
3. ASTM C31 - Standard Practice for Making and Curing Concrete Test Specimens in the Field
4. ASTM C109 - Test Method for Compressive Strength of Hydraulic Cement Mortars.
5. ASTM C150 - Portland Cement.
6. ASTM C618 - Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete.
7. ASTM C939 - Test Method for Flow of Grout for Preplaced - Aggregate Concrete (Flow Cone Method)
8. ASTM C942 - Test Method for Compressive Strength of Grouts for Preplaced-Aggregate Concrete in the Laboratory.
9. ASTM D1143 - Test Method for Piles Under Static Axial Compressive Load.
10. ASTM D3689 - Test Method for Individual Piles Under Static Axial Tensile Load.
11. ASTM D3966 - Test Method for Piles Under Lateral Loads.

1.4 PROTECTION

A. Adequate protection measures shall be provided to protect workers and passers-by at the site. Streets and adjacent property shall be fully protected throughout the operations.

B. In accordance with generally accepted construction practices, the Contractor shall be solely and completely responsible for working conditions at the job site, including safety of all persons and property during performance of the work. This requirement shall apply continuously and shall not be limited to normal working hours.

C. Any construction review of the Contractor's performance conducted by the Geotechnical Engineer is not intended to include review of the adequacy of the Contractor's safety measures, in, on or near the construction site.

D. Adjacent streets and sidewalks shall be kept free of mud, dirt or similar nuisances resulting from earthwork operations.

E. Surface drainage provisions shall be made during the period of construction in a manner to avoid creating a nuisance to adjacent areas.

F. The site and adjacent influenced areas shall be watered as required to suppress dust nuisance.

1.5 EXISTING SITE CONDITIONS

Piling Contractor shall inspect the site and related conditions prior to commencing his/her portion of the work. If unshown active utilities are encountered during the work, the Architect shall be promptly notified for instructions. Failure to notify will make the Contractor liable for damage to these utilities arising from Contractor's operations subsequent to the discovery of such unshown utilities.

1.6 GEOTECHNICAL ENGINEERING REPORT

A. A *Geotechnical Engineering Report* (WKA No. 9955.01, dated January 27, 2014), has been prepared by Wallace - Kuhl & Associates, Geotechnical Engineers of West Sacramento, California; telephone (916) 372-1434; facsimile (916) 372-2565. That report is available for review at the office of Wallace - Kuhl & Associates.

B. The Piling Contractor shall submit in writing to the Architect and/or Structural Engineer, all applicable information as listed in Subsection 1.7 - Submittals for review and approval, in addition to the above experience record.

C. The Owner does not guarantee that the information contained in the Geotechnical Engineering Report is correct nor that the conditions revealed at the actual boring locations will be continuous over the entire site. This report was prepared for purposes of design only. Making the report available to contractors shall not be construed in any way as a waiver of this position. The Piling Contractor shall be responsible for any conclusions he/she may draw from this report. Should the Contractor prefer not to assume such risk, he/she is under obligation to employ their own experts to analyze available information and/or to make their own tests upon which to base their conclusions.

1.7 SUBMITTALS

Submit the following according to Conditions of the Construction Contract and Division 1 Specifications, for Owner's approval.

A. Shop Drawings: Shall clearly indicate but not be limited to:

1. Description of the pile drilling and grouting equipment and procedures to be utilized in installations.
2. Proposed pile grout design mix and description of materials to be used in sufficient detail to indicate their compliance with the specifications and either;
    a. Laboratory tests of trial mixes made with the proposed mix, or
    b. Laboratory tests of the proposed mix used on previous projects.
3. A pile layout plan referenced to the structural plans including a numbering system capable of identifying each individual pile, and indicating pile cutoff elevations.
4. A dimensioned sketch of the pile load test arrangements, including sizes of primary members, data on testing and measuring equipment including required jack and gauge calibrations, load cell and professional engineer seal certifying the adequacy of the reaction frames.
5. Fabrication and installation schedule covering test pile installation, pile testing, and production pile installation, with excavation schedule for pile cap and finished subgrades by area.
6. Qualifications of pile installation construction personnel, supervisor, and technician.

B. Records

1. The Contractor shall submit a pile design report indicating construction methods and materials that will be utilized to install piles of the specified compression and tension capacity, meeting the criteria of this specification and the Contract Drawings. The report shall be prepared and sealed by a Professional Engineer licensed in the state of California.
2. The Contractor shall provide a Technician for each pile rig responsible for observing the auger construction, grout batching, and grouting operations and preparing installation records. The Contractor's inspector shall submit an installation record for each pile not later than two (2) days after installation is completed. The report shall include but not be limited to:
    a. Project name and number
    b. Name of contractor
    c. Pile number
    d. Pile location, date and time of installation
    e. Design pile capacity, compression or tension
    f. Pile diameter
    g. Tip elevation
    h. Cut off elevation
    i. Elevation of butt
    j. Drilling elevation
    k. Rate of advancement of auger and rotation speed
    l. Quantity of grout placed as compared to the theoretical volume for each pile, in five-foot (5') depth increments, and total for pile
    m. Grout pressures
    n. Pile reinforcing steel
    o. Grout flow cone test report
    p. Any unusual occurrences observed during pile installation, and pile deviation from vertical

3. The grout quantity shall be determined by recording grout pump displacement or by other acceptable means; the pile installation record shall reveal the observed measure and quantity.
4. Load test reports shall be in accordance with the applicable ASTM Standards.
5. Grout compression test reports.

C. Hazardous Materials Notification: In the event no alternative product or material is available that does not contain asbestos, PCB or other hazardous materials as determined by the Owners' Authorized Representative, a "Material Safety Data Sheet" (MSDS) equivalent to OSHA Form 20 shall be submitted for that proposed product or material prior to installation.

D. Asbestos and PCB Certification: After completion of installation, but prior to Substantial Completion, Contractor shall certify in writing that products and materials installed, and processes used, do not contain asbestos or polychlorinated biphenyls (PCB), using format in Section ____/Closeout Procedures.

1.8 DELIVERY, HANDLING, STORAGE

Comply with General Conditions and Section 01600/Product Requirements.

1.9 WARRANTY

Comply with General Conditions and Section ____/Product Requirements.

PART 2: PRODUCTS

2.1 QUALITY ASSURANCE

A. The work of this section shall be performed by a company specialized in auger cast pile work with a minimum of five (5) years of documented successful experience, and shall be performed by skilled workers thoroughly experienced in the necessary crafts. Contractor shall submit evidence of successful installation of augered cast-in-place piles under similar job and subsurface conditions, including a job supervisor who shall have a minimum of three (3) years of method specific experience.

B. Work shall comply with all Municipal, State and Federal regulations regarding safety, including the requirements of the Williams-Steiger Occupational Safety and Health Act of 1970.

2.2 MATERIALS

A. Portland Cement: conforming to ASTM C150.

B. Mineral Admixture: Mineral admixture, if used, shall be fly ash or natural pozzolan which possesses the property of combining with the lime liberated during the process of hydration of Portland cement to form compounds containing cementitious properties, conforming to ASTM C618, Class C or Class F.

C. Fluidifier conforming to ASTM C937, except that expansion shall not exceed 4%.

D. Water: Potable, fresh, clean and free of sewage, oil, acid, alkali, salts or organic matter.

E. Fine Aggregate: Conforming to ASTM C33.

F. Grout Mixes:

1. The grout shall consist of Portland cement, sand and water, and may also contain a mineral admixture and approved fluidifier.
   a. The components shall be proportioned and mixed to produce a grout capable of maintaining the solids in suspension, which may be pumped without difficulty and which will penetrate and fill open voids in the adjacent soils.
   b. These materials shall be proportioned to produce a hardened grout which will achieve the design strength within twenty-eight (28) days.
   c. The design grout strength at twenty-eight (28) days for this project shall be a minimum four thousand pounds per square inch (4000 psi).
2. All materials shall be accurately measured by volume or weight as they are fed to the mixer.
   a. Time of mixing shall be not less than one minute at the site.
   b. If agitated continuously, the grout may be held in the mixer or agitator for a period not exceeding two and one-half (2½) hours at grout temperatures below seventy degrees Fahrenheit (70°F) and for a period not exceeding one hundred degrees Fahrenheit (100°F).
   c. Grout shall not be placed when its temperature exceeds one hundred degrees Fahrenheit (100°F).
3. Protect grout from physical damage or reduced strength, which could be caused by frost, freezing actions or low temperatures or from damage during high temperatures in accordance with ACI 305/306.
4. The grout shall be tested by making a minimum of six, two-inch (2") diameter by four-inch (4") tall cylinders for each day during which piles are placed.
   a. A set of six (6) cylinders shall consist of two (2) cylinders tested at seven (7) days, and two (2) cylinders tested at twenty-eight (28) days. Two (2) cylinders shall be held in reserve.
   b. Test cylinders shall be cured and tested in accordance with ASTM C109.
   c. Cylinder specimens shall be cast and cured in accordance with ASTM C31.
   d. Cylinder specimens may be restrained from expansion as described in ASTM C942.
5. Test the flow of grout for each pile and batch of grout. Maintain grout fluidity between fifteen (15) and twenty-five (25) seconds through a three-quarters inch (¾") diameter grout cone.

G. Steel Reinforcing:

1. Minimum reinforcing steel assemblies are shown on the Contract Drawings. Assemblies shall be detailed and fabricated in accordance with the manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315).
2. Reinforcing shall conform to the requirements of ASTM A615, Grade 60.
3. All reinforcing bar shall be epoxy coated, including bars installed for contractor convenience. Wire ties do not require epoxy coating.
4. Contractor shall provide labor, materials, and method for coating cut ends and repairing holidays in epoxy coating.
5. Acceptable materials and methods shall be provided to facilitate proper centering of all steel reinforcing installed.
6. Bars may be bent in place, provided epoxy coating at all bends is inspected, flaked coating is removed by wire brush, and holidays in coating are repaired.

7. A corrugated metal pipe sleeve shall be provided for each pile equal to the diameter of the auger, to define the pile butt and permit cut-off to specified elevations.

2.3 EQUIPMENT

A. Augering Equipment:
1. The auger flighting shall be continuous from the auger head to the top of auger without gaps or other breaks.
2. The auger flighting shall be uniform in diameter throughout its length and shall be the diameter specified for the piles less a maximum of three percent (3%). The hole through which the grout is pumped during the placement of the pile shall be located at the bottom of the auger head below the bar containing the cutting teeth.
3. Augers over forty feet (40') in length shall contain a middle support guide.
4. The piling leads shall be prevented from rotating by a stabilizing arm or by firmly placing the bottom of the leads into the ground or by some other acceptable means.
5. Leads shall be marked at one-foot (1') intervals to facilitate measurement of auger penetration.
6. Auger hoisting equipment shall be provided that will enable the auger to be rotated while being withdrawn.

B. Mixing and Pumping Equipment:
1. Only approved pumping and mixing equipment shall be used in the preparation and handling of the grout.
   a. Provide a screen to remove over-size particles at the pump inlet.
   b. All oil or other rust inhibitor shall be removed from mixing drums and grout pumps before each use.
   c. All materials shall be such as to produce a homogeneous grout of the desired consistency and strength.
2. The grout pump shall be a positive displacement pump capable of developing displacement pressures at the pump of three hundred fifty pounds per square inch (350 psi) or higher.
   a. The grout pump shall be provided with a pressure gauge in clear view of the equipment operator.
   b. The grout pump shall be calibrated at the beginning of the work and periodically during the work to determine the volume of grout pumped per stroke, under operating pressure.
   c. A positive method for automatic counting of grout pump strokes shall be provided. Such methods may include digital or mechanical stroke counters or other acceptable methods.
   d. A second pressure gauge, if required, shall be provided close to the auger rig where it can be readily observed by the inspector, if required.

PART 3: EXECUTION

3.1 EXAMINATION

A. The Contractor is responsible for supporting pile drilling equipment and concrete grout batching and delivery equipment. Equipment shall be supported on timber mats or gravel fill work platforms, if necessary for safety and stability, and to prevent damage.

B. The Contractor shall examine the areas and evaluate conditions under which piles are to be installed and shall include measures for the proper and timely completion of the work in the construction methods and pile design.

3.2 AUGER CAST PILE SYSTEM DESCRIPTION

A. Augered Pressure Grouted Piles

1. Pressure grouted piles shall be made by drilling a continuous-flight, hollow-shaft auger into the ground to the design pile depth, or until refusal criteria is satisfied. The volume of soil extracted shall not be greater than the volume of the steel auger stem inserted.
2. Grout shall be injected through the auger shaft as the auger is being withdrawn. First develop a five-foot (5') plug at the bottom of the auger flights, then inject sufficient grout volume to fill the augered hole one point one five (1.15) times its neat dimension, or more. Grout volumes shall be logged by depth during withdrawal.
3. Post-grouting through a special grout tube for capacity increase is permitted, given these methods are used in the test piles, and consistently throughout the entire work for this project. Post-grouting may be used for compression and tension capacity. Post-grout pressures must be sufficient to open grout portals and cause fracture and flow. Grout volumes and pressures shall be recorded and used as a measure to demonstrate pile compliance with the design and pile load test criteria.

B. Augered Displacement Pressure Grouted Piles

1. Augered Displacement Pressure Grouted piles shall be made by rotating a specialized auger capable of displacing soil surrounding the auger, with minimal soils returned to the ground surface to reach the design pile depth, or until specified refusal criteria is satisfied.
2. Grout shall be injected through the auger shaft as the auger is being withdrawn in such a way as to exert a positive upward grout pressure on the auger, as well as a positive lateral pressure on the soil surrounding the pile.

C. Alternatives

1. Alternative pile types which meet the compression and tension pile criteria given on the drawings may be substituted for augered pressure-grouted pile systems described in this Section.
2. Alternative pile installation systems must be capable of achieving the specified compression and tension, and shall provide a working lateral capacity of eight tons (8).

3.3 PILE DESIGN

A. The ultimate capacity of eighteen inch (18") diameter compression piles shall be greater than two hundred forty tons (240) in axial compression and greater than sixty (60) tons in axial tension or the ultimate capacity of twenty four inch (24") diameter compression piles shall be greater than four hundred twenty (420) tons in axial compression and greater than one hundred five (105) tons in axial tension. Both tension and compression piles shall achieve an ultimate lateral capacity of five (5) tons for eighteen inch (18") diameter piles or ten (10) tons for twenty four inch (24") diameter piles. The allowable design capacities of all piles shall be determined by dividing the ultimate capacity by the appropriate factor of safety as provided in the Geotechnical Engineering Report. Load Testing performed under Part 3.4 of this section shall confirm the ultimate capacity of the piles.

B. Pile design shall be performed by the Contractor and demonstrated by load test before installation of production piles. All piles shall meet the criteria specified on the Contract Drawings.

C. The design shall be described in a pile design report. This report shall indicate variances, if any, from the reinforcing steel specified or the requirements of this section, and shall demonstrate that the design meets or exceeds the specified performance in tension, compression, and bending. The Contractor shall submit design calculations for the proposed piles demonstrating compression and tensile capacity.

3.4 LOAD TESTING

A. Pre-construction Pile Load Tests:

1. Install and test one (1) compression pile, one (1) tension pile, and one (1) lateral load test pile, at the locations shown on the plans or approved alternate location to verify the construction methods and pile capacity. Test piles and reaction piles shall be installed outside of pile cap locations.
2. The Contractor shall provide complete testing materials and equipment as required, install test and reaction piles and perform the load tests only in the presence of the Owner.
3. The pile test reaction frame shall be capable of safely sustaining two hundred fifty (250) tons in axial compression and one hundred (100) tons in axial tension (uplift) for eighteen inch (18") diameter piles or four hundred thirty (430) tons in axial compression and one hundred ten (110) tons in axial tension (uplift) for eighteen inch (24") diameter piles.
4. Preconstruction Pile Load tests shall be performed using ASTM's Quick Test Methods.
5. One successful compression pile load test shall be performed in accordance with ASTM D1143.
6. One successful tension pile load test shall be performed in accordance with ASTM D3689.
7. One lateral pile load test to five (5) tons for eighteen inch (18") diameter piles or ten (10) tons for twenty four inch (24") ultimate load shall be performed in accordance with ASTM D3966.

3.5 INSTALLATION

A. Tolerance

1. Piles shall be located where shown on drawings or where otherwise directed by the Engineer.
    a. Pile centers shall be located to an accuracy of three inches (±3").
    b. Vertical piles shall be plumb within two percent (2%).
    c. Battered piles shall be installed to within four percent (4%) of the specified batter as determined by the angle from horizontal.

B. Adjacent Piles

1. Adjacent piles within ten feet (10'), center-to-center, shall not be installed within twenty-four (24) hours of each other.
2. Within pile caps, piles adjacent within four (4) pile diameters center-to-center, shall not be installed within twenty-four (24) hours of each other.

C. Installation Procedure

1. The length and drilling criteria of production piles will be as defined in the Contractor's design and as demonstrated by the successful pile load tests. Advance and rotate the auger at a continuous rate that prevents removal of excess soil.
2. Stop advancement after reaching the required depth or refusal criteria.
3. The hole in the bottom of the auger shall be closed with a suitable plug while advancing into the ground. The plug shall be removed by grout pressure or mechanically with the reinforcing bar.
4. At the start of pumping grout, raise the auger from six inches (6") to twelve inches (12") and after the grout pressure builds up sufficiently, re-drill the auger to the previously established tip elevation.
5. Maintain a head of at least fifteen feet (15') of grout on the auger flighting above the injection point during auger withdrawal.
    a. Positive rotation of the auger shall be maintained at least until placement of the grout.
    b. Rate of grout injection and rate of auger withdrawal from the soil shall be coordinated so as to maintain at all times the minimum grout head.
    c. The total volume of grout shall be at least one hundred fifteen percent (115%) of the theoretical volume for each pile.
    d. After grout is flowing at the ground surface from the auger flighting, the rate of grout injection and auger withdrawal shall be coordinated so that there is a constant grout flow at the surface.
    e. If pumping grout is interrupted for any reason, the contractor shall reinsert the auger by drilling at least five feet (5') below the depth of the auger where the interruption occurred, and re-grout while withdrawing the auger from that depth.
6. If less than one hundred fifteen percent (115%) of the theoretical volume of grout is placed in any five foot (5') increment (until the grout head on the auger flighting reaches the ground surface), the pile increment shall be reinstalled by advancing the auger ten feet (10') or to the bottom of the pile if that is less, followed by controlled removal and grout injection.
7. Spoil material that accumulates around the auger during injection of the grout shall be promptly cleared away.
8. A steel corrugated metal pipe (CMP) sleeve shall be placed at the top of each pile to a depth of one and one half feet (1½') below the pile cutoff elevation.

