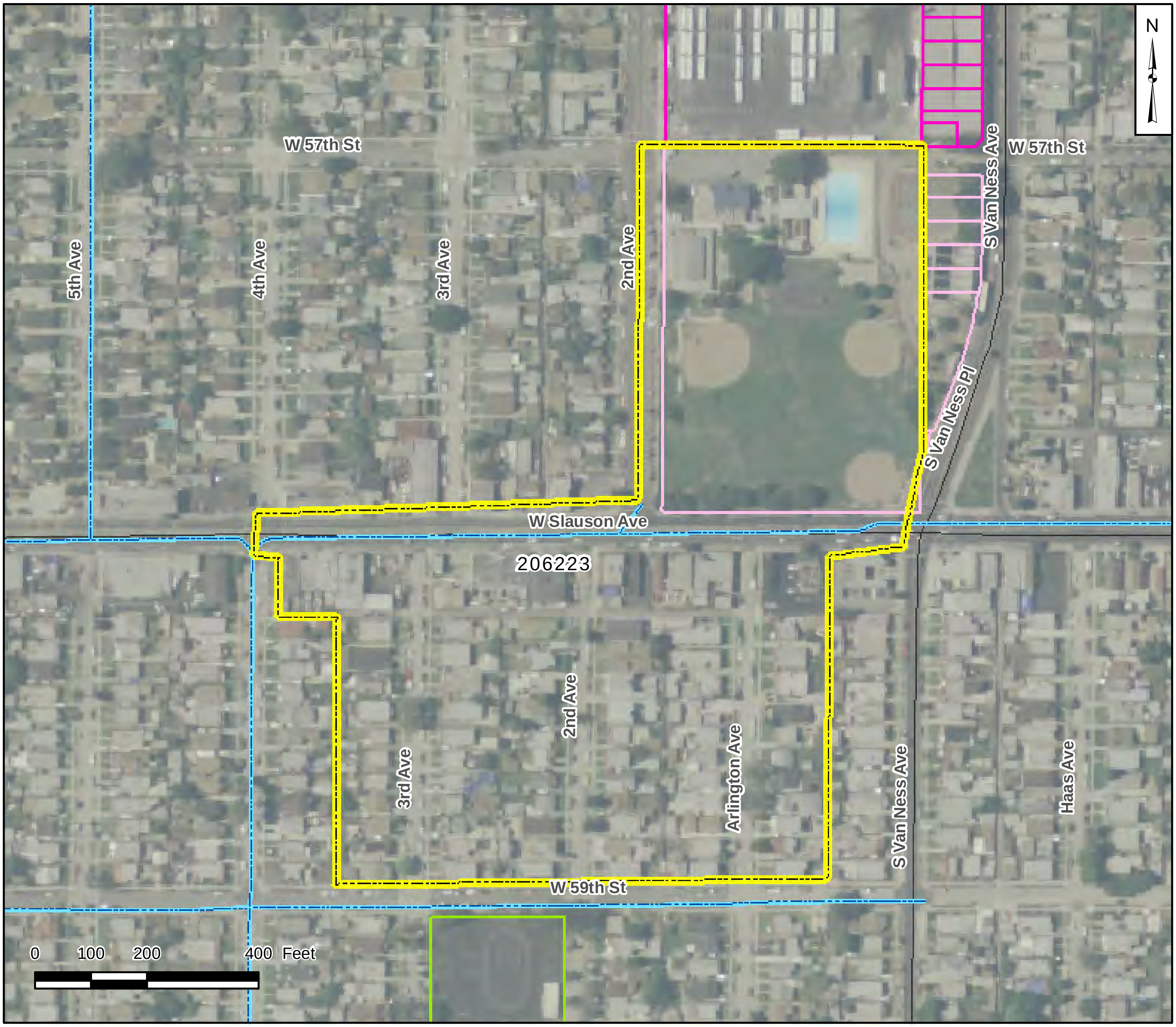
DRAFT

FOR DISCUSSION PURPOSES ONLY

Ballona Creek TMDL Implementation Plan

# Regional Priority Catchments

## Catchment 206223



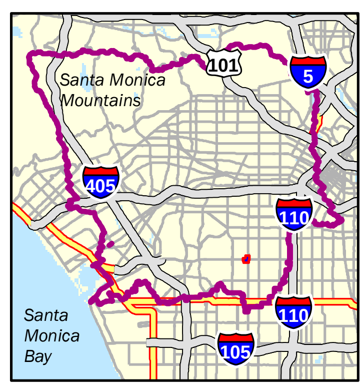

Legend

Priority Regional BMP Catchment

Stormdrains

Parcel Ownership

Other, Private - Com/Ind, Private - Other, Public - City, Public - County, Public - Federal, Public - Other, Public - State

