# APPENDIX F

# PRELIMINARY WATER QUALITY MANAGEMENT PLANS

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# Water Quality Management Plan (WQMP)

## Project Name:

Our Lady of Peace - Church

**Prepared for:**

**Roman Catholic Bishop of Orange**

**13280 Chapman Ave**

**Garden Grove, CA 92840**

**Prepared by:**

**Proactive Engineering Consultants**

**Engineer Amy Jaramillo Registration No. 94608**

**27051 Towne Centre Dr, Suite 270**

**Foothill Ranch, CA 92610**

**949-259-5708**

**Prepared on:**

**November 2025**

# Project Owner's Certification

| | | | |
|---|---|---|---|
| Permit/Application No. | 00955110-PCPU | Grading Permit No. | N/A |
| Tract/Parcel Map No. | N/A | Building Permit No. | N/A |
| CUP, SUP, and/or APN (Specify Lot Numbers if Portions of Tract) | | | N/A |

This Water Quality Management Plan (WQMP) has been prepared for Roman Catholic Bishop of Orange by Proactive Engineering. The WQMP is intended to comply with the requirements of the local NPDES Stormwater Program requiring the preparation of the plan.

The undersigned, while it owns the subject property, is responsible for the implementation of the provisions of this plan and will ensure that this plan is amended as appropriate to reflect up-to-date conditions on the site consistent with the current Orange County Drainage Area Management Plan (DAMP) and the intent of the non-point source NPDES Permit for Waste Discharge Requirements for the County of Orange, Orange County Flood Control District and the incorporated Cities of Orange County within the Santa Ana Region. Once the undersigned transfers its interest in the property, its successors-in-interest shall bear the aforementioned responsibility to implement and amend the WQMP. An appropriate number of approved and signed copies of this document shall be available on the subject site in perpetuity.

| **Owner:** | | | |
|---|---|---|---|
| Title | Deacon Javier H. Rodriguez | | |
| Company | Roman Catholic Bishop of Orange | | |
| Address | 13280 Chapman Ave Garden Grove, CA 92840 | | |
| Email | jrodriguez@rcbo.org | | |
| Telephone # | 714-282-3032 | | |
| Signature | | Date | |

# Contents

**Page No.**



## Attachments

# Section I Discretionary Permit(s) and Water Quality Conditions

Provide discretionary permit and water quality information. *Refer to Section 2.1 in the Technical Guidance Document (TGD) available from the Orange County Stormwater Program (ocwatersheds.com).*

| **Project Infomation** | | | |
|---|---|---|---|
| Permit/Application No. | 00955110-PCPU | Tract/Parcel Map No. | N/A |
| Additional Information/ Comments: | N/A | | |
| **Water Quality Conditions** | | | |
| Water Quality Conditions (list verbatim) | N/A | | |
| **Watershed-Based Plan Conditions** | | | |
| Provide applicable conditions from watershed - based plans including WIHMPs and TMDLS. | N/A | | |

# Section II Project Description

## II.1 Project Description

| Description of Proposed Project | | | | |
|---|---|---|---|---|
| Development Category (Verbatim from WQMP): | New development projects that create 10,000 square feet or more of impervious surface. This category includes commercial, industrial, residential housing subdivisions, mixed-use, and public projects on private or public property that falls under the planning and building authority or the Permittees. | | | |
| Project Area (ft$^2$): 133,294 | Number of Dwelling Units: N/A | | SIC Code: 8661 | |
| Narrative Project Description: | The proposed project replaces an existing temporary church structure with 3 new buildings (church, rectory, parish hall). Project also includes site landscaping, a plaza and two level parking garage. | | | |
| Project Area | Pervious | | Impervious | |
| | Area (acres or sq ft) | Percentage | Area (acres or sq ft) | Percentage |
| Pre-Project Conditions | 66,211 sf | 64% | 37,026 sf | 36% |
| Post-Project Conditions | 30,537 sf | 23% | 102,757 sf | 77% |
| Drainage Patterns/Connections | Once developed, there will be a series of on-site inlets through the parking lot that will connect to the on-site private storm drain system. The plaza will have small area drain inlets that will also connect to the on-site system. The storm drain will flow from east to the west confluencing near the entry off Remington. The storm drain pipe will then continue across the adjacent church parcel just outside of the existing public utility easement along Remington and connect to the existing catch basin at the corner of Roosevelt and Remington.<br>From there it flows in a mainline RCP storm drain in Roosevelt that ranges from 24” to 48”. This pipeline eventually outlets into Central Irvine Channel, then Peters Canyon Channel, into San Diego Creek and then into Newport Bay. | | | |