D. Obstructions and Damaged Piles

1. If non-augerable material is encountered above the desired tip elevation, the pile shall be completed to the depth of the non-augerable material in accordance with these Specifications. Such short piles shall be included for payment, if completed and included within the foundation. If required by the Engineer, additional adjacent piles shall be placed. Additional piles shall also be included in the total number of piles for payment.
2. Damaged piles, and piles installed outside the required installation tolerances, will not be accepted.

3. Cut off and abandon rejected piles after installation, and replace with new piles. Cutoff shall be at a sufficient depth to avoid transfer of load from the structure to the abandoned pile.
4. Piles located within ten feet (10') of existing structures shall be installed in one continuous operation. Re-stroking piles during construction due to auger obstructions or difficulty in installation of reinforcement cages will not be allowed. The structural engineer shall be consulted in the event that replacement piles are required.

E. Cutting-Off
1. Adjust the tops of pile to the cut-off elevations where piles are constructed from a work platform above final subgrade, by removing fresh grout from the top of the pile after the CMP sleeve is in place.
2. Cut off hardened grout and the CMP shell down to final cutoff point after initial set has occurred for all piles in a single cap, or within 15 ft of any pile in a spaced pattern.

F. Disposal
1. The Contractor shall remove and dispose all spoils and grout off site.
2. The Contractor shall determine if any excavated material is contaminated, and if any contaminated material is encountered it shall be disposed of in a method acceptable to all governmental authorities having jurisdiction.

PART 4: MEASUREMENT AND PAYMENT

4.1 MEASUREMENT

A. Each compression pile and each tension pile successfully installed in accordance with the Contractor's design and using the methods and practices of the approved test piles, cut off at the proper elevation, including steel reinforcing, and all records and grout testing specified, shall be considered a single unit price item. Pile design, materials testing, and the Contractor's inspection are considered incidental to construction and shall not be separately measured for payment. Damaged piles and piles installed outside the required installation tolerances will not be measured for payment. Short piles caused by obstructions and meeting the requirements of Part 3.5D shall be measured for payment.

B. Each successful compression, tension and lateral pre-construction load test performed, including load frame and/or reaction piles, test pile, testing, and load test report, shall be considered a single unit price item.

C. Each successful compression, tension and lateral construction quick load test performed, including load frame and/or reaction piles, test pile, testing, and load test report, shall be considered a single unit price item.

4.2 PAYMENT

A. Each compression pile and each tension pile, approved and accepted by the Owner, shall be paid at the unit price indicated on the bid form.

B. Each successful pile load test, approved and accepted by the Owner, shall be paid at the unit prices indicated on the bid form.

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Wallace Kuhl & ASSOCIATES

Geotechnical Engineering Report

21st STREET AND CAPITAL AVENUE MIXED USE

WKA No. 9957.01

January 31, 2014

Prepared For:

LVP Revocable Trust

2020 L Street, 5th Floor

Sacramento, California 95811

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INTRODUCTION

We have completed a geotechnical engineering investigation for the proposed mixed use development northeasterly of the corner of 21st Street and Capitol Avenue in Sacramento, California. The purposes of our work have been to explore the existing site, soil and groundwater conditions beneath the proposed improvement areas and to provide geotechnical engineering conclusions and recommendations for the design and construction of the proposed improvements. This report represents the results of our work.

Work Scope

Our scope of work has included the following tasks:

1. site reconnaissance;
2. review of aerial photographs and available historical groundwater contour maps;
3. subsurface exploration, including the drilling and sampling of two (2) test borings to maximum depths of approximately 28 to 33 feet below the existing ground surface;
4. bulk sampling of near-surface soils;
5. laboratory testing of selected soil samples;
6. engineering analyses, and;
7. preparation of this report.

Our evaluation was performed in general accordance with our *Geotechnical Engineering Services Proposal* dated September 10, 2013.

Figures and Attachments

A Vicinity Map showing the location of the site is included as Figure 1. Figure 2 shows the approximate locations of the borings relative to existing site features. The Logs of Soil Borings are presented as Figures 3 and 4. An explanation of the symbols and classification system used on the logs appears on Figure 5. Appendix A contains general information regarding the field investigation, descriptions of the field exploration and laboratory testing programs, and the results of laboratory tests that do not appear on the Logs of Soil Borings. Appendix B contains guide specifications for construction of auger cast piles for use in preparing contract documents.

Proposed Development

We understand the project will consist of the design and construction of a new two-story, slab-on-grade mixed use development. Associated development is anticipated to consist of exterior concrete flatwork and underground utilities.

FINDINGS

Site Description

The project site is located on the east side of 21st Street between Capitol Avenue and the L Street Capitol Avenue Alley in Sacramento, California. The site is currently occupied by vacant property and asphalt concrete surface parking. Previous residential structures at the southwest end of the site were recently demolished in 2013. The site is bound to the south by Capitol Avenue; to the west by 21st Street; to the west by asphalt concrete parking, beyond which is a commercial building; and, to the north by an existing multi-story residential structure, beyond which is the L Street Capitol Avenue Alley.

Our review of historic aerial photographs indicates the east side of the site was previously covered in residential structures until it was used as asphalt concrete parking after 1960 and before 1968. The residential structures that were recently demolished appear to have occupied the site since at least 1937.

Subsurface Soil Conditions

Two (2) exploratory borings were performed on December 9, 2013 at the approximate locations indicated on Figure 2. The soil conditions at the borings generally consist of about 15 to 25 feet of relatively loose silt layers overlying about seven (7) to 13 feet of stiff clays with interbedded silt layers. The stiff clays are underlain by relative dense gravels extending to the explored 28 to 33 foot depths of the borings.

At the completion of our drilling activities, the test borings were grouted to the surface with a slurry of neat cement and water, as required by the permit issued by the County of Sacramento Environmental Management Department.

For soil conditions at the specific boring locations, please refer to the boring logs contained on Figures 3 and 4.

Groundwater

Groundwater was encountered about 20 feet below the ground surface at the boring locations during and immediately after the drilling operations. Based on our experience in the area, groundwater is anticipated to be as high as about 15 feet below the existing ground surface at the site.

CONCLUSIONS

Seismic Code Parameters – 2013 CBC/ASCE 7-10

We understand the design of the structures will be performed using the 2013 California Building Code (CBC). The 2013 edition of the CBC references American Society of Civil Engineers (ASCE) Standard 7-10 for seismic design. The following seismic parameters were determined based on the site latitude and longitude using the public domain computer program developed by the United States Geological Survey (USGS).

2013 CBC/ASCE 7-10 Seismic Design Parameters

Latitude: 38.5735° N Longitude: 121.4801° W	ASCE 7-10 Table/Figure	2013 CBC Table/Figure	Factor/ Coefficient	Value
Short-Period MCE at 0.2s	Figure 22-1	Figure 1613.3.1(1)	S_S	0.668 g
1.0s Period MCE	Figure 22-2	Figure 1613.3.1(2)	S_1	0.291 g
Soil Class	Table 20.3-1	Section 1613.3.2	Site Class	D
Site Coefficient	Table 11.4-1	Table 1613.3.3(1)	F_a	1.266
Site Coefficient	Table 11.4-2	Table 1613.3.3(2)	F_v	1.817
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equation 16-37	S_{MS}	0.845 g
	Equation 11.4-2	Equation 16-38	S_{M1}	0.530 g
Design Spectral Acceleration Parameters	Equation 11.4-3	Equation 16-39	S_{DS}	0.564 g
	Equation 11.4-4	Equation 16-40	S_{D1}	0.353 g

Latitude: 38.5735° N Longitude: 121.4801° W	ASCE 7-10 Table/Figure	2013 CBC Table/Figure	Factor/ Coefficient	Value
Seismic Design Category	Table 11.6-1	Section 1613.3.5(1)	Occupancy I to IV	D
	Table 11.6-2	Section 1613.3.5(2)	Occupancy I to IV	D

Liquefaction Potential

Liquefaction is a soil strength loss phenomenon that typically occurs in loose, saturated cohesionless sands as a result of strong ground shaking during earthquakes. The potential for liquefaction at a site is usually determined based on the results of a subsurface soil investigation and the groundwater conditions beneath the site. Hazards to structures associated with liquefaction include shallow and deep foundation bearing capacity failure, lateral spreading of soil, and differential settlement of soils below foundations, all of which can contribute to structural damage or collapse.

The soil conditions encountered at the borings indicates the site is primarily underlain by low to medium plasticity silts and clays below the groundwater, which are typically not susceptible to liquefaction. Therefore, it is our opinion that saturated, loose cohesionless soils likely do not exist in significant thickness beneath the groundwater, and therefore the potential for liquefaction of the soils occurring at the site is very low.

Bearing Capacity

Based upon our field and laboratory testing, it is our opinion that the loose, undisturbed native soils overlying the stiff clays and gravel layer are not capable of supporting the planned structures and associated improvements unless the structures are supported on an alternative foundation system, such as shallow foundations supported on an improved subgrade (i.e., excavation and recompaction of the soils as a uniform engineered fill to a specified depth or Geopier® rammed aggregate piers [RAPs]) or a deep foundation system consisting of driven, precast concrete piles (driven piles); drilled, auger cast-in-place piles; or drilled cast-in-place reinforced concrete piers. However, we anticipate noise and vibrations associated with the construction of driven piles at the site will exceed those typically tolerated for projects within close proximity to existing structures such as those adjacent to the site. Therefore, driven piles will not be considered for this project at this time due to noise pollution, disturbances due to vibrations, and other factors associated with construction of driven piles.

The selection of the most appropriate foundation system or systems will depend on the actual loads and configurations (i.e., above grade, below-grade, partially below-grade, etc.) of the structures, the acceptable amount of settlement for the structure, and the construction constraints (i.e., vibrations, noise, equipment access, etc.). A discussion of each foundation type is provided as follows.

Specific recommendations for the various foundation systems are provided in the Foundations section of this report.

Shallow Foundations Supported on Engineered Fill

Excavation and recompaction of the soils as a uniform engineered fill to a depth of five (5) feet below the bottom of the foundations or at least five (5) feet below the existing ground surface, whichever is deeper, is considered suitable for support of the proposed two-story building. Engineered fill should be placed and compacted in accordance with the recommendations of this report.

Shallow Foundations Supported on Geopier® RAPs

Based on the available information, we conclude that shallow foundations supported on an improved subgrade consisting of Geopier® RAPs would be appropriate for support of the proposed improvements. The Geopier® system uses a drilled shaft backfilled with compacted aggregate base to improve subgrade stability and reduce settlements within the treated area. The Geopier® system should be designed by a professional engineer in the State of California that is qualified and experienced in Geopier® rammed aggregate pier design.

Drilled Auger Cast-in-Place Piles

Based on the soil conditions encountered at the site, deep foundations consisting of auger cast-in-place (ACIP) piles extending into the relatively dense gravel layer encountered about 28 to 32 feet below the ground surface at the boring locations are considered feasible at the site. ACIP piles have been used as an alternative to driven piling to reduce detrimental vibration, noise, and other problems associated with driving piles, and can achieve similar bearing, uplift, and lateral resistance of the driven piles.

We anticipate total settlements on the order of ½-inch and differential settlements on the order of ¼-inch for ACIP pile foundations. A contingency plan for loading and off-hauling soil cuttings from the ACIP should be considered in the construction plans and schedule.

Drilled, Cast-in-Place, Reinforced Concrete Piers

Drilled, cast-in-place, reinforced concrete piers (drilled piers) could be used to support the structure. Drilled piers will likely extend below the groundwater table during construction and will require wet construction techniques (i.e., casing and/or drilling slurry). We anticipate drilled piers will extend about 25 to 30 feet below the existing ground surface based on the soil conditions encountered at the boring locations.

We anticipate total settlements on the order of ½-inch and differential settlements of ¼-inch. The use of drilled piers also would provide increased uplift and lateral resistance for the structure.

The construction costs, plan, and schedule should include loading and off-hauling soil cuttings from the drilled pier construction.

Soil Expansion Potential

The near-surface soils encountered at the borings generally consist of granular silts that are not considered expansive. Therefore, special reinforcement of foundations and floor slabs, or special moisture conditioning during site grading to resist or control soil expansion pressures, are not considered necessary on this project.

Pavement Subgrade Quality

Laboratory testing of bulk samples obtained at the site indicates the near-surface soils are relatively good quality materials for support of asphalt concrete and concrete pavements. A Resistance value (R-value) of 60 was obtained on a composite bulk soil samples obtained from the upper three feet of soil at boring location D2. The results of the R-value testing are included on Figure A3 attached.

Material Suitability

The existing on-site materials are considered suitable for use as engineered fill, provided they are free of significant quantities of organics, rubble and deleterious debris, and at a suitable moisture content to achieve the recommended compaction.

Soils beneath existing pavement and slab areas and irrigated areas will likely be at an elevated moisture content regardless of the time of construction and will require drying before compaction or use as fill.

Existing pavements and flatwork (asphalt concrete and concrete) within areas to be demolished, if any, may be broken up and pulverized for use as fill. Asphalt and Portland cement concrete rubble may be used as fill provided it is processed into fragments less than three inches in largest dimension, is mixed with soil to form a compactable mixture, and is approved by the Owner.

The existing aggregate base encountered below the asphalt concrete and concrete surfaces is considered suitable for reuse as engineered fill. Consideration may also be given to reusing the existing aggregate base as aggregate base or subbase. However, additional laboratory testing would be required to verify the material meets the requirements for Caltrans Class 2 aggregate base or subbase.

Excavation Conditions

Based on the information obtained during the field exploration and our local experience, we anticipate the soils at the site will be readily excavatable with conventional earthmoving and trenching equipment. However, larger equipment may be required to remove existing below-grade structures at the site from previous developments and the existing structures (e.g., previous foundations, concrete slabs, etc.). Based on the results of our subsurface exploration, the soils across the site may be classified as Type B soils in accordance with the Occupational Safety and Health (OSHA) classification system.

In general, we anticipate the on-site soils will likely remain stable at near-vertical inclinations without significant caving for relatively short periods (i.e., less than one day) during utility and foundation construction. However, excavations extending into saturated and/or disturbed soils will likely require excavation bracing or shoring to control sloughing and caving for utilities and casing will be required for RAP and/or drilled pier excavations. Excavations deeper than five feet should be sloped or braced in accordance with current OSHA regulations.

Temporarily sloped excavations should be constructed no steeper than a one horizontal to one vertical (1:1) inclination. Temporary slopes likely will stand at this inclination for the short-term duration of construction, provided significant pockets of loose and/or saturated granular soils are not encountered that could slough into excavations.

The contractor must provide a safely sloped excavation or an adequately constructed and braced shoring system in accordance with federal, state and local safety regulations for individuals working in an excavation that may expose them to the danger of moving ground. If material is stored or heavy equipment is operated near an excavation, stronger shoring must be used to resist the extra pressure due to the superimposed loads.

Groundwater

Based on our subsurface exploration and review of groundwater information in the vicinity of the site, a permanent groundwater level of about 15 feet should be used in design of the proposed structure. The permanent groundwater table should not be a significant factor in site development for excavations less than about 15 feet below the existing ground surface. However, it is likely that perched groundwater may be encountered in excavations from rainfall, surface run-off, irrigation, or seepage from perched groundwater sources, especially if construction begins in the winter and early spring months.

For excavations extending less than about 15 feet below the existing ground surface standard sump pit and pumping procedures should be adequate to control localized groundwater. If Geopier® RAPs, ACIP piles, or drilled piers are used for foundation support, the RAP or pile/pier contractor should provide proper equipment and materials to handle the anticipated groundwater depths.

Dewatering of excavations deeper than about 15 feet below the existing ground surface should be anticipated, although the groundwater elevation will vary depending on seasonal rainfall. Temporary dewatering will be necessary to maintain a relatively dry excavation and to limit disturbances to the subgrade at the bottom of the excavation. The groundwater should be temporarily lowered to at least two feet below the bottom of excavations. The spacing interval(s) and depth for dewatering operations will depend on the rate and volume of groundwater flow experienced and should be determined in the field by the dewatering contractor. Note that the dewatering design should take into account the effect dewatering operations will have on the adjacent improvements.

Groundwater levels should be expected to fluctuate throughout the year based on variations in precipitation, temperature, evaporation, run-off, and other factors. The groundwater levels discussed herein, and indicated on the boring logs, represent the conditions at the time the measurements were obtained. The actual groundwater levels at the time of construction may vary.

Seasonal Water

Infiltrating surface run-off water from seasonal moisture during the winter and spring months will create saturated surface soil conditions. It is probable that grading operations attempted following the onset of winter rains and prior to prolonged drying periods will be hampered by high soil moisture contents. Such soils, intended for use as engineered fill, will require a prolonged period of dry weather and aeration or chemical treatment to reach a moisture content suitable for proper compaction.

In addition, soils located beneath existing pavements, slabs, and flatwork, will likely be at elevated moisture contents regardless of the time of year of construction and also require drying. Wet soils should be anticipated and considered in the construction schedule for this project.

Preliminary Soil Corrosion Potential

A sample of near-surface soil was submitted to Sunland Analytical Lab for testing to determine pH, chloride and sulfate concentrations, and resistivity to help evaluate the potential for corrosive attack upon buried structures. Results of the soil corrosivity tests are summarized below; copies of the test results are attached as Figure A5.