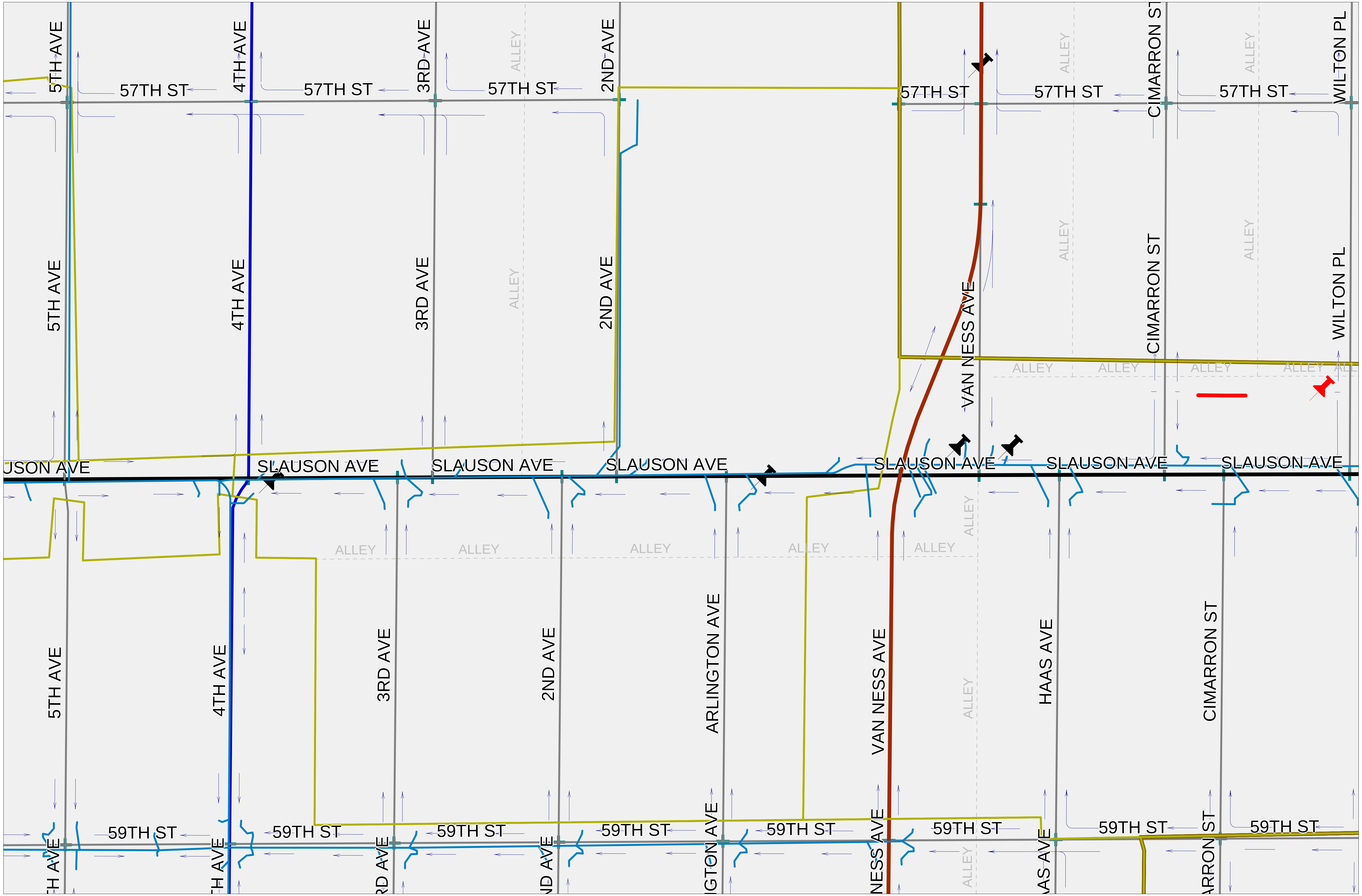



| | | |
|---|---|---|
| Regional BMP Score : | 5 | |
| Nodal CPI Score : | 4 | |
| Acreage : | 21.87 | |

| Polutant-Specific CPI Score | | | | | |
|---|---|---|---|---|---|
| Copper : | 2 | Zinc : | 1 | Fecal Coli : | 4 |
| Lead : | 2 | TSS : | 2 | | |

W 54th St
Arlington Ave
4th Ave
W 57th St
7th Ave
6th Ave
3rd Ave
2nd Ave
S Van Ness Pl
S Van Ness Ave
Cimarron St
W Slauson Ave
5th Ave
©2009 Tele Atlas
Google

# R206223




SCALE 1 : 2,071

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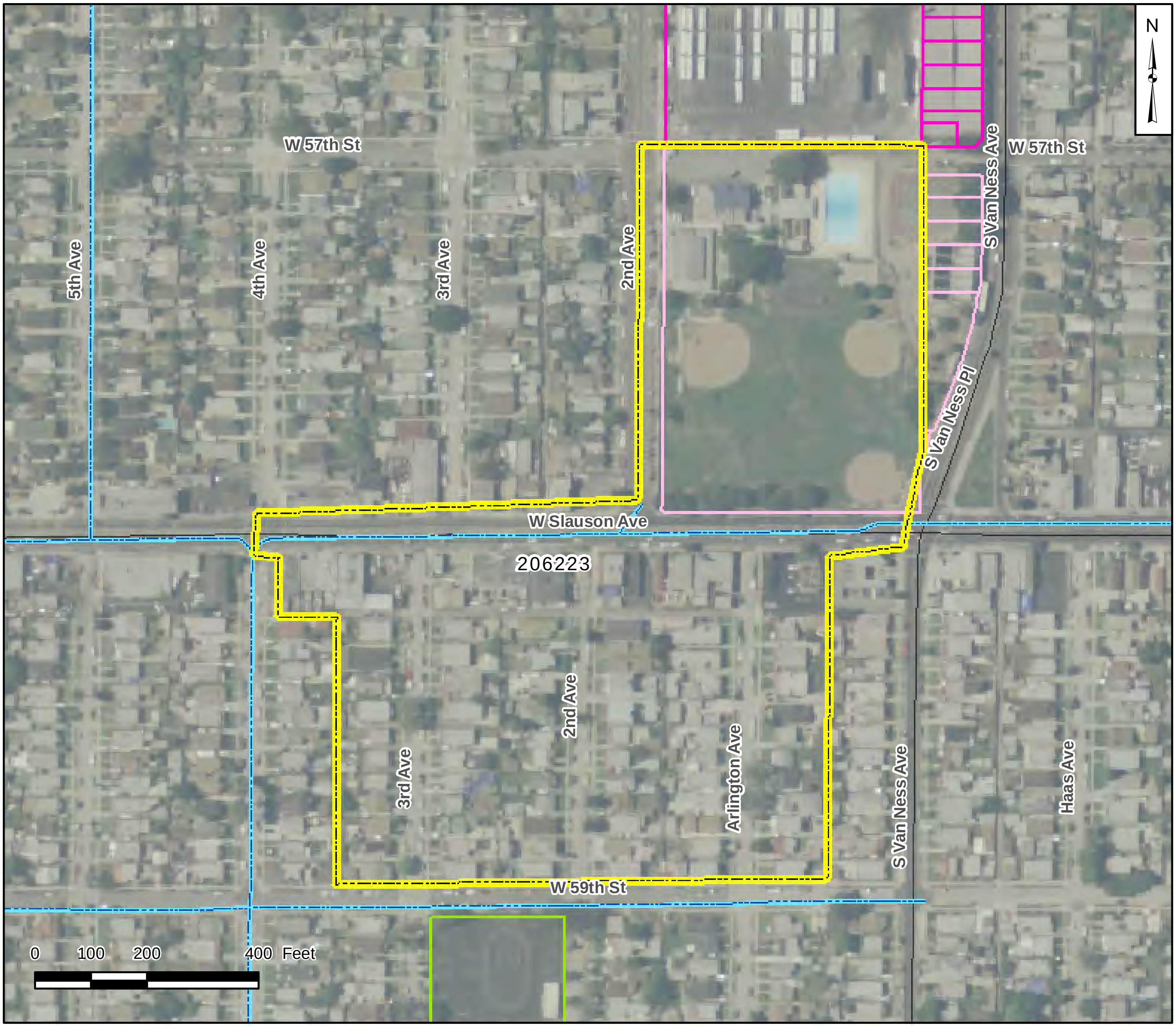
FOR DISCUSSION PURPOSES ONLY