## II.2 Potential Stormwater Pollutants

| Pollutants of Concern | | | |
|---|---|---|---|
| Pollutant | Circle One: E=Expected to be of concern N=Not Expected to be of concern | | Additional Information and Comments |
| Suspended-Solid/ Sediment | E | N | |
| Nutrients | E | N | |
| Heavy Metals | E | N | |
| Pathogens (Bacteria/Virus) | E | N | |
| Pesticides | E | N | |
| Oil and Grease | E | N | |
| Toxic Organic Compounds | E | N | |
| Trash and Debris | E | N | |

## II.3 Hydrologic Conditions of Concern

☐ No – Show map

☒ Yes – Describe applicable hydrologic conditions of concern below.

Streams downstream of the project are potentially susceptible to hydromodification, this includes parts of Peters Canyon Channel, San Diego Creek Channel and Newport Bay.

The 2 year 24 hour post development runoff volume is more than 5% greater than the existing condition 2 year 24 hour runoff volume.

## II.4 Post Development Drainage Characteristics

Once developed, there will be a series of on-site inlets through the parking lot that will connect to the on-site private storm drain system. The plaza will have small area drain inlets that will also connect to the on-site system. The storm drain will flow from east to the west confluencing near the entry off Remington. The storm drain will then flow across the adjacent church parcel(being developed for residential use) adjacent to Remington and connect to the existing catch basin at the corner of Roosevelt and Remington. The downstream flow path will remain unchanged.

## II.5 Property Ownership/Management

The property will be owned by the Roman Catholic Bishop of Orange and managed by Our Lady of Peace Korean Catholic Center.

# Section III Site Description

## III.1 Physical Setting

| | |
|---|---|
| Planning Area/ Community Name | Our Lady of Peace |
| Location/Address | 14010 Remington |
| | Irvine, CA 92620 |
| Land Use | Place of Worship |
| Zoning | Medium-High Density Residential |
| Acreage | 3.06 |
| Predominant Soil Type | C |

## *III.2 Site Characteristics*

| | |
|---|---|
| *Precipitation Zone* | The Design Capture Storm Depth is 0.80" based on OCTGD Rainfall Zone Figure XVI-1. |
| *Topography* | The site is relatively flat with a general grade of 1-2% from northwest to southeast. |
| *Drainage Patterns/Connections* | The site generally drains from the west to the east. Today it sheet flows over much of a barren field. There are a series of on-site drainage inlets that connect to a small on-site storm drain system and eventually connect to the existing catch basin on Remington (at the corner of Remington and Roosevelt).<br><br>From there it flows in a mainline RCP storm drain in Roosevelt that ranges from 24" to 48". This pipeline eventually outlets into Central Irvine Channel, then Peters Canyon Channel, into San Diego Creek and then into Newport Bay.<br><br>Part of the project is a Parcel Map to subdivide the existing parcel into 2 separate pieces. The existing and proposed drainage patterns can be |

| | |
|---|---|
| | found in Attachment E hydrology maps. The majority of what will be the church property flows to the same location at the central entry driveway in both the existing and proposed condition. There is a portion in the south in the existing condition is considered offsite drainage onto the proposed residential project. For this reason the existing and proposed drainage areas are not exactly the same. |
| *Soil Type, Geology, and Infiltration Properties* | The soil is primarily clayey sand, silty sand and sandy clay. The site is underlain by Young Alluvial Fan Deposits (Qyf). |
| *Hydrogeologic (Groundwater) Conditions* | Groundwater was found at approximately 54' deep. Historic high groundwater is estimated at greater than 40'. |
| *Geotechnical Conditions (relevant to infiltration)* | Infiltration testing produced rates of 0.01 and 0.02 inches/hour without a factor of safety. Therefore, infiltration is not feasible. (See report in Attachment D) |
| *Off-Site Drainage* | N/A |
| *Utility and Infrastructure Information* | Sewer & water are served by IRWD. Public storm drain in the adjacent roadways is owned and maintained by the City of Irvine. |