SUMMARY OF CORROSION TEST RESULTS						
Sample Location	Test Depth (feet)	USCS Soil Type	pH	Chloride Content (ppm)	Sulfate Content (ppm)	Resistivity (ohm-cm)
D1	1 to 3	ML	8.19	12.9	26.7	4290

The California Department of Transportation Corrosion Technology Section, Office of Materials and Foundations, *Corrosion Guidelines Version 1.0, September 2003*, considers a site to be corrosive to foundation elements if one or more of the following conditions exists for the representative soil and/or water samples taken: has a chloride concentration greater than or equal to 500 ppm, sulfate concentration greater than or equal to 2000 ppm, or the pH is 5.5 or less. The corrosivity test results suggest that the site soils are not highly corrosive to exposed reinforced concrete. The low resistivity may indicate an increased potential for corrosion of buried metal. Table 4.3.1 – *Requirement for Concrete Exposed to Sulfate-Containing Solutions*, American Concrete Institute (ACI) 318, Section 4.3, as referenced in section 1904A.3 of the 2007 CBC, indicates the sulfate exposure for the samples tested is *Negligible*. Ordinary Type I-II Portland cement is considered suitable for use on this project, assuming a minimum concrete cover is maintained over the reinforcement.

Wallace-Kuhl & Associates are not corrosion engineers. Therefore, if it is desired to further define the soil corrosion potential at the site a corrosion engineer should be consulted.

RECOMMENDATIONS

General

The recommendations presented below are appropriate for typical construction in the late spring through fall months. The on-site soils likely will be saturated by rainfall in the winter and early spring months, and will not be compactable without drying by aeration or chemical treatment to dry the soils. Should the construction schedule require work during wet conditions, additional recommendations can be provided, as conditions dictate.

Soils under existing pavements or slabs and irrigated areas will be wet regardless of the time of year of construction.

Site preparation should be accomplished in accordance with the provisions of this report and the appended guide specifications. A representative of the Geotechnical Engineer should be present during site grading to evaluate compliance with our recommendations and the guide specifications. The Geotechnical Engineer of Record referenced herein should be considered the Geotechnical Engineer that is retained to provide geotechnical engineering observation and testing services during construction.

Site Preparation

Based on the relatively loose nature of the near-surface soils, we conclude the existing soils at the site are not considered suitable for shallow foundation support of the proposed two-story structure unless the subgrade soils are improved or the structure is supported on a deep foundation system. Therefore, site preparation will depend on the specific foundation system chosen. A discussion of the site preparation required for each support option is provided below.

Regardless of the support option chosen, the site should be cleared of existing pavements, flatwork, below-grade structures, vegetation, debris, and other deleterious materials to expose undisturbed native soils. Where practical, the clearing should extend a minimum of five feet beyond the limits of the proposed structural areas of the site. Underground utilities within the proposed construction areas should be completely removed, rerouted, or properly abandoned (i.e., fully grouted provided the abandoned utility is situated at least 2½ feet below the final subgrade level to reduce the potential for localized "hard spots"). Depressions resulting from removal of underground utilities should be cleaned of loose soil and properly backfilled in accordance with the recommendations of this report.

The existing pavements and flatwork (asphalt concrete and concrete) that are not incorporated into the new design should be broken up and removed from the site. Pulverized asphalt and Portland cement concrete rubble may be used as fill below the structures and pavements provided they are processed into fragments less than three inches in largest dimension and mixed with soil to form a compactable mixture.

Soils beneath existing pavement and slab areas and irrigated areas will likely be at an elevated moisture content regardless of the time of construction and will require drying before compaction or use as fill.

Building Pads Supporting Shallow Foundations – Excavation and Recompaction

Shallow foundations are considered suitable for support of the structure if the building pad, and the area extending at least five feet horizontally beyond the proposed exterior edge of foundations, *are excavated to a depth of at least five feet below the bottom of the foundations or at least five feet below the existing ground surface, whichever is deeper*. Following excavation operations, areas to receive fill should be scarified to a depth of at least 12 inches, moisture conditioned to at least the optimum moisture content, and compacted to at least 90 percent relative compaction. Relative compaction should be based on the maximum dry density as determined in accordance with the ASTM D1557 Compaction Test. Compaction operations should be performed in the presence of our representative who will evaluate the performance of the subgrade under compactive load and identify loose or unstable soils that could require additional excavation and/or compaction.

If the exposed subgrade is disturbed and/or wet, a layer of coarse crushed aggregate may be required at the base of the overexcavation to provide a stable working surface on which to place and compact backfill. The crushed aggregate should be placed in lifts no greater than 1-foot in thickness and then worked into the subgrade with the back-end of the backhoe bucket. The required thickness of this stabilization layer will depend on the severity of the disturbed condition. If the crushed aggregate stabilization layer exceeds 9 inches in thickness, we recommend an additional layer of Class 2 aggregate base be used to cap the coarse aggregate to reduce the potential for migration of overlying sand fill into the voids of the coarse aggregate.

Alternatively, consideration may be given to placing a layer of geogrid reinforcement (Tensar® BX 1200 or better) across the entire excavation (i.e., the entire building footprint plus five feet beyond the outer edges of the exterior foundations). The geogrid should be covered with a 6-inch thick lift of an approved granular, graded, compactable import soil. Class 2 aggregate base or Class 3 aggregate subbase (*Caltrans Standard Specifications*) compacted to at least 90 percent relative compaction at no less than the optimum moisture content are considered suitable for this purpose.

The excavation should be restored to grade with engineered fill in accordance with the recommendations provided in the Engineered Fill Construction section of this report.

Compaction operations should be performed in the presence of the Geotechnical Engineer's representative who will evaluate the performance of the subgrade under compactive load and identify loose or unstable soils that could require additional subgrade preparation.

Building Pads Supporting Shallow Foundations – Geopier® Rammed Aggregate Piers

An alternative to excavation and recompaction of soils beneath the building pad would be to use a ground improvement system consisting of Geopier® RAPs. The Geopier® system uses a drilled shaft backfilled with compacted aggregate base to improve subgrade stability and reduce settlements within the treated area. The Geopier® system should be designed by a professional engineer in the State of California that is qualified and experienced in Geopier® rammed aggregate pier design.

Building Pads Supported on a Deep Foundation

If a deep foundation system will be used for building support, the building pad areas should be scarified to a depth of at least 12 inches, moisture conditioned to at least the optimum moisture content and compacted to at least 90 percent of the ASTM D1557 maximum dry density. Any construction debris or subsurface structures encountered during excavation or cross-ripping activities should be removed. The excavation should be restored to grade with engineered fill compacted in accordance with the recommendations provided in the Engineered Fill Construction section of this report.

Pavements

Regardless of the foundation system chosen, we recommend pavement areas be scarified to a depth of at least 12 inches, moisture conditioned to at least the optimum moisture content and compacted to at least 90 percent of the ASTM D1557 maximum dry density. Any construction debris or subsurface structures encountered during excavation or scarification activities should be removed. The excavation should be restored to grade with engineered fill compacted in accordance with the recommendations provided in the Engineered Fill Construction section of this report.

Engineered Fill Construction

Any fill placed within the construction area should be an approved material, free of significant quantities of organics, oversized rubble, or other deleterious materials. The fill should be spread in level layers not exceeding nine inches in loose thickness and compacted to a minimum of 90 percent of the maximum dry density. Maximum dry densities shall be determined in accordance with ASTM D1557.

Engineered fill should be moisture conditioned to at least the optimum moisture content and maintained in that condition.

The on-site soils encountered at the boring locations are considered suitable for use as engineered fill provided they are free of rubble and organic concentrations and are at a compactable moisture content. Imported fill should be an approved compactable granular material, have an Expansion Index of 20 or less, a Resistance value of at least 30 when used within the upper three feet of pavement subgrades, and be free of particles larger than three inches in maximum dimension. The contractor also should supply appropriate documentation for imported fill materials indicating the materials are free of known contamination and have corrosion characteristics within acceptable limits. Our firm must approve import material before being transported to the project site.

The upper six inches of pavement subgrade should be moisture conditioned to at least the optimum moisture content and compacted to no less than 95 percent relative compaction, regardless of whether final subgrade is achieved by excavation, filling or left at existing grade. Final pavement subgrade processing and compaction should be performed after completion of underground utilities and must be stable under construction traffic prior to aggregate base placement.

Permanent excavation and fill slopes should be constructed no steeper than two horizontal to one vertical (2:1), and should be vegetated as soon as practical following grading to minimize erosion. Slopes should be over-built and cutback to design grades and inclinations.

Site preparation should be accomplished in accordance with the recommendations of this report. We recommend the Geotechnical Engineer's representative be present during site preparation and all grading operations to observe and test the fill to verify compliance with the recommendations of this report and the job specifications.

Utility Trench Backfill

Bedding and initial backfill for utility construction should conform with the pipe manufacturer's recommendations and applicable sections of the governing agency standards. General trench backfill should consist of engineered fill backfilled in maximum nine-inch thick loose lifts with each lift compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557. Utility trench backfill within the upper six inches of the final subgrade within pavement areas should be compacted to at least 95 percent of the maximum dry density.

We recommend that all underground utility trenches aligned nearly parallel with existing or new foundations be at least five feet from the foundations, wherever possible. If this is not practical, the trenches should not encroach on a zone extending at a one horizontal to one vertical (1:1) inclination below the foundations.

It is likely that materials excavated from trenches will be at elevated moisture contents and will require significant aeration or a period of drying to reach a compactable moisture content. We recommend bid documents contain a unit price for the removal and drying of saturated soils, or replacement with approved import soils.

Foundation Design Alternatives

We recommend that our office be given the opportunity to review final grading plans, foundation plans and specifications to determine if the intent of our recommendations has been properly implemented into those documents.

The proposed structure may be supported upon continuous and/or isolated spread foundations on an improved subgrade (i.e., properly placed and compacted engineered fill or a Geopier® RAP improved subgrade) or a deep foundation system consisting of drilled ACIP piles or drilled piers. Alternative foundations may be considered at the site and will be evaluated on a case-by-case basis.

Recommendations for each type of foundation system have been provided. Combination foundation systems (i.e. shallow foundations on an improved subgrade used with deep foundations) may be acceptable; however, the structure must be designed to accommodate some differential settlement due to the varying support characteristics of the foundations and elastic properties of various bearing strata. The intent of this recommendation is to minimize differential settlement between the two foundation types.

Our recommendations for shallow spread foundations on an improved subgrade, drilled ACIP piles, and drilled piers are provided in the following sections.

Shallow Foundations on Engineered Fill

Continuous and/or isolated spread foundations bearing on properly prepared engineered fill should extend at least 18 inches below the lowest adjacent soil grade of the properly prepared building pad. For this project, the pad soil grade is the surface on which capillary break materials are placed. Continuous foundations should be at least 12 inches wide; isolated spread foundations should be at least 18 inches wide. Foundations so established may be sized for maximum allowable soils bearing pressures of 3000 psf for dead plus live loads and 4000 psf for all loads, including wind or seismic forces. The weight of foundation concrete extending below adjacent soil grade may be disregarded in sizing computations.

We recommend that all foundations be reinforced to provide structural continuity, reduce cracking and permit spanning of local soil irregularities. The project structural engineer should determine foundation reinforcement. However, as a minimum, we recommend continuous foundations contain at least two No. 4 reinforcing bars, placed one each near the top and bottom of the foundation.

Resistance to lateral foundation displacement for conventional foundations may be computed using an allowable friction factor of 0.30, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an allowable passive earth pressure of 300 psf per foot of depth. These two modes of resistance should not be added unless the frictional value is reduced by 50 percent since full mobilization of these resistances typically occurs at different degrees of horizontal movement.

We estimate total settlement for shallow footing foundations using the recommended maximum net allowable bearing pressure presented above, should be one inch or less. Differential settlements are estimated to be about one-half the total settlement. These settlement estimates are based on the available boring information, our experience with similar structures and soil conditions, and field verification of suitable bearing soils by our firm during foundation construction.

Shallow Foundations on Geopier® Rammed Aggregate Piers

We anticipate a Geopier® RAP system could provide adequate support for the proposed structure supported on continuous and/or isolated spread foundations or a mat foundation. A qualified RAP contractor licensed in the State of California should be contacted directly to provide final recommendations for the Geopier® RAP system, including allowable capacities and settlements.

Continuous and/or isolated spread foundations bearing on a Geopier® RAP improved subgrade should extend at least 18 inches below the lowest adjacent soil grade of the structure pad. For this project, the pad subgrade is the surface on which aggregate materials (i.e., aggregate base below slab areas of the structures or capillary break materials within proposed building areas) are placed. Isolated spread foundations should be at least 18 inches wide.

Preliminary design information indicates allowable rammed aggregate pier capacities of 85 kips and a bearing capacity of 6000 psf for dead plus live load can be achieved on Geopier® RAPs. The RAP layout and final bearing pressures and cell capacities will depend on the actual loading conditions for each structure and should be determined by the RAP designer and should include an appropriate factor of safety. The weight of foundation concrete extending below adjacent soil grade may be disregarded in sizing computations.

Uplift resistance can be provided using ground improvement equipped with a steel uplift anchor and can provide about 35 kips of allowable uplift.

We recommend that all foundations be reinforced to provide structural continuity, reduce cracking and permit spanning of local soil irregularities. The project structural engineer should determine final foundation reinforcement. However, as a minimum, we recommend continuous foundations contain at least four No. 4 reinforcing bars, placed two each near the top and bottom of the foundation.

Preliminary resistance to lateral foundation displacement for conventional foundations supported on RAPs may be computed using an allowable friction factor of 0.45, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an allowable passive earth pressure of 350 psf per foot of depth. These two modes of resistance should not be added unless the frictional value is reduced by 50 percent since full mobilization of these resistances typically occurs at different degrees of horizontal movement.

Auger Cast-in-Place (ACIP) Concrete Piles

The proposed structure also may be supported upon ACIP concrete piles. ACIP concrete piles are installed using special equipment equipped with hollow-stem augers. Once the pile hole has been drilled, grout/concrete is injected under pressure through the auger to displace the soil and provide positive contact with the surrounding soils. Reinforcement is placed into the grouted shaft after withdrawal of the auger.

Piles for the structure should extend to a minimum of approximately two feet into the relatively dense gravels, which were encountered at depths of about 28 to 32 feet below the ground surface at the boring locations. Drilled ACIP concrete piles may be designed utilizing the following maximum allowable loads per pile with appropriate factor of safety (F.S.) as summarized in the table below as follows:

ALLOWABLE ACIP PILE CAPACITIES					
Loading Conditions		18-inch Diameter		24-inch Diameter	
		Allowable Pile Capacity (tons)	Ultimate Pile Capacity (tons)	Allowable Pile Capacity (tons)	Ultimate Pile Capacity (tons)
Axial Compression	DL (F.S. = 3)	80	240	140	420
	DL + LL (F.S. = 2)	120	240	210	420
	Total Load (F.S. = 1.5)	160	240	280	420
Axial Uplift (Tension)	Total Load (F.S. = 1.5)	40	60	70	105

Reductions in pile capacity for consideration of group action are unnecessary, provided piles are spaced no closer (center-to-center) than three times the diameter of the pile.

The indicated uplift pile capacity is based upon the assumption that the piles will be properly reinforced to transfer pullout forces to the pile tip.

Lateral loading information was not available at the time this report was prepared. The lateral resistance of individual piles and the passive resistance of the pile cap against the soil can be combined to provide lateral resistance. For preliminary design purposes, 18-inch ACIP piles can be assumed to provide an allowable lateral resistance of five (5) tons and 24-inch ACIP piles can be assumed to provide an allowable lateral resistance of 10 tons. Both lateral resistance values are based on a pile deflection of one-inch. Resistance to lateral loads for ACIP piles can be determined and presented in a supplemental report using a lateral pile analysis program when final size design information is known and if required to further aid in the structural design.

The weight of pile cap concrete extending below grade and the weight of each pile may be disregarded in determinations of the net compressive load transmitted to the supporting soil.

Concurrent lateral resistance derived in friction between the slab and the supporting subgrade layer may be computed using an allowable friction factor of 0.30 at the interface between the slab and the subgrade.

A pile load test program will be necessary to determine the correct length of the ACIP piles to achieve the specified capacities. Additional load testing could be performed during construction, where as-built pile dimensions differ from the recommended dimensions, which could result from refusal to auger penetration in denser/stiffer soils beneath this site.

Drilled Cast-in-Place Concrete Piers

Drilled, cast-in-place piers (drilled piers) may be used to support the proposed structure. Drilled piers should be at least 24 inches in diameter and extend to at least 25 feet below the existing ground surface into the relatively stiff clays encountered about 15 to 23 below the ground surface at the boring locations. Piers so established may be designed based on an allowable end bearing capacity of 4000 psf for dead plus live loads with a 1/3 increase for short-term effects of wind or seismic forces. We recommend that adjacent piers be constructed no closer than three pier diameters apart, as measured between centers of the piers. Drilled pier foundations should be structurally isolated from any adjacent concrete flatwork by a felt strip or similar material.

Increased bearing capacity can be achieved by increasing the embedment depth of the foundations into the relatively dense gravels. Specifically, drilled piers extending to the relatively dense gravels encountered about 28 to 32 feet below the ground surface at the boring locations can be designed based on an allowable end bearing capacity of 6000 psf for dead plus live loads with a 1/3 increase for short-term effects of wind or seismic forces.

Due to the anticipated depth of groundwater and the required drilled pier depths, the contractor should be prepared to construct the drilled piers using wet drilling methods (i.e., casing, slurry, etc.).

Uplift resistance of the pier foundations may be computed assuming the following resisting forces, where applicable: 1) the unit weight of foundation concrete (150 pound per cubic foot); and, 2) shearing resistance of 350 psf applied over the shaft area of the pier. Increased uplift resistance can be achieved by increasing the diameter of the pier or increasing the depth of the embedment depth.