Ballona Creek TMDL Implementation Plan

# Regional Priority Catchments

## Catchment 206223



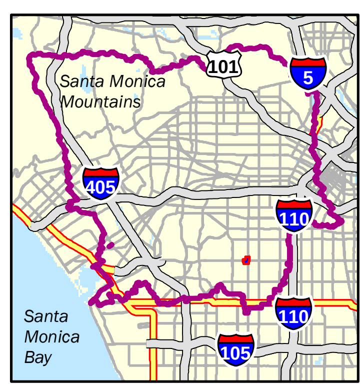

Legend

Priority Regional BMP Catchment

Stormdrains

Parcel Ownership

Other, Private - Com/Ind, Private - Other, Public - City, Public - County, Public - Federal, Public - Other, Public - State




| | | Polutant-Specific CPI Score | | | | | |
|---|---|---|---|---|---|---|---|
| Regional BMP Score : | 5 | Copper : | 2 | Zinc : | 1 | Fecal Coli : | 4 |
| Nodal CPI Score : | 4 | Lead : | 2 | TSS : | 2 | | |
| Acreage : | 21.87 | | | | | | |


57TH ST
57TH ST
57TH ST
57TH ST
3RD AVE
2ND AVE
VAN NESS AVE
135
SLAUSON AVE
ARLINGTON AVE
HAAS AVE

3RD AVE
2ND AVE
57TH ST
55802461212086
135
VAN NESS AVE
CIMARRON ST
D-7500
55802461111088
D-83
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55802461111110
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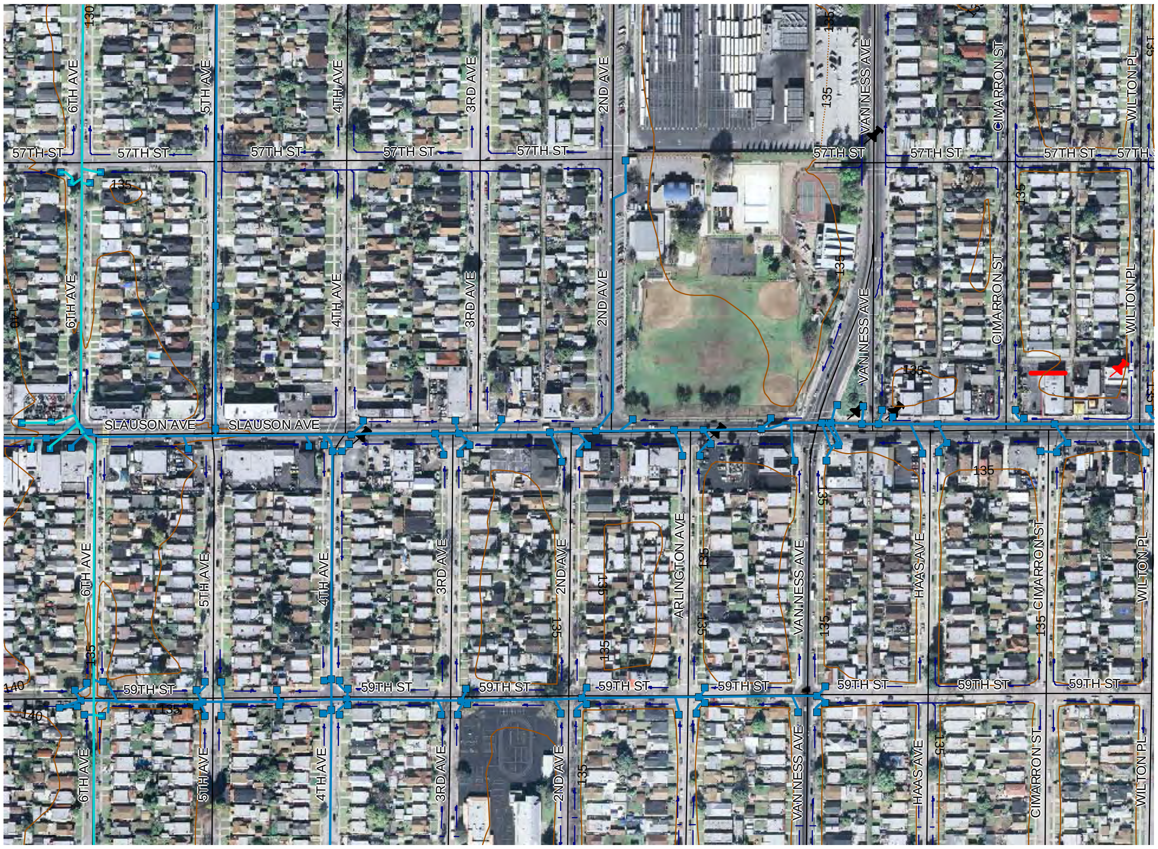
57TH ST
SLAUSON AVE
59TH ST
6TH AVE
5TH AVE
4TH AVE
3RD AVE
2ND AVE
ARLINGTON AVE
VAN NESS AVE
HAAS AVE
CIMARRON ST
WILTON PL
135
130
140

**CITY OF LOS ANGELES**

**DEPARTMENT OF PUBLIC WORKS**

**BUREAU OF ENGINEERING**

# GEOTECHNICAL ENGINEERING DIVISION

**GEOTECHNICAL ENGINEERING REPORT**
**NEW MODERN GYM**
**VAN NESS RECREATION CENTER**
**5720 S. 2ND AVENUE (TRACT 1093, LOT 33)**
**LOS ANGELES, CALIFORNIA**

**W.O. # E170276D**
**GED FILE # 02-070**
**AUGUST 13, 2002**

**TABLE OF CONTENTS**

1.0 INTRODUCTION....1
2.0 PROJECT SCOPE....1
3.0 EXPLORATION PROGRAM....1
4.0 LABORATORY TESTING....2
Table 1 – Soil Design Parameters....3
5.0 REGIONAL GEOLOGY....3
6.0 SITE CONDITIONS....3
6.1 Surface Conditions....3
6.2 Subsurface Conditions....3
6.3 Groundwater....4
7.0 FAULTING AND SEISMICITY....4
7.1 Ground Motion....5
7.2 Liquefaction....5
7.3 Dynamic Settlement....6
7.4 Other Seismic Hazards....6
7.5 Seismic Design Parameters....7
Table 2 – Seismic Design Data....7
8.0 SITE RECOMMENDATIONS....8
8.1 General....8
8.2 Site Preparation and Earthwork....8
8.2.1 Site Clearing....8
8.2.2 Over-Excavation....9
8.2.3 Temporary Excavations....9
8.2.4 Slot Cuts....9
8.2.5 Subgrade Preparation....10
8.2.6 Fill Materials....11
8.2.7 Fill and Backfill Placement....11
8.2.8 Trench Backfill....11
8.2.9 Fill Certification....11
8.3 Structure Foundations....12
8.4 Planter and Fence Wall Foundations....12