# III.3 Watershed Description

| | |
|---|---|
| Receiving Waters | Peters Canyon Channel<br>San Diego Creek<br>Newport Bay |
| 303(d) Listed Impairments | **San Diego Creek Reach 1** (Benthic Community Effects, DDT, Indicator Bacteria, Malathion, Nutrients, Sedimentation/Siltation, Selenium, Toxaphene, Toxicity)<br>**Newport Bay Upper** (Chlordane, Copper, DDT, Indicator Bacteria, Malathion, Nutrients, PCBs, Sedimentation/Siltation, Toxicity<br>**Newport Bay Lower** (Chlordane, Copper, DDT, Indicator Bacteria, Nutrients, PBCs, Toxicity) |
| Applicable TMDLs | All waterbodies listed above are requirement status:<br>Category 5A - a TMDL is required, but not yet completed<br>Category 5B - being addressed by USEPA approved TMDL |
| Pollutants of Concern for the Project | Suspended-Solid/ Sediment<br>Nutrients<br>Pesticides<br>Oil and Grease<br>Toxic Organic Compounds<br>Trash and Debris |
| Environmentally Sensitive and Special Biological Significant Areas | N/A |

# Section IV Best Management Practices (BMPs)

## IV. 1 Project Performance Criteria

| | | |
|---|---|---|
| (NOC Permit Area only) Is there an approved WIHMP or equivalent for the project area that includes more stringent LID feasibility criteria or if there are opportunities identified for implementing LID on regional or sub-regional basis? | YES ☐ | NO ☒ |
| If yes, describe WIHMP feasibility criteria or regional/sub-regional LID opportunities. | N/A | |

| Project Performance Criteria (continued) | |
|---|---|
| If HCOC exists, list applicable hydromodification control performance criteria (Section 7.II-2.4.2.2 in MWQMP) | • Post-development runoff volume for the two-year frequency storm does not exceed that of the predevelopment condition by more than five percent<br>• Time of concentration of post-development runoff for the two-year storm event is not less than that for the predevelopment condition by more than five percent |
| List applicable LID performance criteria (Section 7.II-2.4.3 from MWQMP) | • Priority Projects must infiltrate, harvest and use, evapotranspire, or biotreat/biofilter, the 85th percentile, 24-hour storm event (Design Capture Volume).<br>• A properly designed biotreatment system may only be considered if infiltration, harvest and use, and evapotranspiration (ET) cannot be feasibly implemented for the full design capture volume. In this case, infiltration, harvest and use, and ET practices must be implemented to the greatest extent feasible and biotreatment may be provided for the remaining design capture volume. |
| List applicable treatment control BMP performance criteria (Section 7.II-3.2.2 from MWQMP) | • If it is not feasible to meet LID performance criteria through retention and/or biotreatment provided on-site or at a sub-regional/regional scale, then treatment control BMPs shall be provided on-site or offsite prior to discharge to waters of the US. Sizing of treatment control BMP(s) shall be based on either the unmet volume after claiming applicable water quality credits, if appropriate (See Section 7.II-3.1 Water Quality Credits) and as calculated in TGD Appendix VI. If treatment control BMPs can treat all of the remaining unmet volume and have a medium to high effectiveness for reducing the primary POCs, the project is considered to be in compliance; a waiver application and participation in an alternative program is not required.<br>• If the cost of providing treatment control BMPs greatly outweighs the pollution control benefits they would provide, a waiver of treatment control and LID requirements can be requested and alternative compliance approaches must be used to fulfill the remaining unmet volume. |

| | |
|---|---|
| Calculate LID design storm capture volume for Project. | LID Design Storm Capture Volume (DCV)<br><br>Defined as:<br>$V = C \times d \times A \times 43{,}560$ sf/ac × 1/12 in/ft<br><br>Where:<br>V = runoff volume during the design storm event, cu-ft<br>C = runoff coefficient = (0.75 × imp + 0.15)<br>imp = impervious fraction of drainage area (ranges from 0 to 1)<br>d = storm depth (inches)<br>A = tributary area (acres)<br><br>The Design Capture Volume for the project is as follows:<br>85th percentile, 24-hr storm depth = d = 0.80 inches<br>Drainage Area = A = 3.06 acres<br>Imperviousness = imp = 0.77<br><br>V = 3.06 ac × 0.80 inches × ((0.75 × 0.77) + 0.15) × 43,560 sf/ac × 1/12 in/ft<br>V = 6,465 cu-ft<br>*Note: Volume calculated is for reference only, LID BMP solution will be a flow based proprietary biotreatment solution. See Attachment B for calculations.* |

# IV.2. SITE DESIGN AND DRAINAGE PLAN

Where feasible, impervious area dispersion was utilized to allow for evapotranspiration and incidental infiltration of runoff, prior to discharging to the storm drain system.

Infiltration was considered as a BMP for the development but due to low measured rates the project is infeasible for infiltration per the Geotechnical Report in Attachment D

Rainwater harvesting and re-use was considered as a BMP. The total landscape area does not require enough demand to justify the use of harvesting BMPs. See Worksheet J in Attachment B.