It will be essential that our representative be present during pier drilling operations to verify compliance with our recommendations and the job specifications.

Lateral resistance of drilled piers can also be evaluated by determining the shear, moment and deflection of the pier using a computer model of the pier and soil (i.e. LPILE). Such an analysis is beyond the current scope of this evaluation and can be accomplished after the dimensions of the piers and loading conditions are known, if desired.
The bottom of the pier excavations should be free of loose or disturbed soils prior to placement of the concrete. Cleaning of the bearing surface may be done mechanically with the belling bucket, but should be verified by the geotechnical engineer prior to concrete placement.

Reinforcement and concrete should be placed in the pier excavations as soon as possible after excavation is completed to reduce the potential of sidewall caving into the excavations. Excessive sloughing of the sidewalls during pier construction is anticipated for piers extending deeper than about 10 feet below the existing ground surface. Therefore, we recommend that the pier contractor be prepared to case the pier holes or use drilling fluid (slurry) if conditions require.

To reduce lateral movement of the drilled shafts, it is necessary to place the concrete for the drilled shafts in intimate contact with the surrounding soil. Any voids or enlargements in the shafts due to over-excavation or temporary casing installation shall be filled with concrete at the time the shaft concrete is placed.

If the drilled piers are constructed in the "dry" (with dry being less than two inches of water at the base of the excavation), the concrete may be placed by the free-fall method, using a short hopper or back-chute to direct the concrete flow out of the truck into a vertical stream of flowing concrete with a relatively small diameter. The stream is directed to avoid hitting the sides of the excavation or any reinforcing cages. For the free-fall method of concrete placement, we recommend the concrete mix be designed with a slump of five to seven inches.

In general, we anticipate the drilled pier excavations will be relatively dry for pier excavations extending less than about 15 feet below the existing ground surface. For excavations extending deeper than about 15 feet below the existing ground surface we anticipate groundwater will be encountered which cannot be controlled such that more than six (6) inches of water accumulates at the bottom of the pier excavation. After it is confirmed that the excess water cannot be removed from the caisson excavation by bailing or with pumps, concrete should be placed using a tremie. For concrete placed using the tremie method, a slump of six to eight (8) inches, and a maximum aggregate size of ¾-inch is recommended. The required slump should be obtained by using plasticizers or water-reducing agents. Addition of water on-site to establish the recommended slump should not be allowed.

When extracting temporary casings or tremie methods from the excavation, care should be taken to maintain a head of concrete to prevent infiltration of water and soil into the shaft area. The head of concrete should always be greater than the head of water trapped outside the pier or tremie, taking into account the differences in unit weights of concrete and water.
We estimate total settlement for drilled pier foundations using the recommended maximum net allowable bearing pressure and allowable capacities presented above, will be less than one (1) inch. Differential settlements may be as much as the total settlement between individual pier elements. The settlement estimates are based on the available soil information, our experience with similar structures and soil conditions, and field verification of suitable bearing soils during foundation construction.

Pile Load Testing Program

If ACIP are used for support of the structure, a pile loading testing program conducted prior to installation of production piles will be necessary to determine and verify the appropriate length of pile to achieve the **ultimate capacity** of the piles. The pile load test program should include both static load tests and pile driving analyzer (PDA) tests. The purpose of the PDA testing for the pre-construction piles would be to develop a correlation between the static load test results and the PDA testing that would be used during the construction of production piles in lieu of "quick" load tests. The advantage of PDA testing over the "quick" load pile testing is the savings in time to set up the load test frame that typically takes three to five days, and a "quick" load test program often takes about eight hours per pile to complete

Static Load Testing

The pile load test frame and supply of the personnel and equipment necessary to conduct the load tests should be constructed in accordance with the latest version of ASTM Test Method D1143 for compressive loads, ASTM Test Method D3689 for tensile loads, and ASTM Test Method D3966 for lateral loads as delineated in the *Guide Specifications for Auger Cast Piles* provided as Appendix B.

One test pile should be cast-in-place to reach minimum tip elevations of at least 30 feet below the ground surface and at least two (2) feet into the gravel stratum. Additional test piles will be required if multiple pile sizes are used in the design or if alternate pile capacities are being considered. The reaction system should be capable of resisting forces from tests on the test piles in axial compression and tension as specified in the previous Allowable Pile Capacities table. We intend to test the test pile in compression and tension, and to perform a lateral load test between adjacent piles. The pile may be loaded to failure in any of the test configurations.

Submittals for the load testing frame, hydraulic pumps, hydraulic jacks, dial indicators, and calibration documentation must be provided by the pile contractor in accordance with the project plans and specifications.

Prior to beginning load tests, the pile concrete should achieve a minimum compressive strength of 4000 pounds per square inch when tested in accordance with ASTM C109. Construction activities must be restricted during the load-testing program. Construction activities may proceed during the set up of the load frame and installation of the test piles. Excessive vibration of the ground near the load test can cause movement of the test frame and the sensitive pile deflection measurement devices.

Final pile construction criteria will be determined from the results of the load-testing program. It is intended that the pile load test setup will be located outside the location of any permanent pile caps or grade beams, and that the test piles and reaction piles will be abandoned upon completion of the testing.

Pile Driving Analyzer Testing

Following the static load testing program, the test pile will be subjected to PDA testing, provided the pile is not damaged during the static load testing. PDA testing involves instrumenting piles and recording the response of the pile during dynamic loading. PDA testing consists of dropping a heavy weight from a certain height on to the pile head and monitoring the response of the pile. The capacity of the piles can be computed from the analyses of the PDA test.

Additional PDA testing will be performed during construction of production piles, in the event that as-built pile dimensions differ from the recommended dimensions, which could result from refusal to auger penetration or in random areas across the site to verify that the earth materials are supporting the piles as indicated by the load test program.

Surveillance/Protection

We recommend that photographic and written records be kept of both the pre-existing condition and new damage (if any) sustained by improvements in and around the site. The elevation of sidewalks and buildings adjacent to the construction site should be measured prior to construction activities. The elevations of selected survey points should be measured on a weekly basis during the initial stages of construction. Elevation of improvements and photographs should include basic data for determining the validity of claims lodged by nearby property owners or tenants.

Below-Grade Walls and Drainage

Foundations for below-grade walls, if any, may be designed and constructed as noted in the Foundation Design section of this report. The walls may be designed for an "active" earth pressure of 50 psf per foot of wall height, assuming the wall is free to rotate. If the wall is restrained at the top, or is rigid enough so that it does not rotate sufficiently to reach the active earth pressure condition, a higher lateral "at rest" earth pressure of 70 psf per foot of wall height should be used for design of rigid walls. These values do not include the effect of hydrostatic forces and assume the wall backfill is fully drained or that free water cannot collect behind the walls. Lateral resistance may be computed using an allowable passive earth pressure of 250 psf per foot of depth.

If the walls are designed to include the effects of hydrostatic forces, active and at rest pressures would increase to 90 pcf and 100 pcf, respectively, to include the effect of hydrostatic pressures. Passive pressures below the groundwater table can be evaluated using 185 pcf.

Retaining walls could experience additional surcharge loading if equipment is stored within a 1:1 projection from the bottom of the excavation. Surcharge loading under these circumstances will need to be evaluated on a case-by-case basis.

Based on recent research (Lew, et al. 2010), the seismic increment of earth pressure may be neglected if the maximum ground acceleration is 0.4 g or less. Our analysis indicates the maximum ground acceleration will be about 0.23 g; therefore, the seismic increment of earth pressure may be neglected. Earth pressures due to seismic loading may be evaluated using a total active earth pressure of 50 psf per foot of wall height and a total passive earth pressure of 200 psf per foot of wall height. The resultant active force should be applied at 1/3 times the height of the retaining wall, measured from the bottom of the wall.

Wall drainage should consist of a drainage blanket of Class 2 permeable material (Caltrans Specification Section 68-1.025) at least one foot wide extending from the base of wall to within one foot of the top of the wall. The top foot above the drainage layer should consist of engineered fill placed in accordance with the recommendations of this report. Perforated pipe should be provided at the base of the wall to collect accumulated water. Drain pipes, if used, should slope to discharge at no less than a one percent fall to a suitable sump system or drainage facilities. Open-graded ½- to ¾-inch crushed rock may be used in lieu of the Class 2 permeable material, if the rock and drain pipe are completely enveloped in an approved non-woven geotextile filter fabric. Alternatively, geotextile drainage composites such as

MiraDRAIN® may be used in lieu of the drain rock layer. If used, geocomposite drain panels should be installed in accordance with the manufacturer's recommendations.

If efflorescence (discoloration of the wall face) or moisture penetration of the wall is not acceptable, waterproofing measures should be applied to the back face of the wall. A specialist in protection against moisture penetration should be consulted to determine specific waterproofing measures.

Structural backfill materials for retaining walls should be placed and compacted as noted in the Engineered Fill Construction section of this report. Pea gravel and crushed rock are not considered suitable backfill materials for retaining walls.

Interior Grade Slab Support

The interior concrete slabs-on-grade can be supported upon the soil subgrade prepared in accordance with the recommendations in this report and maintained in that condition. Slabs-on-grade that will be used for vehicle support should be designed in accordance with the recommendations provided in the Pavement Design section of this report.

Interior slab-on-grade concrete slabs that will not be used for vehicle support should be at least four inches thick and, as a minimum, contain chaired No. 3 reinforcing bars on 18-inch center-on-center spacing, located at mid-slab depths. All reinforcing should be located at mid-slab depth. This slab reinforcement is suggested as a guide "minimum" only for crack control; final reinforcement and joint spacing should be determined by the structural engineer. Wheel loads from forklifts, storage of palletized materials, cranes, etc., anticipated during construction should be considered in the design of the slab-on-grade floors.

Conventional floor slabs may be underlain by a layer of free-draining gravel serving as a deterrent to migration of capillary moisture. If used, the gravel layer should be at least four inches thick and graded such that 100 percent passes a one-inch sieve and no appreciable amount passes a No. 4 sieve. Additional moisture protection may be provided by placing a water vapor retarder (at least 10-mils thick) directly over the gravel. If used, the water vapor retarder should meet or exceed that standard specification as outlined in ASTM E1745.

Floor slab construction practice over the past 25 years or more has included placement of a thin layer of sand over the vapor retarder membrane. The intent of the sand is to aid in the proper curing of the slab concrete. However, recent debate over excessive moisture vapor emissions from floor slabs includes concern of water trapped within the sand. As a consequence, we consider use of the sand layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

The recommendations presented above should reduce significant soils-related cracking of slab-on-grade floors. Also important to the performance and appearance of a Portland cement concrete slab is the quality of the concrete, the workmanship of the concrete contractor, the curing techniques utilized and spacing of control joints.

Floor Slab Moisture Penetration Resistance

It is likely the floor slab subgrade soils will become saturated at some time during the life of the structure, especially when slabs are constructed during the wet season and when constantly wet ground or poor drainage conditions exist adjacent to structures. For this reason, it should be assumed that all interior slabs, particularly those intended for moisture-sensitive floor coverings or materials, require protection against moisture or moisture vapor penetration. Standard practice includes placing a layer of rock and a vapor retarder membrane (and possibly a layer of sand) as discussed above. Recommendations contained in this report concerning foundation and floor slab design are presented as minimum requirements only from the geotechnical engineering standpoint.

Use of sub-slab gravel and a vapor retarder membrane will not "moisture proof" the slab, nor does it assure that slab moisture vapor transmission levels will be low enough to prevent damage to floor coverings or other building components. It is emphasized that we are not slab moisture proofing or moisture protection experts. The sub-slab gravel and vapor retarder membrane simply offer a first line of defense against soil-related moisture. If increased protection against moisture vapor penetration of the slab is desired, a concrete moisture protection specialist should be consulted. It is commonly accepted that maintaining the lowest practical water-cement ratio in the slab concrete is one of the most effective ways to reduce future moisture vapor penetration of the completed slab.

Exterior Concrete Flatwork

Exterior concrete flatwork may be constructed directly on the prepared soil subgrade prepared and compacted in accordance with the recommendations of this report. A four-inch layer of aggregate base could be used as a leveling course under flatwork if necessary, compacted to not less than 95 percent relative compaction.

Flatwork should be at least four inches thick and reinforced for crack control. Reinforcement should include, as a minimum, chaired No. 3 rebar located on maximum 18-inch centers, both ways, throughout slabs. Accurate and consistent location of the reinforcement at mid-slab is essential to its performance and the risk of uncontrolled drying shrinkage slab cracking is increased if the reinforcement is not properly located within the slab.

Uniform moisture conditioning of subgrade soils is important to reduce the risk of non-uniform moisture withdrawal from the concrete and the possibility of plastic shrinkage cracks. Practices recommended by the Portland Cement Association (PCA) for proper placement and curing of concrete should be followed during exterior concrete flatwork construction. Flatwork should be independent of the building foundations and felt strips should be used to separate concrete slabs from building foundations.

The architect or civil engineer should determine the final thickness, strength, reinforcement, and joint spacing of exterior slab-on-grade concrete. Exterior flatwork next to landscaped areas should be thickened to twice the slab thickness for a width of at least 12 inches to help support lawn mowing equipment and other maintenance equipment.

Exterior flatwork should be constructed independent of the building foundations. Isolated column foundations should be structurally separated from adjacent flatwork by the placement of a layer of felt, or other appropriate material, between the flatwork and foundations. Practices recommended by the Portland Cement Association (PCA) for proper placement and curing of concrete should be followed during exterior concrete flatwork construction.

Exterior flatwork that will be traversed by vehicles or heavy equipment should be designed in accordance with the recommendations provided in the Pavement Design section of this report.

Pavement Design

We are providing several alternative pavement designs based on the soil conditions encountered at the site, the results of laboratory testing previously obtained at the site, and our experience.

The procedures used to design the pavement sections are in general conformance with the "Flexible Pavement Structural Design Guide for California Cities and Counties" dated January 1979, and the *California Highway Design Manual, Sixth Edition*. Laboratory testing of the on-site soils indicates an R-value of 60 was obtained on the near-surface soils at the site. Based on our experience with similar soil conditions and the variability of the near-surface soils, an R-value of 40 is considered appropriate for design of pavements at the site.

PAVEMENT DESIGN ALTERNATIVES
R-value = 40

Traffic Index (TI)	Traffic Condition	Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)
4.5	Automobile Parking Only	2½*	4	--
		--	4	4
7.0	Entrance/Exit Driveways	3	9	--
		4*	7	--
		--	4	6

* = Asphalt thickness includes Caltrans Factor of Safety.

We emphasize that the performance of the pavement is critically dependent upon adequate and uniform compaction of the subgrade soils, including utility trench backfill within the limits of the pavements. The upper six inches of untreated pavement subgrade should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. Aggregate base materials should be compacted to at least 95 percent of the maximum dry density. Class 2 aggregate base should conform to Section 26 of the Caltrans Standard Specifications.

It has been our experience that pavement failures may occur where a non-uniform or disturbed subgrade soil condition is created. Subgrade disturbances can result if pavement subgrade preparation is performed prior to underground utility construction and/or if a significant time period passes between subgrade preparation and placement of aggregate base. Therefore, we recommend that final pavement subgrade preparation (i.e. scarification, moisture conditioning, and compaction) be performed just prior to aggregate base placement.

We suggest that concrete slabs be constructed with thickened edges at least two inches plus the slab thickness and 36 inches wide in accordance with American Concrete Institute (ACI) design standards. Reinforcing for concrete pavement crack control, if desired, should consist of No. 3 reinforcing bars placed on maximum 18-inch centers each way throughout the slab.

Reinforcement must be located at mid-slab depth to be effective. Portland cement concrete should achieve a minimum compressive strength of 3500 psi at 28 days. Concrete curing and joint spacing and details should conform to current PCA and ACI guidelines.

We suggest considering the use of full depth curbs where pavements abut landscaping. The curbs should extend to at least the surface of the soil subgrade. Weep holes also could be provided at storm drain drop inlets, located at the subgrade-base interface, to allow water to drain from beneath the pavements.

Site Drainage

Site drainage should be accomplished to provide positive drainage of surface water away from the proposed structures and prevent ponding of water adjacent to foundations. The subgrade adjacent to the proposed structures should be sloped away from foundations at a minimum two percent gradient for at least 10 feet, where possible. We recommend consideration be given to connecting all roof drains to non-perforated rigid pipes which are connected to available drainage features to convey water away from the structure, or discharging the drains onto paved surfaces that slope away from the foundations. Ponding of surface water should not be allowed adjacent to the proposed structures or pavements.

Observation and Testing of Earthwork Construction

Site preparation should be accomplished in accordance with the recommendations of this report. Representatives of the Geotechnical Engineer should be present during site preparation and all grading operations to observe and test the fill to verify compliance with our recommendations and the job specifications. These services are beyond the scope of work authorized for this investigation.

Additional Services

We recommend that our firm be retained to review the final plans and specifications to determine if the intent of our recommendations has been implemented in those documents.

LIMITATIONS

Our recommendations are based upon the information provided regarding the proposed project, combined with our analysis of site conditions revealed by the field exploration and laboratory testing programs. We have used prudent engineering judgment based upon the information provided and the data generated from our investigation.

This report has been prepared in substantial compliance with generally accepted geotechnical engineering practices that exist in the area of the project at the time the report was prepared. No warranty, either express or implied, is provided.

If the proposed construction is modified or re-sited; or, if it is found during construction that subsurface conditions differ from those we encountered at our boring locations, we should be afforded the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations must be modified.

We emphasize that this report is applicable only to the proposed construction and the investigated site, and should not be utilized for construction on any other site. The conclusions and recommendations of this report are considered valid for a period of two years. If design is not completed and construction has not started within two years of the date of this report, the report must be reviewed and updated if necessary.