8.5 Slab-on-Grade ........................................................................................13
8.6 Cement Type and Corrosion Measures ...........................................................13
9.0 SUPPLEMENTAL GEOTECHNICAL SERVICES ..................................................14
9.1 Review of Plans and Specifications ................................................................14
9.2 Geotechnical Observation and Testing During Construction ..........................14
9.3 Closure ........................................................................................................15

Plate 1 – Vicinity Map
Plate 2 – Boring Location Map
Plate 3 – Regional Geology
Plate 4 – Seismic Hazard Zones
Plate 5 – Regional Fault Map
Plate 6 – Active Fault Near Source Zones

Appendix A – URS Data Report

Appendix B- Probabilistic Seismic Hazard Analyses

## 1.0 INTRODUCTION

This report presents the results of a geotechnical investigation conducted for the proposed construction of a new gymnasium at the Van Ness Recreation Center located at 5720 S. 2nd Avenue in the City of Los Angeles. This investigation was conducted to determine subsurface characteristics and to provide geotechnical recommendations for design and construction of the project. The Geotechnical Engineering Division (GED) prepared this report in response to the Architectural Division request dated June 28, 2002 and notice to proceed dated July 17, 2002.

This report is based on visual observation, subsurface investigation and laboratory testing. At the request of GED, URS Corporation (URS) performed subsurface exploration at the site and laboratory testing of samples collected from the site. The results of their field investigation and laboratory tests are included in their Data Report (Appendix A) dated August 6, 2002 (received by GED on August 9, 2002). GED has reviewed this Data Report, concurs with and accepts responsibility for the use of its contents.

## 2.0 PROJECT SCOPE

The project will consist of the construction of a new modern gym at the Van Ness Recreation Center. The construction will take place entirely within the boundaries of the park. A vicinity map of the project site is shown on Plate 1. The proposed new gym will be located south of the existing Childcare Center, west of the exiting locker rooms and swimming pool, east of the existing gym, and north of the recreation areas.

The project will consist of removal of existing landscape and recreation features and construction of the new gym. The gym will be approximately 9,400 square feet with and indoor Recreation and Park regulation-size basketball court with dimensions of 50 feet by 84 feet and with additional space providing 80 seats along the sidelines for spectators. The new facility will also include space for an office, storage room for sporting equipment, electrical room, janitorial room, and ADA-compliance male and female restrooms. The structure will be two stories in height and consist of wood, masonry, concrete, and steel construction.

For the purpose of this report we have assumed that maximum column and wall loads will be on the order of 50 to 75 kips or less and 3 to 5 kips per linear foot or less, respectively. Final site grades are expected to be within 1 to 2 feet of current site grades. No subterranean construction is planned.

## 3.0 EXPLORATION PROGRAM

Four exploratory borings were drilled in the area of the proposed construction to depths of approximately 21.5 to 51.5 feet below the ground surface. The exploratory borings were drilled using a truck-mounted drill rig using 8-inch diameter conventional flight augers. Approximate locations of the borings are depicted on the Boring Location Map, Plate 2.

Standard Penetration Test (SPT) samples were collected from one boring (B-1) at depths of 2.5-feet, 5-feet, 7.5-feet, 10-feet, and every 5-feet thereafter to the explored depth for classification and laboratory testing. Ring samples were collected from the other three borings (B-2 through B-4) at depths of 2.5-feet, 5-feet, 7.5-feet, 10-feet, and every 5-feet thereafter to the explored depth for classification and laboratory testing. Samplers were driven into the bottom of the borings with successive drops of a 140-pound automatic hammer falling 30 inches. Blow counts required to advance the SPT sampler the last 12 inches of 18-inch sample interval is the SPT field "N" value. Blows required to advance the ring and SPT samplers the last 12-inches of the sample interval are shown on the boring logs in the "Blows per Foot" column (Appendix A). Bulk samples were also collected from the upper few feet of each boring.

Each collected sample was inspected and described in general conformance with the Unified Soil Classification System (USCS). The descriptions were entered on the boring logs, which are included within the Data Report presented in Appendix A of this report. All samples were sealed and packaged for transportation to the consultant's laboratory. After completion of drilling, borings were backfilled with soil cuttings.

Screening in the field for volatile organic compounds (VOCs) and selected laboratory testing of some soil samples was performed to evaluate whether fuel spills or other contamination of the soil has occurred in the project area. Soil samples were screened in the field with a portable Photo Ionizing Detector (PID) for the presence of VOCs. A multigas detector was used as a screening tool for methane (Lower Explosive Limit or LEL), oxygen, carbon dioxide and hydrogen sulfide. None of the samples registered VOC detection above background readings. No methane or hydrogen sulfide was detected. These results indicate no significant spills of fuel hydrocarbons, solvents or other VOC-containing compounds at the boring locations.

## 4.0 LABORATORY TESTING

Selected soil samples were tested for the following properties:

- In-Place Dry Density and Field Moisture (ASTM D2937)
- Laboratory Maximum Dry Density and Optimum Moisture Content (ASTM D1557)
- Consolidation (ASTM D2435)
- Direct Shear (ASTM D3080)
- Grain Size Analysis (ASTM D422)
- Percent Passing Number 200 Sieve
- Expansion Index
- pH (CA DOT Method)
- Sulfate Content (CA DOT Method)
- Chloride Content (CA DOT Method)
- Electrical Resistivity

Laboratory test results are presented in the enclosed Data Report (Appendix A). Soils parameters used for design purposes are summarized in Table 1, Soil Design Parameters.

Table 1 – Soil Design Parameters

| Material | Unit Weight | Cohesion | Friction |
|---|---|---|---|
| Compacted Fill | 120 pcf | 250 psf | 29 ° |
| Native | 110 pcf | 300 psf | 32 ° |

## 5.0 REGIONAL GEOLOGY

The subject site is located southeast of the Baldwin Hills within the Los Angeles Basin (Basin), a structural trough located within Southern California. The Basin is a northwest-trending alluviated lowland plain about 50 miles long and 20 miles wide. Mountains and hills that generally expose Late Cretaceous to Late Pleistocene-age sedimentary and igneous rocks bound the Basin along the north, northeast, east, and southeast.

The Basin is part of the Peninsular Ranges geomorphic province of California, which is characterized by subparallel blocks sliced longitudinally by young, steeply dipping northwest-trending fault zones. The Basin is interpreted to be as much as 31,000 feet thick in the center of the trough.