Wallace - Kuhl & Associates

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Senior Engineer

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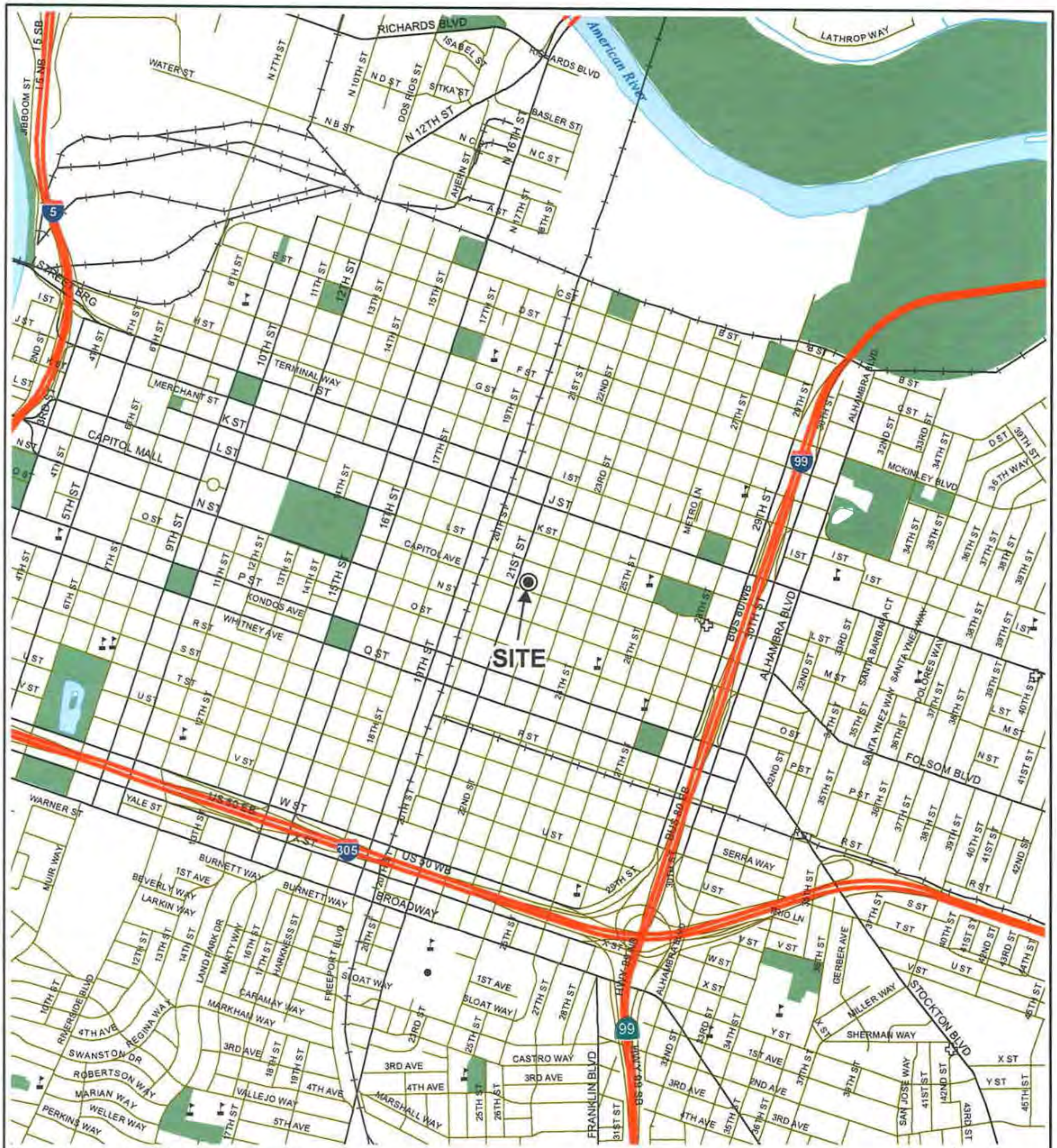

Street data courtesy of Sacramento County.
Hydrography courtesy of the U.S. Geological Survey
acquired from the GIS Data Depot, December, 2007.
Projection: NAD 83, California State Plane, Zone II

N

0 1,000 2,000
Feet

WallaceKuhl & ASSOCIATES

VICINITY MAP

21ST AND CAPITOL AVENUE MIXED USE

Sacramento, California

FIGURE	1
DRAWN BY	TJC
CHECKED BY	MSM
PROJECT MGR	MSM
DATE	1/14
WKA NO. 9957.01	

Adapted from a Google Earth aerial photograph, dated August 14, 2013.
Projection: NAD 83, California State Plane, Zone II

Legend

Approximate soil boring location

N

0 25 50
Feet

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SITE PLAN

21ST AND CAPITOL AVENUE MIXED USE

Sacramento, California

FIGURE	2
DRAWN BY	TJC
CHECKED BY	MSM
PROJECT MGR	MSM
DATE	1/14
WKA NO. 9957.01	

Project: 21st and Capitol Ave (NE Corner)	LOG OF SOIL BORING D1
Project Location: Sacramento, California	Sheet 1 of 1
WKA Number: 9957.01P	

Date(s) Drilled: 12/9/13	Logged By: GJF	Checked By: MSM
Drilling Method: Hollow Stem Auger	Drilling Contractor: V&W Drilling, Inc.	Total Depth of Drill Hole: 33.2 feet
Drill Rig Type: CME-55	Diameter(s) of Hole, inches: 8"	Approx. Surface Elevation, ft MSL:
Groundwater Depth [Elevation], feet: 20.0	Sampling Method(s): California Modified	Drill Hole Backfill: Cement Grout
Remarks:		Driving Method and Drop: 140-lb automatic hammer; 30-inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, very loose to loose, sandy silt (ML)						
					D1-1I	3	25.8	62	
	5				D1-2I	10	19.4	83	
	10				D1-3I	6	28.1	93	TR
	15		Brown, moist to wet, stiff, clayey silt (ML)		D1-4I	10	29.0	94	
	20				D1-5I	14	25.5	100	
			Brown, wet, very stiff, silty clay (CL)						
	25				D1-6I	36			
	30		Brown, wet, very stiff, clayey silt (ML)						
			Brown, wet, medium dense, sandy silt (ML)		D1-7I	34			
			Gray brown, wet, very dense, silty sandy gravel (GM)		D1-8I	50/2"			
			Boring terminated at 33 feet below existing site grade. Groundwater was encountered about 20 feet below the ground surface during and immediately after the drilling operations.						

BORING LOG 9957.01P - 21ST AND CAPITOL AVE (NE CORNER).GPJ WKA.GDT 1/31/14 1:15 PM

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FIGURE 3

Project: 21st and Capitol Ave (NE Corner)
Project Location: Sacramento, California
WKA Number: 9957.01P

LOG OF SOIL BORING D2

Date(s) Drilled	12/9/13	Logged By	GJF	Checked By	MSM
Drilling Method	Hollow Stem Auger	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	28.3 feet
Drill Rig Type	CME-55	Diameter(s) of Hole, inches	8"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	20.0	Sampling Method(s)	California Modified	Drill Hole Backfill	Cement Grout
Remarks				Driving Method and Drop	140-lb automatic hammer; 30-inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0		Dark brown, moist, loose, sandy silt (ML)						
				■	D2-1I	7	24.1	70	
	5			■	D2-2I	7	29.4	83	TR
	10			■	D2-3I	12	28.9	91	
	15		Brown, moist, to wet, very stiff to hard, silty clay (CL)	■	D2-4I	21	25.6	99	
	20		▼	■	D2-5I	23	26.1	95	
	25			■	D2-6I	45			
			Gray brown, wet, very dense, silty sandy gravel (GM)	■	D2-7I	50/4"			

Boring terminated at 28 feet below existing site grade. Groundwater was encountered about 20 feet below the ground surface during and immedately after the drilling operations.

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FIGURE 4

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		SYMBOL	CODE	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of soil > no. 200 sieve size)	GRAVELS (More than 50% of coarse fraction > no. 4 sieve size)	GW		Well graded gravels or gravel - sand mixtures, little or no fines
		GP		Poorly graded gravels or gravel - sand mixtures, little or no fines
		GM		Silty gravels, gravel - sand - silt mixtures
		GC		Clayey gravels, gravel - sand - clay mixtures
	SANDS (50% or more of coarse fraction < no. 4 sieve size)	SW		Well graded sands or gravelly sands, little or no fines
		SP		Poorly graded sands or gravelly sands, little or no fines
		SM		Silty sands, sand - silt mixtures
		SC		Clayey sands, sand - clay mixtures
FINE GRAINED SOILS (50% or more of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL		Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS LL ≥ 50	MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH		Inorganic clays of high plasticity, fat clays
		OH		Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils
ROCK		RX		Rocks, weathered to fresh
FILL		FILL		Artificially placed fill material

OTHER SYMBOLS

= Drive Sample: 2-1/2" O.D. Modified California sampler

= Drive Sampler: no recovery

= SPT Sampler

= Initial Water Level

= Final Water Level

= Estimated or gradational material change line

= Observed material change line

Laboratory Tests

PI = Plasticity Index

EI = Expansion Index

UCC = Unconfined Compression Test

TR = Triaxial Compression Test

GR = Gradational Analysis (Sieve)

K = Permeability Test

GRAIN SIZE CLASSIFICATION

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL coarse (c) fine (f)	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND coarse (c) medium (m) fine (f)	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.074 4.76 to 2.00 2.00 to 0.420 0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

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UNIFIED SOIL CLASSIFICATION SYSTEM

21ST AND CAPITOL AVENUE MIXED USE

Sacramento, California

FIGURE	5
DRAWN BY	TJC
CHECKED BY	MSM
PROJECT MGR	MSM
DATE	1/14
WKA NO. 9957.01	

APPENDICES

APPENDIX A
Field and Laboratory Testing

APPENDIX A

A. GENERAL INFORMATION

The performance of a geotechnical engineering investigation for the proposed 21st Street and Capitol Avenue Mixed Use development in Sacramento, California was authorized by Steve Vannatta on November 20, 2013. Authorization was for an investigation as described in our proposal letter dated September 10, 2013, sent to our client LVP Revocable Trust, whose address is 2020 L Street, 5th Floor, Sacramento, California 95811; telephone (916) 447-7100; facsimile (916) 447-7112.

B. FIELD EXPLORATION

Two (2) borings were drilled at the site on December 9, 2013, at the approximate locations indicated on Figure 2 utilizing a CME-75 truck-mounted drill rig. The borings were drilled to maximum depths of approximately 28 to 33 feet below existing site grades using eight-inch (8") diameter, hollow-stem helical augers. At various intervals, relatively undisturbed soil samples were recovered with a 2½-inch O.D., 2-inch I.D., modified California sampler driven by an automatic 140-pound hammer freely falling 30 inches. The number of blows of the hammer required to drive the 18-inch long sampler each six-inch (6") interval was recorded. The sum of the blows required to drive the sampler the lower 12-inch interval, or portion thereof, is designated the penetration resistance or "blow count" for that particular drive.

The samples were retained in two-inch (2") diameter by six-inch (6") long thin-walled brass tubes contained within the sampler. Immediately after recovery, the soils in the tubes were visually classified by the field engineer and the ends of the tubes were sealed to preserve the natural moisture contents. All samples were taken to our laboratory for soil classification and selection of samples for testing.

The Logs of Soil Borings, Figures 3 and 4, contain descriptions of the soils encountered at each boring location. A Boring Legend explaining the Unified Soil Classification System and the symbols used on the logs is contained on Figure 5.

C. LABORATORY TESTING

Selected undisturbed samples of the soils were tested to determine dry unit weight (ASTM D2937), natural moisture content (ASTM D2216) and shear strength by triaxial strength testing (ASTM D4767). The results of the moisture/density tests are included on the boring logs at the depth each sample was obtained. The results of the shear strength testing are presented on Figures A1 and A2.

A bulk sample of the anticipated pavement subgrade soil was subjected to Resistance-value ("R-value") testing in accordance with California Test 301. The results of the R-value test, which were used in the pavement design, are presented on Figure A3.

A composite sample of near-surface soil was submitted to Sunland Analytical of Rancho Cordova, California, for corrosivity testing in accordance with California Test (CT) Nos. 643 (Modified Small Cell), CT 422 and CT 417. Copies of the analytical results are presented on Figure A4.

APPENDIX B
Guide Specificaionts for Auger Cast Piles

APPENDIX B

GUIDE SPECIFICATIONS FOR AUGER CAST PILES

21ST STREET AND CAPITOL AVENUE MIXED USE

Sacramento, California

WKA No. 9957.01

PART 1: GENERAL

1.1 SUMMARY

A. This Section includes construction of compression and tension auger cast piles, where shown on contract drawings and specified herein.

B. The Contractor shall furnish all labor, materials, tools, and equipment necessary for designing, furnishing, installing, inspecting and testing augered cast-in-place piles, and shall remove and dispose spoils generated by pile construction.

1.2 WORK NOT INCLUDED UNDER THIS SECTION

A. Concrete pile caps: Section _______.

B. Excavations: Section ______.

C. Shoring and bracing of earth banks: Section ______

D. Dewatering: Section ______.

1.3 REFERENCE STANDARDS

A. Requirements, abbreviations and acronyms for reference standards are defined in Section _____.

B. American Concrete Institute (ACI)

1. ACI 305 - Hot Weather Concreting.
2. ACI 306 - Cold Weather Concreting.
3. ACI 315 - Details and Detailing of Concrete Reinforcement.

C. American Society for Testing and Materials (ASTM) latest editions

1. ASTM A615 - Deformed and Plain Billet-Steel Bars for Concrete Reinforcement.
2. ASTM C33 - Concrete Aggregates.
3. ASTM C31 - Standard Practice for Making and Curing Concrete Test Specimens in the Field
4. ASTM C109 - Test Method for Compressive Strength of Hydraulic Cement Mortars.
5. ASTM C150 - Portland Cement.
6. ASTM C618 - Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete.
7. ASTM C939 - Test Method for Flow of Grout for Preplaced - Aggregate Concrete (Flow Cone Method)
8. ASTM C942 - Test Method for Compressive Strength of Grouts for Preplaced-Aggregate Concrete in the Laboratory.
9. ASTM D1143 - Test Method for Piles Under Static Axial Compressive Load.
10. ASTM D3689 - Test Method for Individual Piles Under Static Axial Tensile Load.
11. ASTM D3966 - Test Method for Piles Under Lateral Loads.

1.4 PROTECTION

A. Adequate protection measures shall be provided to protect workers and passers-by at the site. Streets and adjacent property shall be fully protected throughout the operations.

B. In accordance with generally accepted construction practices, the Contractor shall be solely and completely responsible for working conditions at the job site, including safety of all persons and property during performance of the work. This requirement shall apply continuously and shall not be limited to normal working hours.

C. Any construction review of the Contractor's performance conducted by the Geotechnical Engineer is not intended to include review of the adequacy of the Contractor's safety measures, in, on or near the construction site.

D. Adjacent streets and sidewalks shall be kept free of mud, dirt or similar nuisances resulting from earthwork operations.

E. Surface drainage provisions shall be made during the period of construction in a manner to avoid creating a nuisance to adjacent areas.

F. The site and adjacent influenced areas shall be watered as required to suppress dust nuisance.

1.5 EXISTING SITE CONDITIONS

Piling Contractor shall inspect the site and related conditions prior to commencing his/her portion of the work. If unshown active utilities are encountered during the work, the Architect shall be promptly notified for instructions. Failure to notify will make the Contractor liable for damage to these utilities arising from Contractor's operations subsequent to the discovery of such unshown utilities.

1.6 GEOTECHNICAL ENGINEERING REPORT

A. A *Geotechnical Engineering Report* (WKA No. 9957.01, dated January 31, 2014), has been prepared by Wallace - Kuhl & Associates, Geotechnical Engineers of West Sacramento, California; telephone (916) 372-1434; facsimile (916) 372-2565. That report is available for review at the office of Wallace - Kuhl & Associates.

B. The Piling Contractor shall submit in writing to the Architect and/or Structural Engineer, all applicable information as listed in Subsection 1.7 - Submittals for review and approval, in addition to the above experience record.

C. The Owner does not guarantee that the information contained in the Geotechnical Engineering Report is correct nor that the conditions revealed at the actual boring locations will be continuous over the entire site. This report was prepared for purposes of design only. Making the report available to contractors shall not be construed in any way as a waiver of this position. The Piling Contractor shall be responsible for any conclusions he/she may draw from this report. Should the Contractor prefer not to assume such risk, he/she is under obligation to employ their own experts to analyze available information and/or to make their own tests upon which to base their conclusions.

1.7 SUBMITTALS

Submit the following according to Conditions of the Construction Contract and Division 1 Specifications, for Owner's approval.

A. Shop Drawings: Shall clearly indicate but not be limited to:
   1. Description of the pile drilling and grouting equipment and procedures to be utilized in installations.
   2. Proposed pile grout design mix and description of materials to be used in sufficient detail to indicate their compliance with the specifications and either;
      a. Laboratory tests of trial mixes made with the proposed mix, or
      b. Laboratory tests of the proposed mix used on previous projects.
   3. A pile layout plan referenced to the structural plans including a numbering system capable of identifying each individual pile, and indicating pile cutoff elevations.
   4. A dimensioned sketch of the pile load test arrangements, including sizes of primary members, data on testing and measuring equipment including required jack and gauge calibrations, load cell and professional engineer seal certifying the adequacy of the reaction frames.
   5. Fabrication and installation schedule covering test pile installation, pile testing, and production pile installation, with excavation schedule for pile cap and finished subgrades by area.
   6. Qualifications of pile installation construction personnel, supervisor, and technician.

B. Records
   1. The Contractor shall submit a pile design report indicating construction methods and materials that will be utilized to install piles of the specified compression and tension capacity, meeting the criteria of this specification and the Contract Drawings. The report shall be prepared and sealed by a Professional Engineer licensed in the state of California.
   2. The Contractor shall provide a Technician for each pile rig responsible for observing the auger construction, grout batching, and grouting operations and preparing installation records. The Contractor's inspector shall submit an installation record for each pile not later than two (2) days after installation is completed. The report shall include but not be limited to:
      a. Project name and number
      b. Name of contractor
      c. Pile number
      d. Pile location, date and time of installation
      e. Design pile capacity, compression or tension
      f. Pile diameter
      g. Tip elevation
      h. Cut off elevation
      i. Elevation of butt
      j. Drilling elevation
      k. Rate of advancement of auger and rotation speed
      l. Quantity of grout placed as compared to the theoretical volume for each pile, in five-foot (5') depth increments, and total for pile
      m. Grout pressures
      n. Pile reinforcing steel
      o. Grout flow cone test report
      p. Any unusual occurrences observed during pile installation, and pile deviation from vertical

3. The grout quantity shall be determined by recording grout pump displacement or by other acceptable means; the pile installation record shall reveal the observed measure and quantity.
4. Load test reports shall be in accordance with the applicable ASTM Standards.
5. Grout compression test reports.

C. Hazardous Materials Notification: In the event no alternative product or material is available that does not contain asbestos, PCB or other hazardous materials as determined by the Owners' Authorized Representative, a "Material Safety Data Sheet" (MSDS) equivalent to OSHA Form 20 shall be submitted for that proposed product or material prior to installation.

D. Asbestos and PCB Certification: After completion of installation, but prior to Substantial Completion, Contractor shall certify in writing that products and materials installed, and processes used, do not contain asbestos or polychlorinated biphenyls (PCB), using format in Section ____/Closeout Procedures.

1.8 DELIVERY, HANDLING, STORAGE

Comply with General Conditions and Section 01600/Product Requirements.

1.9 WARRANTY

Comply with General Conditions and Section ____/Product Requirements.

PART 2: PRODUCTS

2.1 QUALITY ASSURANCE

A. The work of this section shall be performed by a company specialized in auger cast pile work with a minimum of five (5) years of documented successful experience, and shall be performed by skilled workers thoroughly experienced in the necessary crafts. Contractor shall submit evidence of successful installation of augered cast-in-place piles under similar job and subsurface conditions, including a job supervisor who shall have a minimum of three (3) years of method specific experience.

B. Work shall comply with all Municipal, State and Federal regulations regarding safety, including the requirements of the Williams-Steiger Occupational Safety and Health Act of 1970.

2.2 MATERIALS

A. Portland Cement: conforming to ASTM C150.

B. Mineral Admixture: Mineral admixture, if used, shall be fly ash or natural pozzolan which possesses the property of combining with the lime liberated during the process of hydration of Portland cement to form compounds containing cementitious properties, conforming to ASTM C618, Class C or Class F.

C. Fluidifier conforming to ASTM C937, except that expansion shall not exceed 4%.

D. Water: Potable, fresh, clean and free of sewage, oil, acid, alkali, salts or organic matter.

E. Fine Aggregate: Conforming to ASTM C33.

F. Grout Mixes.
1. The grout shall consist of Portland cement, sand and water, and may also contain a mineral admixture and approved fluidifier.
   a. The components shall be proportioned and mixed to produce a grout capable of maintaining the solids in suspension, which may be pumped without difficulty and which will penetrate and fill open voids in the adjacent soils.
   b. These materials shall be proportioned to produce a hardened grout which will achieve the design strength within twenty-eight (28) days.
   c. The design grout strength at twenty-eight (28) days for this project shall be a minimum four thousand pounds per square inch (4000 psi).
2. All materials shall be accurately measured by volume or weight as they are fed to the mixer.
   a. Time of mixing shall be not less than one minute at the site.
   b. If agitated continuously, the grout may be held in the mixer or agitator for a period not exceeding two and one-half (2½) hours at grout temperatures below seventy degrees Fahrenheit (70°F) and for a period not exceeding one hundred degrees Fahrenheit (100°F).
   c. Grout shall not be placed when its temperature exceeds one hundred degrees Fahrenheit (100°F).
3. Protect grout from physical damage or reduced strength, which could be caused by frost, freezing actions or low temperatures or from damage during high temperatures in accordance with ACI 305/306.
4. The grout shall be tested by making a minimum of six, two-inch (2") diameter by four-inch (4") tall cylinders for each day during which piles are placed.
   a. A set of six (6) cylinders shall consist of two (2) cylinders tested at seven (7) days, and two (2) cylinders tested at twenty-eight (28) days. Two (2) cylinders shall be held in reserve.
   b. Test cylinders shall be cured and tested in accordance with ASTM C109.
   c. Cylinder specimens shall be cast and cured in accordance with ASTM C31.
   d. Cylinder specimens may be restrained from expansion as described in ASTM C942.
5. Test the flow of grout for each pile and batch of grout. Maintain grout fluidity between fifteen (15) and twenty-five (25) seconds through a three-quarters inch (¾") diameter grout cone.

G. Steel Reinforcing:
1. Minimum reinforcing steel assemblies are shown on the Contract Drawings. Assemblies shall be detailed and fabricated in accordance with the manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315).
2. Reinforcing shall conform to the requirements of ASTM A615, Grade 60.
3. All reinforcing bar shall be epoxy coated, including bars installed for contractor convenience. Wire ties do not require epoxy coating.
4. Contractor shall provide labor, materials, and method for coating cut ends and repairing holidays in epoxy coating.
5. Acceptable materials and methods shall be provided to facilitate proper centering of all steel reinforcing installed.
6. Bars may be bent in place, provided epoxy coating at all bends is inspected, flaked coating is removed by wire brush, and holidays in coating are repaired.

7. A corrugated metal pipe sleeve shall be provided for each pile equal to the diameter of the auger, to define the pile butt and permit cut-off to specified elevations.

2.3 EQUIPMENT

A. Augering Equipment:

1. The auger flighting shall be continuous from the auger head to the top of auger without gaps or other breaks.
2. The auger flighting shall be uniform in diameter throughout its length and shall be the diameter specified for the piles less a maximum of three percent (3%). The hole through which the grout is pumped during the placement of the pile shall be located at the bottom of the auger head below the bar containing the cutting teeth.
3. Augers over forty feet (40') in length shall contain a middle support guide.
4. The piling leads shall be prevented from rotating by a stabilizing arm or by firmly placing the bottom of the leads into the ground or by some other acceptable means.
5. Leads shall be marked at one-foot (1') intervals to facilitate measurement of auger penetration.
6. Auger hoisting equipment shall be provided that will enable the auger to be rotated while being withdrawn.

B. Mixing and Pumping Equipment:

1. Only approved pumping and mixing equipment shall be used in the preparation and handling of the grout.
   a. Provide a screen to remove over-size particles at the pump inlet.
   b. All oil or other rust inhibitor shall be removed from mixing drums and grout pumps before each use.
   c. All materials shall be such as to produce a homogeneous grout of the desired consistency and strength.
2. The grout pump shall be a positive displacement pump capable of developing displacement pressures at the pump of three hundred fifty pounds per square inch (350 psi) or higher.
   a. The grout pump shall be provided with a pressure gauge in clear view of the equipment operator.
   b. The grout pump shall be calibrated at the beginning of the work and periodically during the work to determine the volume of grout pumped per stroke, under operating pressure.
   c. A positive method for automatic counting of grout pump strokes shall be provided. Such methods may include digital or mechanical stroke counters or other acceptable methods.
   d. A second pressure gauge, if required, shall be provided close to the auger rig where it can be readily observed by the inspector, if required.

PART 3: EXECUTION

3.1 EXAMINATION

A. The Contractor is responsible for supporting pile drilling equipment and concrete grout batching and delivery equipment. Equipment shall be supported on timber mats or gravel fill work platforms, if necessary for safety and stability, and to prevent damage.

B. The Contractor shall examine the areas and evaluate conditions under which piles are to be installed and shall include measures for the proper and timely completion of the work in the construction methods and pile design.

3.2 AUGER CAST PILE SYSTEM DESCRIPTION

A. Augered Pressure Grouted Piles

1. Pressure grouted piles shall be made by drilling a continuous-flight, hollow-shaft auger into the ground to the design pile depth, or until refusal criteria is satisfied. The volume of soil extracted shall not be greater than the volume of the steel auger stem inserted.
2. Grout shall be injected through the auger shaft as the auger is being withdrawn. First develop a five-foot (5') plug at the bottom of the auger flights, then inject sufficient grout volume to fill the augered hole one point one five (1.15) times its neat dimension, or more. Grout volumes shall be logged by depth during withdrawal.
3. Post-grouting through a special grout tube for capacity increase is permitted, given these methods are used in the test piles, and consistently throughout the entire work for this project. Post-grouting may be used for compression and tension capacity. Post-grout pressures must be sufficient to open grout portals and cause fracture and flow. Grout volumes and pressures shall be recorded and used as a measure to demonstrate pile compliance with the design and pile load test criteria.

B. Augered Displacement Pressure Grouted Piles

1. Augered Displacement Pressure Grouted piles shall be made by rotating a specialized auger capable of displacing soil surrounding the auger, with minimal soils returned to the ground surface to reach the design pile depth, or until specified refusal criteria is satisfied.
2. Grout shall be injected through the auger shaft as the auger is being withdrawn in such a way as to exert a positive upward grout pressure on the auger, as well as a positive lateral pressure on the soil surrounding the pile.

C. Alternatives

1. Alternative pile types which meet the compression and tension pile criteria given on the drawings may be substituted for augered pressure-grouted pile systems described in this Section.
2. Alternative pile installation systems must be capable of achieving the specified compression and tension, and shall provide a working lateral capacity of eight tons (8).

3.3 PILE DESIGN

A. The ultimate capacity of eighteen inch (18") diameter compression piles shall be greater than two hundred forty tons (240) in axial compression and greater than sixty (60) tons in axial tension or the ultimate capacity of twenty four inch (24") diameter compression piles shall be greater than four hundred twenty (420) tons in axial compression and greater than one hundred five (105) tons in axial tension. Both tension and compression piles shall achieve an ultimate lateral capacity of five (5) tons for eighteen inch (18") diameter piles or ten (10) tons for twenty four inch (24") diameter piles. The allowable design capacities of all piles shall be determined by dividing the ultimate capacity by the appropriate factor of safety as provided in the Geotechnical Engineering Report. Load Testing performed under Part 3.4 of this section shall confirm the ultimate capacity of the piles.

B. Pile design shall be performed by the Contractor and demonstrated by load test before installation of production piles. All piles shall meet the criteria specified on the Contract Drawings.

C. The design shall be described in a pile design report. This report shall indicate variances, if any, from the reinforcing steel specified or the requirements of this section, and shall demonstrate that the design meets or exceeds the specified performance in tension, compression, and bending. The Contractor shall submit design calculations for the proposed piles demonstrating compression and tensile capacity.

3.4 LOAD TESTING

A. Pre-construction Pile Load Tests:

1. Install and test one (1) compression pile, one (1) tension pile, and one (1) lateral load test pile, at the locations shown on the plans or approved alternate location to verify the construction methods and pile capacity. Test piles and reaction piles shall be installed outside of pile cap locations.
2. The Contractor shall provide complete testing materials and equipment as required, install test and reaction piles and perform the load tests only in the presence of the Owner.
3. The pile test reaction frame shall be capable of safely sustaining two hundred fifty (250) tons in axial compression and one hundred (100) tons in axial tension (uplift) for eighteen inch (18") diameter piles or four hundred thirty (430) tons in axial compression and one hundred ten (110) tons in axial tension (uplift) for eighteen inch (24") diameter piles.
4. Preconstruction Pile Load tests shall be performed using ASTM's Quick Test Methods.
5. One successful compression pile load test shall be performed in accordance with ASTM D1143.
6. One successful tension pile load test shall be performed in accordance with ASTM D3689.
7. One lateral pile load test to five (5) tons for eighteen inch (18") diameter piles or ten (10) tons for twenty four inch (24") ultimate load shall be performed in accordance with ASTM D3966.

3.5 INSTALLATION

A. Tolerance

1. Piles shall be located where shown on drawings or where otherwise directed by the Engineer.
   a. Pile centers shall be located to an accuracy of three inches (±3").
   b. Vertical piles shall be plumb within two percent (2%).
   c. Battered piles shall be installed to within four percent (4%) of the specified batter as determined by the angle from horizontal.

B. Adjacent Piles

1. Adjacent piles within ten feet (10'), center-to-center, shall not be installed within twenty-four (24) hours of each other.
2. Within pile caps, piles adjacent within four (4) pile diameters center-to-center, shall not be installed within twenty-four (24) hours of each other.

C. Installation Procedure

1. The length and drilling criteria of production piles will be as defined in the Contractor's design and as demonstrated by the successful pile load tests. Advance and rotate the auger at a continuous rate that prevents removal of excess soil.
2. Stop advancement after reaching the required depth or refusal criteria.
3. The hole in the bottom of the auger shall be closed with a suitable plug while advancing into the ground. The plug shall be removed by grout pressure or mechanically with the reinforcing bar.
4. At the start of pumping grout, raise the auger from six inches (6") to twelve inches (12") and after the grout pressure builds up sufficiently, re-drill the auger to the previously established tip elevation.
5. Maintain a head of at least fifteen feet (15') of grout on the auger flighting above the injection point during auger withdrawal.
    a. Positive rotation of the auger shall be maintained at least until placement of the grout.
    b. Rate of grout injection and rate of auger withdrawal from the soil shall be coordinated so as to maintain at all times the minimum grout head.
    c. The total volume of grout shall be at least one hundred fifteen percent (115%) of the theoretical volume for each pile.
    d. After grout is flowing at the ground surface from the auger flighting, the rate of grout injection and auger withdrawal shall be coordinated so that there is a constant grout flow at the surface.
    e. If pumping grout is interrupted for any reason, the contractor shall reinsert the auger by drilling at least five feet (5') below the depth of the auger where the interruption occurred, and re-grout while withdrawing the auger from that depth.
6. If less than one hundred fifteen percent (115%) of the theoretical volume of grout is placed in any five foot (5') increment (until the grout head on the auger flighting reaches the ground surface), the pile increment shall be reinstalled by advancing the auger ten feet (10') or to the bottom of the pile if that is less, followed by controlled removal and grout injection.
7. Spoil material that accumulates around the auger during injection of the grout shall be promptly cleared away.
8. A steel corrugated metal pipe (CMP) sleeve shall be placed at the top of each pile to a depth of one and one half feet (1½') below the pile cutoff elevation.

D. Obstructions and Damaged Piles

1. If non-augerable material is encountered above the desired tip elevation, the pile shall be completed to the depth of the non-augerable material in accordance with these Specifications. Such short piles shall be included for payment, if completed and included within the foundation. If required by the Engineer, additional adjacent piles shall be placed. Additional piles shall also be included in the total number of piles for payment.
2. Damaged piles, and piles installed outside the required installation tolerances, will not be accepted.

3. Cut off and abandon rejected piles after installation, and replace with new piles. Cutoff shall be at a sufficient depth to avoid transfer of load from the structure to the abandoned pile.
4. Piles located within ten feet (10') of existing structures shall be installed in one continuous operation. Re-stroking piles during construction due to auger obstructions or difficulty in installation of reinforcement cages will not be allowed. The structural engineer shall be consulted in the event that replacement piles are required.

E. Cutting-Off
1. Adjust the tops of pile to the cut-off elevations where piles are constructed from a work platform above final subgrade, by removing fresh grout from the top of the pile after the CMP sleeve is in place.
2. Cut off hardened grout and the CMP shell down to final cutoff point after initial set has occurred for all piles in a single cap, or within 15 ft of any pile in a spaced pattern.

F. Disposal
1. The Contractor shall remove and dispose all spoils and grout off site.
2. The Contractor shall determine if any excavated material is contaminated, and if any contaminated material is encountered it shall be disposed of in a method acceptable to all governmental authorities having jurisdiction.

PART 4: MEASUREMENT AND PAYMENT

4.1 MEASUREMENT

A. Each compression pile and each tension pile successfully installed in accordance with the Contractor's design and using the methods and practices of the approved test piles, cut off at the proper elevation, including steel reinforcing, and all records and grout testing specified, shall be considered a single unit price item. Pile design, materials testing, and the Contractor's inspection are considered incidental to construction and shall not be separately measured for payment. Damaged piles and piles installed outside the required installation tolerances will not be measured for payment. Short piles caused by obstructions and meeting the requirements of Part 3.5D shall be measured for payment.

B. Each successful compression, tension and lateral pre-construction load test performed, including load frame and/or reaction piles, test pile, testing, and load test report, shall be considered a single unit price item.

C. Each successful compression, tension and lateral construction quick load test performed, including load frame and/or reaction piles, test pile, testing, and load test report, shall be considered a single unit price item.

4.2 PAYMENT

A. Each compression pile and each tension pile, approved and accepted by the Owner, shall be paid at the unit price indicated on the bid form.

B. Each successful pile load test, approved and accepted by the Owner, shall be paid at the unit prices indicated on the bid form.

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WATER SUPPLY TEST - DEPARTMENT OF UTILITIES

City of Sacramento Development Services		TEST NUMBER:	14-049	FILE NUMBER:	R14-049
Planning & Building Department		COMPLETE DATE:		PC NUMBER:	wst-1409771
300 Richards Blvd., 3rd Floor		ANALYSIS FEE:	$420.00	DATE PAID:	8/7/2014
Sacramento, CA 95811		FIELD TEST FEE:	$774.00	DATE PAID:	8/7/2014
CONTACT:	Richard Chavez	PHONE NUMBER:	916-788-2884	FAX NUMBER:	916-788-4408
COMPANY:	RSC Engineering Inc.	CELL NUMBER:	916-580-9724	EMAIL: R.Chavez@rsc-engr.com	
COMPLETE CO.	2250 Douglas Blvd. Ste. 150	STREET ADDRESS OF TEST:		2015 L Street	
ADDRESS:	Roseville, CA 95661	ASSESSOR'S PARCEL NUMBER:		007-0086-012,018,021, & 022	

When more than one water supply test has been performed contact your Fire Planchecker to determine which test to use.