As shown on Plate 3, Regional Geology, the site area is mapped as being underlain with young alluvial valley (Qya2) materials. These deposits predominately consist of interlayered loose to dense sands and silts and stiff clays. The site area is relatively flat to gently sloping with ground surface elevations on the order of 132 feet above mean sea level. Drainage of the site area is toward the north.

No active faults are known or mapped as crossing the site. The Newport-Inglewood Fault is the closest known fault with a surface projection of potential rupture area that is located approximately 1.5 km (1mile) from the site.

## 6.0 SITE CONDITIONS

### 6.1 Surface Conditions

The project site is located within the Van Ness Recreation Center in the City of Los Angeles. The gym site is bounded by an existing Childcare Center to the north, a locker room and swimming pool to the east, recreational areas to the south and an existing gymnasium to the west. The site is relatively flat with surface elevations of approximately 132 feet above mean sea level (MSL). Sandboxes, landscaping, planter areas and hardscaping currently occupy the location of the proposed construction.

### 6.2 Subsurface Conditions

Uncertified fill materials, consisting primarily of brown fine sand and sandy silt, were observed to depths of 1 to 4 feet below the ground surface at three of the four boring locations. Natural alluvium materials generally consisted of sandy and clayey silt with interlayered silty clay and silty sand to a depth of approximately 32 feet in the deeper boring (B-1) and to the explored depths (21-1/2 feet) in the shallower borings (B-2 through B-4). The silts and clays were typically medium stiff to very stiff and the interlayered silty sand medium dense. Below a depth of approximately 32 feet in boring B-1, the soils in boring B-1 consisted primarily of dense to very dense sand to the explored depth of approximately 51-1/2 feet. More detailed descriptions of the soils can be found on the boring logs presented in the Geotechnical Data Report (Appendix A).

Based on the results of subsurface exploration and experience, variations in the continuity and depth of subsurface conditions should be anticipated. Care should be exercised in interpolating or extrapolating subsurface conditions between or beyond borings. Fill depths could vary greatly between borings and throughout the site.

### 6.3 Groundwater

Groundwater was encountered at a depth of approximately 46 feet in the deeper boring. The Los Angeles County office of Hydrologic Records has a monitoring well, Well Number 2679D, located at the intersection of 5th Avenue and 48th Street, approximately 3/4 miles north of the site. On May 16, 2002, groundwater was reportedly measured at a depth of approximately 165 feet below the ground surface in this well. The elevation of the ground surface at this well location is 130 feet. Groundwater data obtained from California Division of Mines and Geology (CDMG, 1998) indicates that the shallowest reported historic depth to groundwater in the site area was on the order of 10 feet below the ground surface.

## 7.0 FAULTING AND SEISMICITY

The fault classification system adopted by the CDMG, relative to the State legislation delineating the Earthquake Fault Zones along active or potentially active faults (Alquist-Priolo Act), is used for structures. CDMG defines an active fault (or fault zone) as a fault that has moved within Holocene time (about the last 11,000 years). Faults with no know displacement within Holocene time that showed evidence of movement during Quaternary time (the last 1.6 million years) have been defined as potentially active.

Ground surface rupturing along faults, ground shaking and liquefaction are three of the important seismic considerations for properties in Southern California. The site is not located within an Alquist-Priolo Special Studies Zones (Hart, 1992). Thus, the potential for ground surface rupture impacting the site is considered low. As shown on the Seismic Hazard Zones, Plate 4, the site is within a liquefaction Seismic Hazard Zone but not a landslide Seismic Hazard Zone (CDMG, 1999). The site lies within Seismic Zone 4 of the 1999 Los Angeles Building Code (LABC), (1997 Uniform Building Code (UBC)). Based on the current understanding of the geologic framework of the site area, the seismic hazard which is expected to have the highest probability of affecting the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in Southern California. Known regional faults that could produce significant ground shaking at the site include the Compton Thrust, Newport-Inglewood, Elysian Park Thrust, Hollywood, Santa Monica, Palos Verdes, Raymond and Verdugo Faults, among others. The closest of these are the Compton Thrust and Newport-Inglewood Faults with surface projections of potential rupture area located approximately 0.4 km and 1.5 km, respectively, from the site. The location of the site in relation to known strike slip faults is shown on Plate 5, Regional Fault Map.

### 7.1 Ground Motion

A probabilistic seismic hazard analysis was performed using the computer program FRISKSP (Blake, 1998b) in order to estimate the Peak Ground Acceleration (PGA) that could occur at the site, based on recurrence interval. The probabilistic analysis considered various magnitudes of earthquakes, along their respective fault lengths, that could occur along active or potentially active faults within a 100-km radius of the site. Standard deviation was applied during the analysis to assess the uncertainty inherent in the calculation with respect to magnitude, distance, and ground motion. An average of three attenuation relationships (Boore et al.-NEHRP Class D site, 1997; Campbell and Bozorgnia –Alluvium, 1994/1997; and Sadigh-Deep Soil, 1997) were used to estimate ground motions at the site for multiple distance/magnitude calculation combinations inherent in the probabilistic analysis.

The results of the probabilistic seismic hazard analysis suggest a maximum probable earthquake, MPE, (10 percent probability of exceedance in 50 years - 475 year return period) ground acceleration of 0.49g for the site. The upper bound earthquake, UBE, (10 percent probability of exceedance in 100 years - 950 year return period) ground acceleration was determined to be approximately 0.61g. The results of the probabilistic analysis in terms of probability of exceedance, as well as average return period (ARP), are included in Appendix B of this report.

### 7.2 Liquefaction

The site is shown on the State of California Seismic Hazard Zones map as being within an area that has potential for liquefaction. Liquefaction typically occurs when near surface (usually upper 50 feet), saturated, clean, fine-grained loose sands are subject to intense ground shaking.

Groundwater was encountered at a depth of approximately 46 feet in the deep boring (B-1). Groundwater data obtained from California Division of Mines and Geology (CDMG, 1998) indicates that the shallowest reported depth to groundwater in the site area was on the order of 10 feet below the ground surface.

SPT sampler blow counts in the sandy soils below a depth of approximately 32 feet in boring B-1 were relatively high (45 to 72) indicating dense to very dense materials with the exception of the SPT at a depth of 40 feet. The soil sample retrieved from the SPT at 40 feet contained interlayered sand and clay. It is our opinion that the interlayered clayey soils resulted in a lower blow count (15) over the sample interval and that this blow count is not a representative indication of the denseness of the granular soils. Considering the SPT blow counts above and below this depth, this soil layer was considered non-liquefiable.