The undersigned agrees to the following items and conditions:

(1) *The street address shown above is correct*

(2) *Water supply data is developed from several sources of information which may include water supply test data pipe network, computer models, and continuous pressure recording stations. The design water supply data given below is to be used for design purposes.*

(3) *Although the water supply data reported herein is believed to be accurate, the City makes no warranty, guaranty, certification or other representation of any kind that such data is accurate or correct, or that the pressures and/or flow rates reported herein can or will be maintained. The undersigned agrees that the City, its officers and employees shall not be liable for any damages of any kind resulting from the use of or reliance upon the water supply data reported herein by the undersigned or by any third party.*

(4) *When more than one water supply test has been performed, the decision is left to the Fire Plan Checker as to which water supply test is to be used.*

(5) *If the undersigned desires to witness the water supply test performed by the City, please check the box below:*

☐ *I want to witness this water supply test, which will be scheduled at the convenience of the Department of Utilities.*

(6) *If the undersigned elects to hire a licensed engineer, at the undersigned's sole expense, to witness and certify the water supply test performed by the City, please check the box below:*

☐ *At my expense, I will arrange for a licensed engineer to witness and certify this water supply test, which will be scheduled at the convenience of the Department of Utilities.*

PRINT NAME:		SIGNATURE:	
DATE:		FIELD REQUEST DATE:	
DATE OF TEST:	8/15/2014	TIME OF TEST:	5:43 AM
WTR. MAIN SIZE:	6"	TEST CONDUCTED BY:	SAL MIANO

	Hydrant Number	Map Page	Static Pres. (PSI)	Residual Pres. (PSI)	Pitot Pres. (PSI)	Outlet Dia. (Inches)	Coefficient C_1	Coefficient C_2	Calc. Flow @ Pres. (GPM)	Flow @ 20 PSI (G.P.M.)
Residual	601	DD15	56	38						
Flowed	402	DD16			16	2.5	0.90	1.00	671	884
Flowed	401	DD16			12	2.5	0.90	1.00	581	766
Flowed										
Flowed										

* THE WATER SUPPLY TEST DATA IS NOT TO BE USED FOR THE DESIGN OF DOMESTIC WATER SYSTEMS.

* (STATIC PRES. - RESIDUAL PRES.) / (STATIC PRES. - 20 PSI) MUST NOT BE LESS THAN 25%. THEREFORE, THESE RESULTS ARE ONLY VALID FOR RESIDUAL PRESSURES LESS THAN 47 PSI

WATER SUPPLY DATA SUMMARY

	Design (1)
Static Pressure	50 PSI
Residual Pressure	32 PSI
Total Flow @ Residual	1300 G.P.M.
Total Flow @ 20 PSI	1700 G.P.M.

(1) The Design Water Supply Data reflects fluctuations and future demands on the water distribution system. It is to be used for design purposes. 7/2

8/18/2014

WATER SUPPLY TEST - DEPARTMENT OF UTILITIES

City of Sacramento Development Services	TEST NUMBER: 50	FILE NUMBER: R14-049
Planning & Building Department	COMPLETE DATE:	PC NUMBER: wst-1409771
300 Richards Blvd., 3rd Floor	ANALYSIS FEE: $420.00	DATE PAID: 8/7/2014
Sacramento, CA 95811	FIELD TEST FEE: $774.00	DATE PAID: 8/7/2014
CONTACT: Richard Chavez	PHONE NUMBER: 916-788-2884	FAX NUMBER: 916-788-4408
COMPANY: RSC Engineering Inc.	CELL NUMBER: 916-580-9724	EMAIL: R.Chavez@rsc-engr.com
COMPLETE CO. 2250 Douglas Blvd. St	STREET ADDRESS OF TEST: 2015 L Street	
ADDRESS: Roseville, CA 95661	ASSESSOR'S PARCEL NUMBER: 007-0086-012,018,021, & 022	

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(4) When more than one water supply test has been performed, the decision is left to the Fire Plan Checker as to which water supply test is to be used.

(5) If the undersigned desires to witness the water supply test performed by the City, please check the box below:

☐ *I want to witness this water supply test, which will be scheduled at the convenience of the Department of Utilities.*

(6) If the undersigned elects to hire a licensed engineer, at the undersigned's sole expense, to witness and certify the water supply test performed by the City, please check the box below:

☐ *At my expense, I will arrange for a licensed engineer to witness and certify this water supply test, which will be scheduled at the convenience of the Department of Utilities.*

PRINT NAME:	SIGNATURE:
DATE:	FIELD REQUEST DATE:
DATE OF TEST: 8/15/2014	TIME OF TEST: 5:25 AM
WTR. MAIN SIZE: 12" & 8"	TEST CONDUCTED BY: SAL MIANO

	Hydrant Number	Map Page	Static Pres. (PSI)	Residual Pres. (PSI)	Pitot Pres. (PSI)	Outlet Dia. (Inches)	Coefficient C_1	Coefficient C_2	Calc. Flow @ Pres. (GPM)	Flow @ 20 PSI (G.P.M.)
Residual	406	DD16	55	34						
Flowed	407	DD16			23	4.5	0.90	0.83	2164	2624
Flowed	405	DD16			21	4.5	0.90	0.83	2068	2507
Flowed										
Flowed										

* THE WATER SUPPLY TEST DATA IS NOT TO BE USED FOR THE DESIGN OF DOMESTIC WATER SYSTEMS.

* (STATIC PRES. - RESIDUAL PRES.) / (STATIC PRES. - 20 PSI) MUST NOT BE LESS THAN 25%. THEREFORE, THESE RESULTS ARE ONLY VALID FOR RESIDUAL PRESSURES LESS THAN 46 PSI

WATER SUPPLY DATA SUMMARY

	Design (1)
Static Pressure	50 PSI
Residual Pressure	29 PSI
Total Flow @ Residual	4200 G.P.M.
Total Flow @ 20 PSI	5100 G.P.M.

(1) The Design Water Supply Data reflects fluctuations and future demands on the water distribution system. It is to be used for design purposes.

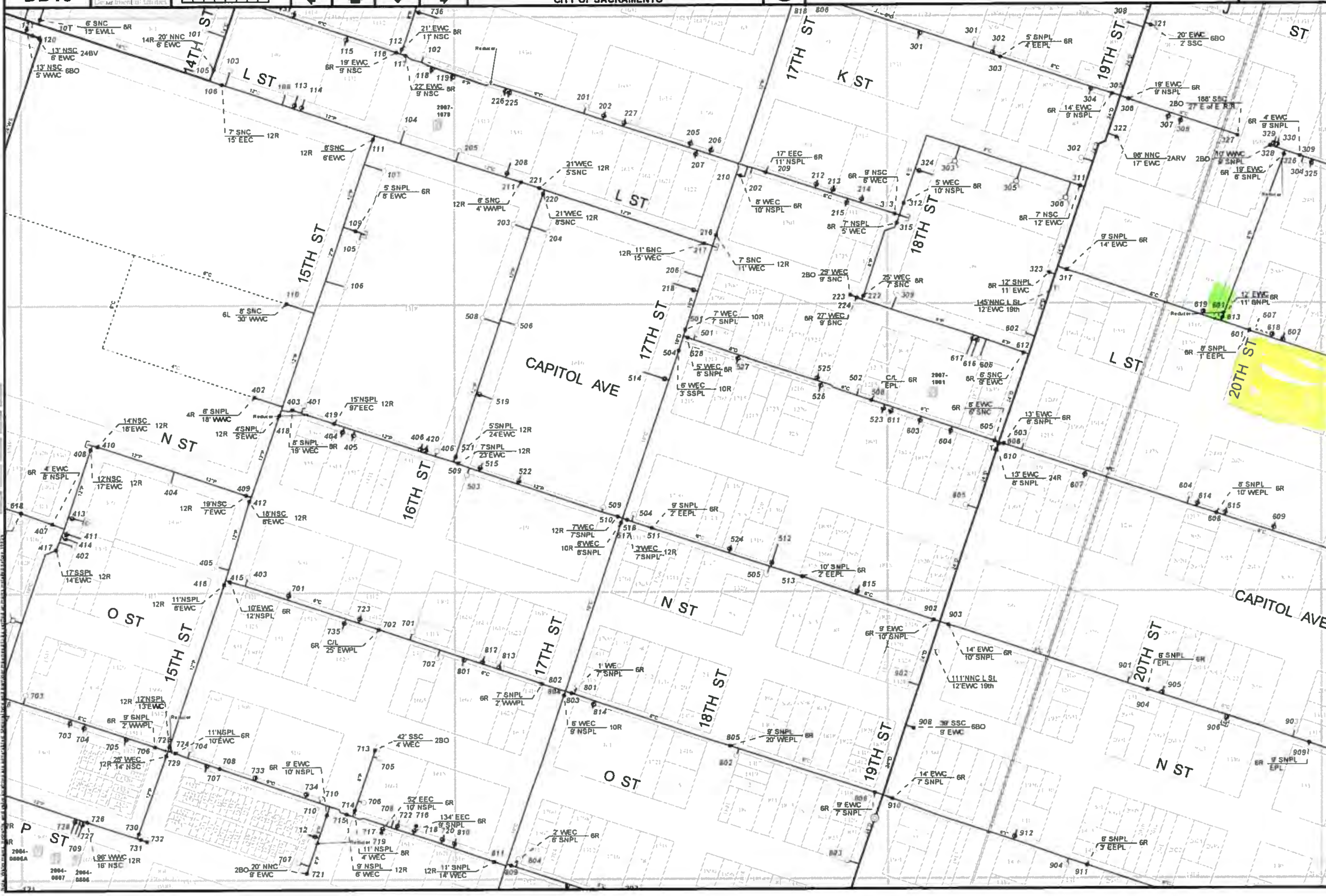
8/18/2014

DD15 | SACRAMENTO Department of Utilities | Map Created: 5/14/2014 | DD14 | CC15 | EE15 | DD16

WATER MAP 2014

CITY OF SACRAMENTO

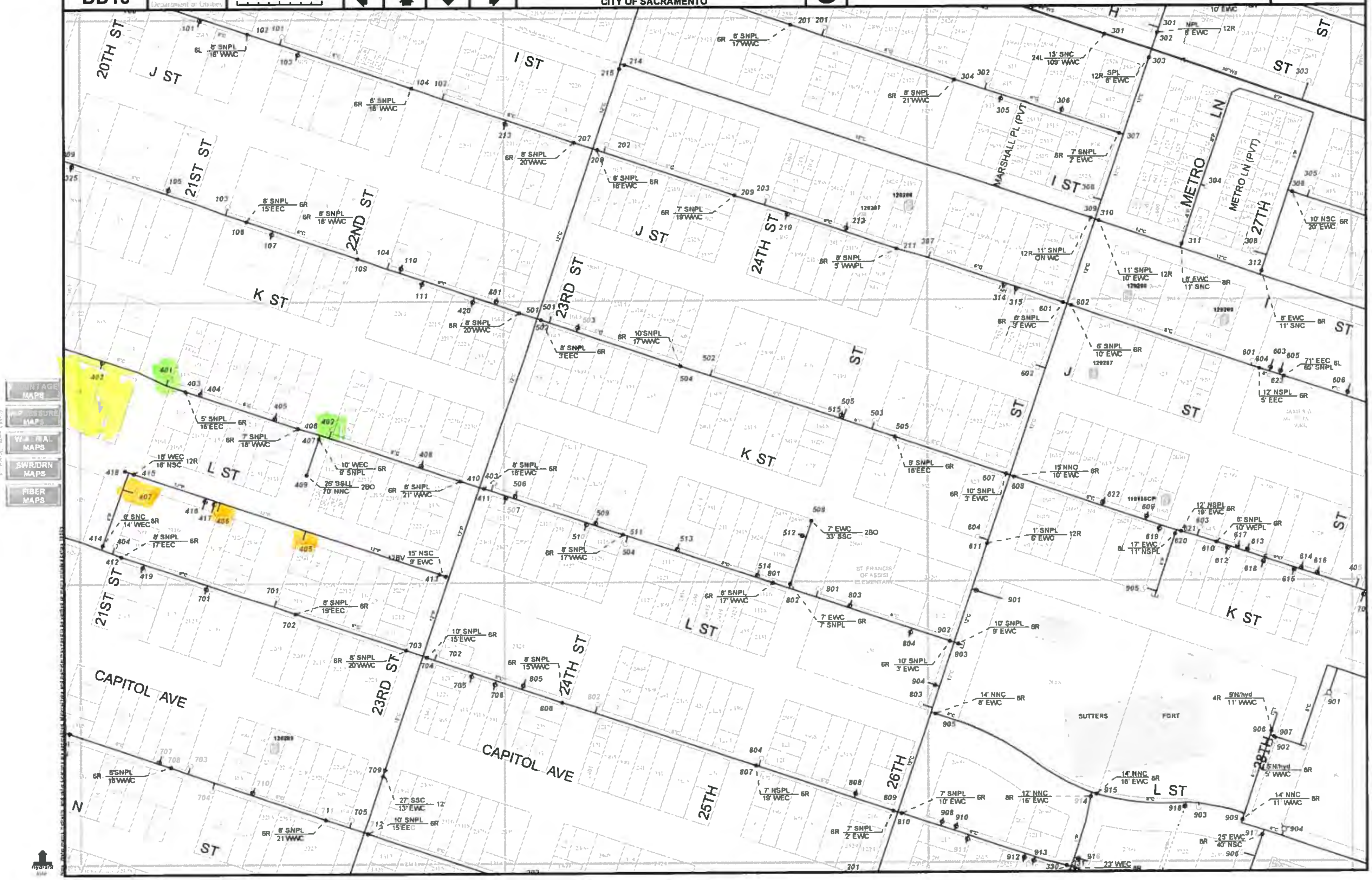
DD15



DD16 | SACRAMENTO Department of Utilities | Map Created: 4/4/2014 | DD15 | CC16 | EE16 | DD17

WATER MAP 2014
CITY OF SACRAMENTO

DD16



WATER SUPPLY TEST - DEPARTMENT OF UTILITIES

City of Sacramento Development Services		TEST NUMBER:	14-051	FILE NUMBER:	R14-051
Planning & Building Department		COMPLETE DATE:	9/15/2014	PC NUMBER:	wst-1410424
300 Richards Blvd., 3rd Floor		ANALYSIS FEE:	$420.00	DATE PAID:	8/22/2014
Sacramento, CA 95811		FIELD TEST FEE:	$774.00	DATE PAID:	8/22/2014
CONTACT:	Richard Chavez	PHONE NUMBER:	916-788-2884	FAX NUMBER:	916-788-4408
COMPANY:	RSC Engineering Inc.	CELL NUMBER:	916-580-9724	EMAIL: R.Chavez@rsc-engr.com	
COMPLETE CO.	2250 Douglas Blvd. # 150	STREET ADDRESS OF TEST:		1223 21st Street & 2101 & 2117 Capitol Avenue	
ADDRESS:	Roseville, CA 95661	ASSESSOR'S PARCEL NUMBER:		007-0151-025, 026 & 027	

When more than one water supply test has been performed contact your Fire Planchecker to determine which test to use.

The undersigned agrees to the following items and conditions:

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(2) *Water supply data is developed from several sources of information which may include water supply test data pipe network, computer models, and continuous pressure recording stations. The design water supply data given below is to be used for design purposes.*

(3) *Although the water supply data reported herein is believed to be accurate, the City makes no warranty, guaranty, certification or other representation of any kind that such data is accurate or correct, or that the pressures and/or flow rates reported herein can or will be maintained. The undersigned agrees that the City, its officers and employees shall not be liable for any damages of any kind resulting from the use of or reliance upon the water supply data reported herein by the undersigned or by any third party.*

(4) *When more than one water supply test has been performed, the decision is left to the Fire Plan Checker as to which water supply test is to be used.*

(5) *If the undersigned desires to witness the water supply test performed by the City, please check the box below:*

☐ *I want to witness this water supply test, which will be scheduled at the convenience of the Department of Utilities.*

(6) *If the undersigned elects to hire a licensed engineer, at the undersigned's sole expense, to witness and certify the water supply test performed by the City, please check the box below:*

☐ *At my expense, I will arrange for a licensed engineer to witness and certify this water supply test, which will be scheduled at the convenience of the Department of Utilities.*

PRINT NAME:		SIGNATURE:	
DATE:		FIELD REQUEST DATE:	
DATE OF TEST:	9/10/2014	TIME OF TEST:	5:57 a.m.
WTR. MAIN SIZE:	6"	TEST CONDUCTED BY:	Sal Miano

	Hydrant Number	Map Page	Static Pres. (PSI)	Residual Pres. (PSI)	Pitot Pres. (PSI)	Outlet Dia. (Inches)	Coefficient C_1	Coefficient C_2	Calc. Flow @ Pres. (GPM)	Flow @ 20 PSI (G.P.M.)
Residual	404	DD16	50	42						
Flowed	701	DD16			31	2.5	0.90	1.00	934	1907
Flowed	604	DD15			30	2.5	0.90	1.00	919	1876
Flowed										
Flowed										

* THE WATER SUPPLY TEST DATA IS NOT TO BE USED FOR THE DESIGN OF DOMESTIC WATER SYSTEMS.

* (STATIC PRES. - RESIDUAL PRES.) / (STATIC PRES. - 20 PSI) MUST NOT BE LESS THAN 25%. THEREFORE, THESE RESULTS ARE ONLY VALID FOR RESIDUAL PRESSURES LESS THAN 43 PSI

WATER SUPPLY DATA SUMMARY

	Design (1)
Static Pressure	50 PSI
Residual Pressure	42 PSI
Total Flow @ Residual	1900 G.P.M.
Total Flow @ 20 PSI	3800 G.P.M.

(1) The Design Water Supply Data reflects fluctuations and future demands on the water distribution system. It is to be used for design purposes. WO#262788

9/15/2014

WATER SUPPLY TEST - DEPARTMENT OF UTILITIES

City of Sacramento Development Services		TEST NUMBER:	14-052	FILE NUMBER:	R14-052
Planning & Building Department		COMPLETE DATE:	9/15/2014	PC NUMBER:	wst-1410425
300 Richards Blvd., 3rd Floor		ANALYSIS FEE:	$420.00	DATE PAID:	8/22/2014
Sacramento, CA 95811		FIELD TEST FEE:	$774.00	DATE PAID:	8/22/2014
CONTACT:	Richard Chavez	PHONE NUMBER:	916-788-2884	FAX NUMBER:	916-788-4408
COMPANY:	RSC Engineering Inc.	CELL NUMBER:	916-580-9724	EMAIL: R.Chavez@rsc-engr.com	
COMPLETE CO.	2250 Douglas Blvd. #150	STREET ADDRESS OF TEST:		1216 21st Street	
ADDRESS:	Roseville, CA 95661	ASSESSOR'S PARCEL NUMBER:		007-0145-014-0000	

When more than one water supply test has been performed contact your Fire Planchecker to determine which test to use.

The undersigned agrees to the following items and conditions:

(1) The street address shown above is correct

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PRINT NAME:		SIGNATURE:	
DATE:		FIELD REQUEST DATE:	
DATE OF TEST:	9/10/2014	TIME OF TEST:	5:40 a.m.
WTR. MAIN SIZE:	6"	TEST CONDUCTED BY:	Sal Miano

	Hydrant Number	Map Page	Static Pres. (PSI)	Residual Pres. (PSI)	Pitot Pres. (PSI)	Outlet Dia. (Inches)	Coefficient C_1	Coefficient C_2	Calc. Flow @ Pres. (GPM)	Flow @ 20 PSI (G.P.M.)
Residual	604	DD15	50	42						
Flowed	603	DD15			40	2.5	0.90	1.00	1061	2167
Flowed	404	DD16			37	2.5	0.90	1.00	1021	2084
Flowed										
Flowed										

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WATER SUPPLY DATA SUMMARY

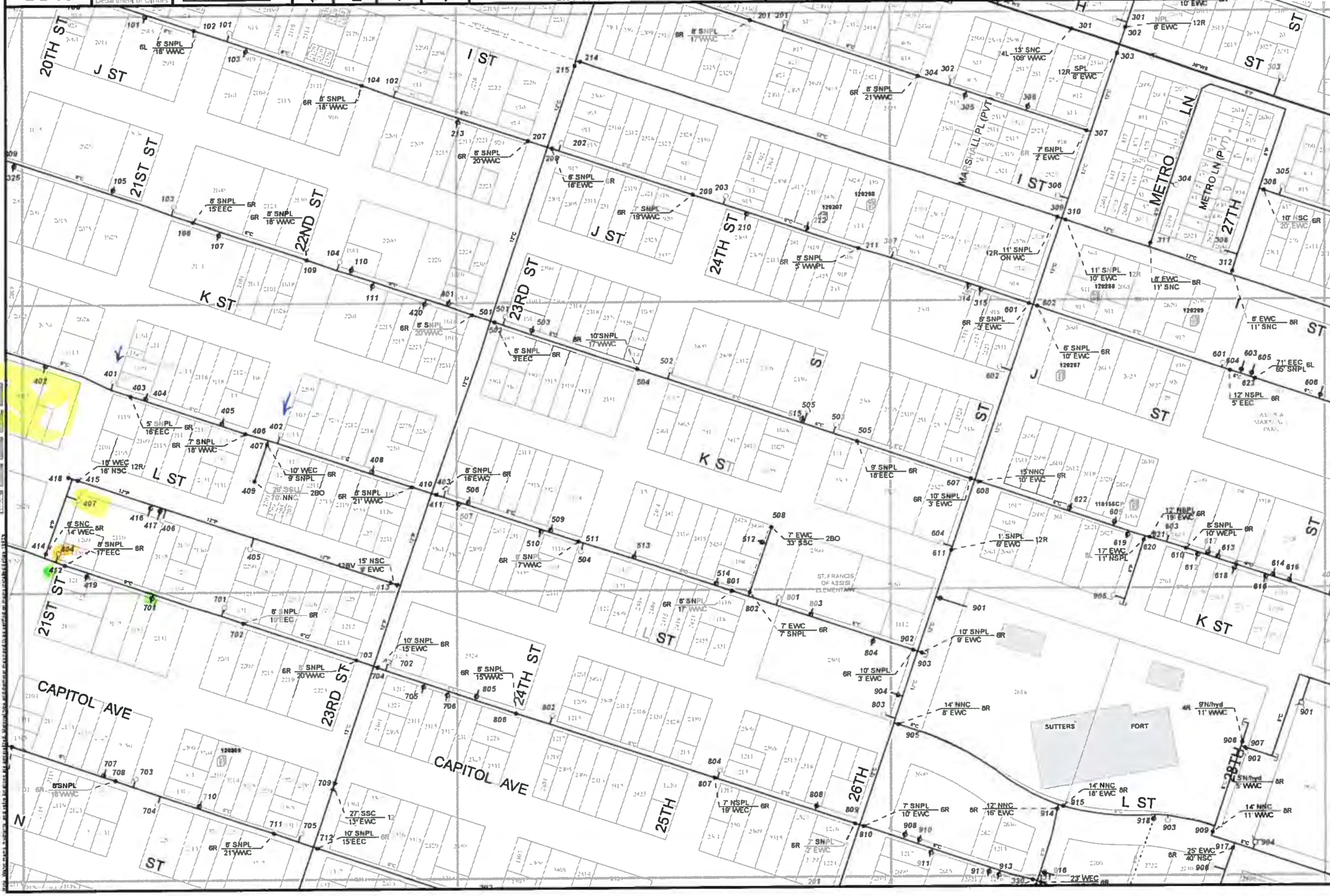
	Design (1)
Static Pressure	50 PSI
Residual Pressure	42 PSI
Total Flow @ Residual	2100 G.P.M.
Total Flow @ 20 PSI	4300 G.P.M.

(1) The Design Water Supply Data reflects fluctuations and future demands on the water distribution system. It is to be used for design purposes. WO#262793 9/15/2014

DD16 | SACRAMENTO Department of Utilities | Map Created: 4/4/2014 | DD15 | CC16 | EE16 | DD17

WATER MAP 2014
CITY OF SACRAMENTO

DD16

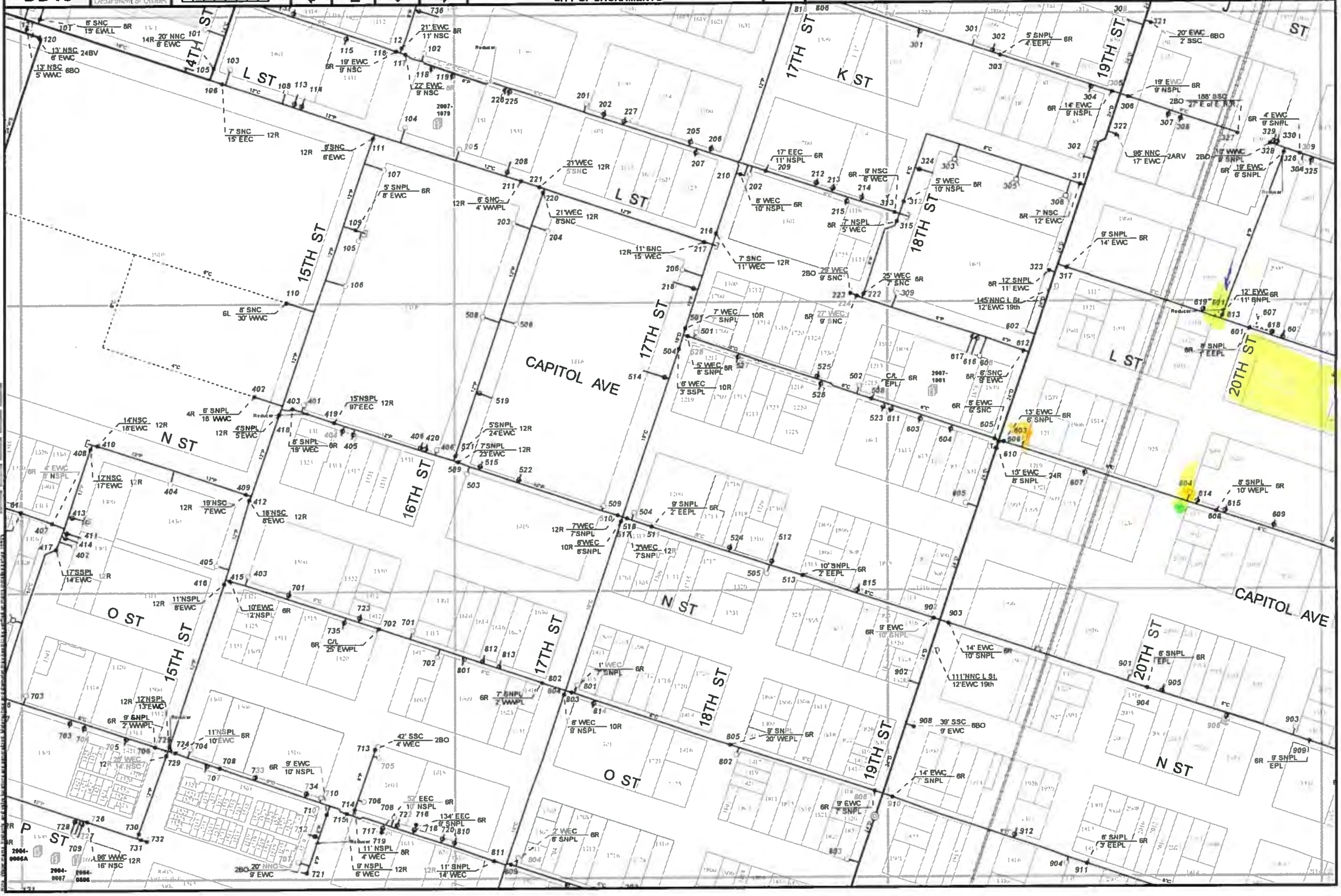


W-MAIN / AGE MAPS
W-PRESSURE MAPS
W-AERIAL MAPS
SWR/DRN MAPS
FIBER MAPS

DD15 | SACRAMENTO Department of Utilities | Map Created: 5/14/2014 0 50 100 200 Feet | DD14 | CC15 | EE15 | DD16

WATER MAP 2014
CITY OF SACRAMENTO

DD15



WATER SUPPLY TEST - DEPARTMENT OF UTILITIES

City of Sacramento Development Services		TEST NUMBER:	14-049	FILE NUMBER:	R14-049
Planning & Building Department		COMPLETE DATE:		PC NUMBER:	wst-1409771
300 Richards Blvd., 3rd Floor		ANALYSIS FEE:	$420.00	DATE PAID:	8/7/2014
Sacramento, CA 95811		FIELD TEST FEE:	$774.00	DATE PAID:	8/7/2014
CONTACT:	Richard Chavez	PHONE NUMBER:	916-788-2884	FAX NUMBER:	916-788-4408
COMPANY:	RSC Engineering Inc.	CELL NUMBER:	916-580-9724	EMAIL: R.Chavez@rsc-engr.com	
COMPLETE CO.	2250 Douglas Blvd. Ste. 150	STREET ADDRESS OF TEST:		2015 L Street	
ADDRESS:	Roseville, CA 95661	ASSESSOR'S PARCEL NUMBER:		007-0086-012,018,021, & 022	

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PRINT NAME:		SIGNATURE:	
DATE:		FIELD REQUEST DATE:	
DATE OF TEST:	8/15/2014	TIME OF TEST:	5:43 AM
WTR. MAIN SIZE:	6"	TEST CONDUCTED BY:	SAL MIANO

	Hydrant Number	Map Page	Static Pres. (PSI)	Residual Pres. (PSI)	Pitot Pres. (PSI)	Outlet Dia. (Inches)	Coefficient C_1	Coefficient C_2	Calc. Flow @ Pres. (GPM)	Flow @ 20 PSI (G.P.M.)
Residual	601	DD15	56	38						
Flowed	402	DD16			16	2.5	0.90	1.00	671	884
Flowed	401	DD16			12	2.5	0.90	1.00	581	766
Flowed										
Flowed										

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WATER SUPPLY DATA SUMMARY

	Design (1)
Static Pressure	50 PSI
Residual Pressure	32 PSI
Total Flow @ Residual	1300 G.P.M.
Total Flow @ 20 PSI	1700 G.P.M.

(1) The Design Water Supply Data reflects fluctuations and future demands on the water distribution system. It is to be used for design purposes. 7/2

8/18/2014

WATER SUPPLY TEST - DEPARTMENT OF UTILITIES

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Planning & Building Department	COMPLETE DATE:	PC NUMBER: wst-1409771
300 Richards Blvd., 3rd Floor	ANALYSIS FEE: $420.00	DATE PAID: 8/7/2014
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COMPLETE CO. 2250 Douglas Blvd. St	STREET ADDRESS OF TEST: 2015 L Street	
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DATE OF TEST: 8/15/2014	TIME OF TEST: 5:25 AM
WTR. MAIN SIZE: 12" & 8"	TEST CONDUCTED BY: SAL MIANO

	Hydrant Number	Map Page	Static Pres. (PSI)	Residual Pres. (PSI)	Pitot Pres. (PSI)	Outlet Dia. (Inches)	Coefficient C_1	Coefficient C_2	Calc. Flow @ Pres. (GPM)	Flow @ 20 PSI (G.P.M.)
Residual	406	DD16	55	34						
Flowed	407	DD16			23	4.5	0.90	0.83	2164	2624
Flowed	405	DD16			21	4.5	0.90	0.83	2068	2507
Flowed										
Flowed										

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WATER SUPPLY DATA SUMMARY

	Design (1)
Static Pressure	50 PSI
Residual Pressure	29 PSI
Total Flow @ Residual	4200 G.P.M.
Total Flow @ 20 PSI	5100 G.P.M.

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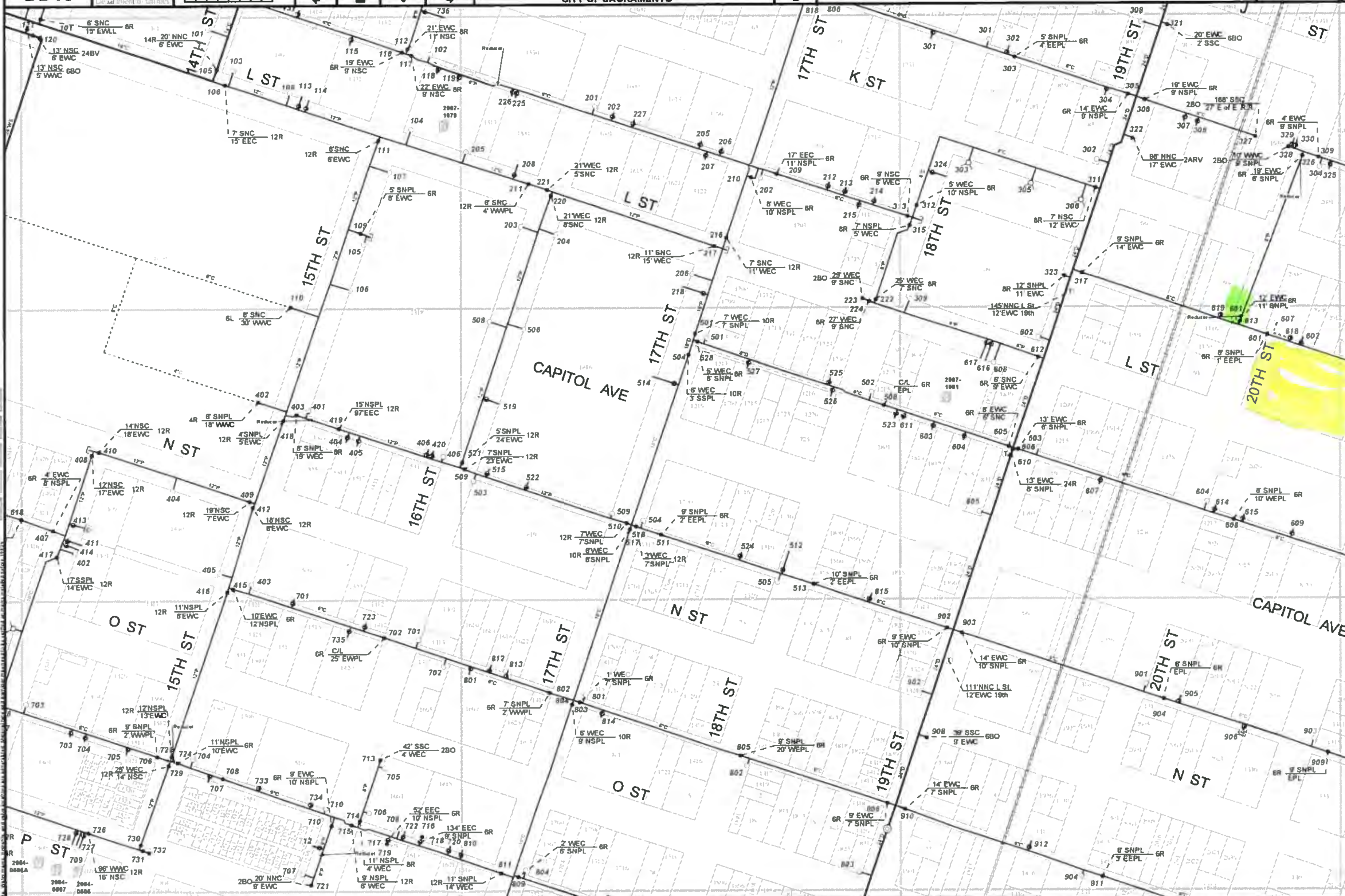
8/18/2014

DD15 | SACRAMENTO | Map Created: 5/14/2014 | DD14 | CC15 | EE15 | DD16

WATER MAP 2014

CITY OF SACRAMENTO

DD15



DD16 | SACRAMENTO Department of Utilities | Map Created: 4/4/2014 | DD15 | CC16 | EE16 | DD17

WATER MAP 2014

CITY OF SACRAMENTO

DD16