Site-specific simplified liquefaction evaluations were performed in accordance with Section 1804.5 of the 1998 CBC for the site-specific estimated maximum probable ground acceleration of 0.49g. The analyses were performed using blowcount data obtained during our field exploration and a shallow historic groundwater depth of 10 feet below the ground surface. Particle soil size information obtained through laboratory testing, procedures outlined by Tokimatsu and Seed (1984) and information presented in the Salt Lake City NCEER Workshop Proceedings (1997) were utilized for the analyses. The results of the analyses indicate that the site soils are not susceptible to notable liquefaction even if groundwater was to rise to historic levels (10 feet below the ground surface).

### 7.3 Dynamic Settlement

Seismic dynamic settlement is a typical term applied to settlement of loose to medium dense granular soils above groundwater. Based on the soil characteristics, the site is potentially susceptible to seismically induced dynamic settlement as the result of a significant earthquake on one of the nearby faults. Calculations were performed to evaluate the possible magnitude of seismically induced dynamic settlement using the procedure presented by Tokimatsu and Seed (1984) and simplified by Pradel (1998). SPT blow counts were corrected for drive energy and fines content in accordance with the recommendations presented in the Salt Lake City NCEER Workshop Proceedings (1997). Results of the analyses indicate that potential total and differential dynamic settlements would be negligible.

### 7.4 Other Seismic Hazards

In addition to surface fault rupture, ground shaking and seismic-induced settlement, other effects of seismic activity include landsliding, lateral spreading, earthquake-induced flooding, seiches, and tsunamis. Results of a site-specific evaluation of the potential for these effects affecting the project site are presented below:

- Landslides: Seismically-induced landslides and other slope failures are common occurrences during or soon after earthquakes. The project site is relatively flat. In the absence of significant ground slopes, the potential for seismically-induced landslides to affect the proposed site is considered to be negligible.
- Lateral Spreading: Seismically-induced lateral spreading involves primarily lateral movement of earth materials due to ground shaking. It differs from slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. Based on the materials encountered in our borings and with consideration to the depth of historic and current groundwater levels, the potential for lateral spreading of the project area is very low.
- Earthquake-Induced Flooding: This is flooding caused by failure of dams or other water-retaining structures as a result of earthquakes. The potential of earthquake-induced flooding is considered to be low.

- Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. The site not located adjacent to any enclosed large bodies of water that could experience seiches during an earthquake. Thus, the potential for seiches impacting the site is considered very low.
- Tsunamis: Tsunamis are tidal waves generated in large bodies of water by fault displacement or major ground movement. Based on the location of the site, tsunamis do not pose a significant hazard to the project site.

### 7.5 Seismic Design Parameters

Section 1636 of the 1999 LABC defines six different soil profile types ($S_A$ – $S_F$). These soil profile types are used to determine seismic coefficients $C_a$ and $C_v$ from Tables 16-Q and 16-R respectively. These coefficients, together with the near source factors are used to calculate the pseudo-static base shear force that structures must be designed to withstand.

Soil Profile Types are based on the average properties of the upper 100 feet of soil at the site. The appropriate soil profile type is chosen by referencing the average shear wave velocity, average Standard Penetration Test (SPT) blow counts, and/or the undrained shear strength of the soil. Based on the SPT blow counts of the deepest borings, Soil Profile Type $S_D$ was selected for this site.

Near source factors are also used to determine seismic coefficients $C_a$ and $C_v$. Near source factors $N_a$ and $N_v$ can be determined from 1999 LABC Tables 16-S and 16-T respectively. In order to use the tables it is necessary to know the distance and fault type of the nearest controlling fault as found in the State of California Department of Conservation “Active Fault Near-Source Zones” maps. According to this map, the nearest controlling fault is the Newport-Inglewood Fault (a type “B” fault) at a distance of approximately 1.5 km.

Using the above information and the appropriate tables within the 1999 LABC, the near source factors and seismic coefficient can be determined. These values are summarized in Table 2, Seismic Design Data.

Table 2 – Seismic Design Data

| | |
|---|---|
| Seismic Zone Factor (Z) | 0.4 |
| Soil Profile Type | $S_D$ |
| Near Source Factor, $N_a$ | 1.3 |
| Near Source Factor $N_v$ | 1.6 |
| Seismic Coefficient $C_a$ | 0.57 |
| Seismic Coefficient $C_v$ | 1.02 |

## 8.0 SITE RECOMMENDATIONS

### 8.1 General

Our primary geotechnical considerations with respect to the proposed construction is the variable thickness, consistency and engineering characteristics of the existing uncertified fill materials, the relatively high moisture content of the shallow natural soils, and the expansion potential and compressibility characteristics of the shallow natural soils. We recommend that the shallow soils within areas of proposed structures be over-excavated and replaced with properly compacted fill soils. Over-excavation is recommended to remove any undocumented existing fill soils and loose natural soils immediately beneath proposed shallow foundations. Spread footings bearing on compacted fill materials may then be used for support of the structures.

Detailed geotechnical engineering recommendations addressing the surficial soils, site preparation, site earthwork, foundations, and slabs-on-grade are presented in the remaining portions of this report. The following opinions, conclusions, and recommendations are based on the properties of the materials encountered in the exploratory borings, and laboratory test results.

A representative of GED will need to provide observation and testing services during site earthwork and construction of foundations. This will allow us the opportunity to compare actual conditions with those encountered in the exploratory borings, if necessary, to expedite supplemental recommendations if warranted by the exposed subsurface conditions. We shall also review your preliminary foundation and earthwork plans and specifications. This review will provide us an opportunity to detect misinterpretation or misunderstandings of our recommendations prior to the start of construction.

### 8.2 Site Preparation and Earthwork

#### 8.2.1 Site Clearing

Prior to construction, all organic or inorganic materials and debris shall be removed from the construction area and disposed of outside the site. Any existing structural or landscape elements within these areas, including any foundation elements, shall be demolished and removed from the site. Any utilities, whether active or inactive, shall be identified and removed from the site or relocated per project plans and specifications. Any cavities resulting from removal of any existing foundations or utility lines should be properly backfilled and compacted in accordance with the following sections.

### 8.2.2 Over-Excavation

Any existing fill materials and shallow natural soils within foundation areas shall be over-excavated to a depth of four feet below existing site grade or to a depth of three feet below proposed footings, whichever is deeper, and replaced with compacted fill. Slab-on-grade areas within the building shall be over excavated to suitable undisturbed natural soil or a minimum of 18 inches below the design subgrade elevation; whichever is deeper, and replaced with compacted fill soil. Over-excavation depths may have to be greater in some areas to remove unsuitable soils. Removal excavations should extend a horizontal distance beyond the edges of the foundations equal to the depth of over excavation below the footings or a minimum of 5 feet, whichever is greater.

### 8.2.3 Temporary Excavations

Based on our observations during subsurface investigation and results of laboratory tests, the materials at the site should be readily excavated by conventional earthmoving equipment in good operating condition. All temporary excavations shall conform to the State of California Construction Safety Orders (CAL/OSHA). Unsurcharged, temporary vertical excavations can be a maximum depth of 5 feet. Unsurcharged excavations greater than 5 feet and to a maximum of 15 feet shall be sloped at a 1:1 (H:V) or flatter inclination from the existing ground surface to the bottom of the excavation or should be shored. Excavations greater than 15 feet are not anticipated for the project. Any excavation that enters the influence zone of an adjacent structure or right-of-way shall utilize slot cuts or shoring.

### 8.2.4 Slot Cuts

Slot cuts for unsurcharged excavations may be used per the following recommendations:

- The excavation side shall be initially slopped uniformly at an inclination of 1:1 (horizontal to vertical) with the top of the slope placed two feet from the edge of the footing of the existing structure, and the bottom of the slope at the bottom of the over-excavation
- Slots shall be constructed in an A, B, C sequence with neither adjacent slot excavated until the slot is completely backfilled to the grade of the initial 1:1 slope (horizontal to vertical).
- Maximum width of slots shall not exceed 8-feet and maximum height of slots shall not exceed 8-feet.
- Prior to placing of any fill in any slots, the bottom of the slot, shall be scarified to a minimum depth of 6-inches, moisture-conditioned and mechanically compacted in accordance with the requirements of Section 8.2.6 of this report.
- Fill shall be placed in lifts not exceeding 8-inches in thickness shall be moisture conditioned and compacted in accordance with the requirements of Section 8.2.8 of this report. Also,

each lift shall be moisture-conditioned between optimum-moisture content and a few percent above the optimum and mechanically compacted.

- Following completion of the slot cutting, all fill placed adjacent to the initial 1:1 slope (horizontal to vertical) shall be benched into the slope in accordance with the 1999 LABC requirements.
- All excavation of the slots shall be performed under the continuous observation of the geotechnical engineer of record or a GED representative working under direct supervision of the geotechnical engineer of record.
- The GED representative working under the supervision of the geotechnical engineer of record shall be able to take the place of a Building and Safety Deputy-Grading inspector.
- Backfill placed in the slots shall be tested for compaction as it is placed.

If excessive sloughing and caving occurs, slot cuts shall be backfilled immediately and shoring shall be installed. If necessary, recommendations can be provided for slot cutting of surcharged excavations on a case by case basis.

### 8.2.5 Subgrade Preparation

All exposed subgrade soil surfaces, including over-excavation bottoms, should be observed by a representative of the GED and the City of Los Angeles Grading Inspector prior to placement of fill. If soft, yielding, or unsuitable soils are exposed at the subgrade surface, then the unsuitable soils should be removed and replaced with properly compacted fill soils. Due to the nature of the site soils and the past long term watering of landscaped areas, unsuitable soft wet soils may be encountered to considerable depths below the ground surface in some areas. If a suitable bottom is not obtainable within a reasonable depth, supplemental recommendations will be provided during construction to stabilize areas of soft soils.

Subgrade surfaces suitable for fill placement should be scarified to a depth of 6 inches, moisture-conditioned between optimum-moisture content and a few percent above the optimum, and compacted. Existing site soils are typically several percentage points wet of the optimum moisture content. Thus, drying of these wet site soils or mixing of these soils with dryer soils may be required prior to compaction. Clayey subgrade soils (soils with 15% or more by weight finer than 0.005 mm) in over-excavation areas of structures shall be compacted to a minimum 90 percent of the ASTM Test Method D1557-91 laboratory maximum density. Granular subgrade soils (soils with less than 15 % by weight finer than 0.005 mm) in over-excavation areas of structures shall be compacted to a minimum 95 percent of the ASTM Test Method D1557-91 laboratory maximum density. Subgrade soils in non-structural areas shall be scarified to a minimum depth of six inches and compacted to a minimum 90 percent of the ASTM Test Method D1557-91 laboratory maximum density. After the completion and acceptance of the subgrade preparation, fill material may be placed in accordance with the following recommendations. Subgrade soils shall not be allowed to dry out and shall be keep moist (between optimum moisture content and a few percent above the optimum moisture content) until covered with subsequent construction.

### 8.2.6 Fill Materials

Fill soils shall consist of on-site soils or approved import material. The on-site soils encountered during the geotechnical investigation are acceptable for use as fill material for this project. Before being used as fill, on-site soils shall be cleaned of all organic or inorganic debris and all materials larger than 3 inches. Existing site soils are typically several percentage points wet of the optimum moisture content. Thus, drying of these wet site soils or mixing of these soils with dryer soils may be required prior to being used as compacted fill. If import material is required for use as fill for this project, it shall be predominantly granular, non-expansive (EI less than 20), and shall be free of organic or inorganic debris, contamination and materials larger than 3 inches. Import material shall be tested and approved by GED prior to importing to the job site. GED shall be notified a minimum of three days prior to the scheduled importing of the soil to the project site.

### 8.2.7 Fill and Backfill Placement

Structural fill shall be used for the support of all foundation elements and may only be placed on approved surfaces/subgrades prepared in accordance with Section 8.2.6 of this report. Fill shall be placed in loose lifts not exceeding 8 inches in thickness, moisture-conditioned between optimum-moisture content and a few percent above the optimum moisture content and mechanically compacted. Clayey fill soils (soils with 15% or more finer than 0.005 mm) placed in structure over-excavation areas shall be compacted to a minimum of 90 percent relative compaction, as determined by ASTM Test Method D1557-91. Granular fill soils (soils with less than 15 % finer than 0.005 mm) placed in structure over-excavation areas shall be compacted to a minimum of 95 percent relative compaction, as determined by ASTM Test Method D1557-91. Fill soils placed in other areas such as sidewalk and landscaped areas shall be compacted to a minimum of 90 percent relative compaction, as determined by ASTM Test Method D1557-91. Any aggregate base should be moisture-conditioned between optimum and two percent above optimum-moisture and compacted to a minimum of 95 percent relative compaction. Fill compaction shall be tested and recorded by a certified compaction testing agency working under the direct supervision of GED. Densification by flooding or jetting is not allowed. Compacted fill soils shall not be allowed to dry out and shall be keep moist (between optimum moisture content and a few percent above the optimum moisture content) until covered with subsequent construction.

### 8.2.8 Trench Backfill

Trench excavations for utility pipes may be backfilled with onsite soils under the observation of a representative of GED. After utility pipes have been laid and properly bedded, the space around the pipe should be backfilled with clean sand (having sand equivalent of 30 or greater) or gravel to a depth of approximately 1 foot over the top of the pipe, before the controlled backfill is placed. Controlled backfill shall be moisture conditioned, placed and compacted in accordance with the recommendations presented in Section 8.2.7 of this report.

### 8.2.9 Fill Certification

The Contractor shall not excavate for footings within compacted fill until an approval letter is issued by the Department of Building and Safety, Grading Division. The Contractor shall allow at least ten (10) working days from the time that the fill operations are completed to the issuance of the Fill Certification Approval Letter from the Department of Building and Safety.

### 8.3 Structure Foundations

Based on our understanding of the proposed construction and the characteristics of the on-site soils, proposed structures may be supported on conventional continuous and/or spread footings. Footings shall be founded on at least three feet of properly compacted fill material and shall be founded at least 18 inches below the lowest adjacent grade. A minimum width of 18 inches for continuous footings and 24 inches for column footings is recommended. Footings with these minimum sizes, bearing on at least 36 inches of properly compacted fill, may be designed for a net allowable vertical bearing pressure of 2,500 pounds per square foot (psf) for dead-plus-live loads. This allowable bearing pressure shall not be increased for any additional depth or width of the footing. A 1/3 increase may be used for short term loading conditions such as wind or seismic forces.

We recommended that all continuous footings should be reinforced with a minimum of two No.5 steel reinforcing bar at the top and bottom to provide structural continuity and to permit spanning of local irregularities. The structural engineer should design the actual footing reinforcement.

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 300 psf per foot of depth, to a maximum of 3,000 psf, may be used for sides of footings poured against properly compacted fill. This allowable passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. The allowable passive pressure may be increased by 33 percent of lateral loading due to wind or seismic forces.

Total static settlement of the proposed foundations, designed and constructed in accordance with the recommendations presented herein, should not exceed ¾-inch. Differential settlements should not exceed one-half of the total settlement between adjacent foundations. The majority of these settlements are expected during construction and initial occupation of the building as the loads are applied.

GED will inspect all footing excavations prior to the placement of reinforcing steel and concrete. The contractor shall be responsible for and protecting the footing excavations from inundation and erosion during wet weather and maintaining the excavations free of loose and/or disturbed material and the proper moisture content of the footing excavations until the placement of the concrete.

### 8.4 Planter and Fence Wall Foundations

Spread footing foundations are suitable for the support of accessory walls and retaining walls less than 8 feet in height that are structurally isolated from the main structures. In addition, we recommend that separation joints be provided in the wall at increments of 25 feet or less. Footings with a minimum width of 12 inches and embedded a minimum of 18 inches below the lowest adjacent grade, bearing on undisturbed natural soils or properly compacted fill soils, may be designed for an allowable bearing pressure of 1,500 pounds per square foot (psf).

We recommended that all continuous footings should be reinforced with a minimum of two No.5 steel reinforcing bar at the top and bottom to provide structural continuity and to permit spanning of local irregularities. The structural engineer should design the actual footing reinforcement.

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.30 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 200 psf per foot of depth, to a maximum of 2,000 psf, may be used for sides of footings poured against undisturbed natural soils or properly compacted fill. This allowable passive pressure is applicable for level (ground slope equal to or flatter than 5:1, horizontal:vertical) conditions only.

Bearing values indicated above are for total dead-load and frequently applied live-loads. The above vertical bearing may be increased by 33 percent for short durations of loading which will include the effect of wind or seismic forces. The allowable passive pressure may be increased by 33 percent of lateral loading due to wind or seismic forces.

### 8.5 Slab-on-Grade

All slab-on-grade areas shall be prepared in accordance with Section 8.2 of this report. Slab subgrade soils shall not be allowed to dry out between the time the subgrade is prepared and until covered with subsequent construction. As a minimum, we suggest that the conventional slabs-on-grade that will not be subjected to vehicular traffic be a minimum of 4 inches thick and reinforced with No. 4 deformed steel reinforcing bars at 16 inches on center each way placed at mid-depth through the slab.

The minimum recommended steel will not prevent the development of slab cracks but will aid in keeping joints relatively tight and will reduce the potential for differential movement between adjacent panels. Care should be taken to avoid slab curling if slabs are poured in hot weather. Slabs should be designed and constructed as promulgated by the Portland Cement Association (PCA). Prior to the slab pour, all utility trenches should be properly backfilled and compacted.

In areas where a moisture-sensitive floor covering (such as vinyl, tile, or carpet) is used, slabs can be protected by a minimum 10-mil-thick polyethylene vapor barrier between the slab and compacted subgrade. Where the barrier is used, it should be placed between two 1-inch layers of sand to protect it from punctures and to aid in the concrete cure. Vapor barrier seams should be overlapped a minimum of 6 inches and taped or otherwise sealed.

### 8.6 Cement Type and Corrosion Measures

Based on the results of soluble sulfate content testing on near-surface soils at the subject site, concrete should be designed in accordance with the “negligible” category defined within Table 19-A-4 of the 1999 LABC (1997 UBC).

Based on the resistivity measurement, the near-surface soils are considered to be potentially corrosive to buried metals. As a minimum, the following corrosion mitigation measures are considered appropriate for the site soils:

- All steel and wire concrete reinforcement should have at least 3 inches of concrete cover where cast against soil, unformed.
- As a minimum, below grade ferrous metals should be given a high-quality protective coating, such as 18-mil plastic tape, extruded polyethylene, coal-tar enamel, or Portland Cement mortar.
- Below grade metals should be electrically insulated (isolated) from above grade metals by means of dielectric fittings in ferrous utilities and/or exposed metal structures breaking grade.

## 9.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

### 9.1 Review of Plans and Specifications

The final grading and foundation plans and specifications should implement the recommendations presented in this report and should be reviewed by GED.

### 9.2 Geotechnical Observation and Testing During Construction

All grading, excavation, and construction of foundations should be performed under the observation and testing of the Geotechnical Engineer at the following stages:

- Upon completion of site clearing;
- During subgrade preparation;
- During fill placement;
- After building footing excavations and immediately prior to placement of foundation concrete;
- During excavation and backfilling of all utility trenches; and
- When any unusual or unexpected geotechnical conditions are encountered.

### 9.3 Closure

If there are any questions regarding this report, please contact Juan Yanez at (213) 847-4030 or Patrick J. Schmidt at (213) 847-4046.

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| Civil Engineering Associate III | Geotechnical Engineer I |

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# APPENDIX A

# URS DATA REPORT

# APPENDIX B

# PROBABILISTIC SEISMIC HAZARD ANALYSES