Therefore, bio-treatment was selected as the proposed BMP to meet LID performance criteria. Runoff from the project will be collected and treated by modular wetland systems. Bio treatment unit details are included in Attachment C.

The entire site was treated as one drainage management area. One unit at the downstream end will treat the required flows and the peak flows will bypass the water quality treatment system. The WQMP site showing the BMP location and overall drainage pattern can be found in Attachment A.

DMA 1:

A=3.1 acres
Imp=0.77
C=0.73
Qdesign=0.52cfs

# IV.3 LID BMP SELECTION AND PROJECT CONFORMANCE ANALYSIS

## IV.3.1 Hydrologic Source Controls

| Name | Included? |
|---|---|
| Localized on-lot infiltration | ☐ |
| Impervious area dispersion (e.g. roof top disconnection) | ☐ |
| Street trees (canopy interception) | ☒ |
| Residential rain barrels (not actively managed) | ☐ |
| Green roofs/Brown roofs | ☐ |
| Blue roofs | ☐ |
| Impervious area reduction (e.g. permeable pavers, site design) | ☒ |
| Other: | ☐ |

**HSC-3 Street Trees**

Trees will be planted within the parking lot and central plaza area. These trees will intercept and provide some volume reduction benefits for the project.

**Impervious Area Reduction**

The central church plaza area will incorporate as much landscape and pervious area as possible to provide aesthetic and water quality benefits.

**NOTE:** DCV reduction credit has not been determined for these areas as their benefits are incidental and the areas not specifically designed to retain runoff.

## IV.3.2 Infiltration BMPs

| Name | Included? |
|---|---|
| Bioretention without underdrains | ☐ |
| Rain gardens | ☐ |
| Porous landscaping | ☐ |
| Infiltration planters | ☐ |
| Retention swales | ☐ |
| Infiltration trenches | ☐ |
| Infiltration basins | ☐ |
| Drywells | ☐ |
| Subsurface infiltration galleries | ☐ |
| French drains | ☐ |
| Permeable asphalt | ☐ |
| Permeable concrete | ☐ |
| Permeable concrete pavers | ☐ |
| Other: | ☐ |
| Other: | ☐ |

Per the Geotech report in Attachment D infiltration testing produced rates of 0.01 and 0.02 inches/hour without a factor of safety. Therefore, infiltration is not feasible.

## IV.3.3 Evapotranspiration, Rainwater Harvesting BMPs

| Name | Included? |
|---|---|
| All HSCs; *See Section IV.3.1* | ☐ |
| Surface-based infiltration BMPs | ☐ |
| Biotreatment BMPs | ☐ |
| Above-ground cisterns and basins | ☐ |
| Underground detention | ☐ |
| Other: | ☐ |
| Other: | ☐ |
| Other: | ☐ |

Due to the small landscape area onsite harvest and re-use is not practical or economical. Refer to Worksheet J in Attachment B.

## IV.3.4 Biotreatment BMPs

| Name | Included? |
|---|---|
| Bioretention with underdrains | ☐ |
| Stormwater planter boxes with underdrains | ☐ |
| Rain gardens with underdrains | ☐ |
| Constructed wetlands | ☐ |
| Vegetated swales | ☐ |
| Vegetated filter strips | ☐ |
| Proprietary vegetated biotreatment systems | ☒ |
| Wet extended detention basin | ☐ |
| Dry extended detention basins | ☐ |
| Other: | ☐ |
| Other: | ☐ |

The LID DCV criteria will be met by using a flow-based biotreatment BMP solution. One unit will treat the whole site's required flows. Peak flows will bypass the water quality system. See Attachment B for calculations and Attachment C for system details.

## IV.3.5 Hydromodification Control BMPs

| BMP Name | BMP Description |
|---|---|
| Oversized storm drain pipes | In order to mitigate the peak flow differential from the proposed condition down to the existing condition storm drain pipelines will be oversized. The additional pipe volume will serve as a detention system and an orifice/weir structure will be installed to outlet pipes at a rate similar to the existing condition. |
| | |
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| | |
| | |

## IV.3.8 Non-structural Source Control BMPs

Fill out non-structural source control check box forms or provide a brief narrative explaining if non-structural source controls were not used.

| Non-Structural Source Control BMPs | | | | |
|---|---|---|---|---|
| **Identifier** | **Name** | **Check One** | | **If not applicable, state brief reason** |
| | | **Included** | **Not Applicable** | |
| N1 | Education for Property Owners, Tenants and Occupants | ☒ | ☐ | |
| N2 | Activity Restrictions | ☒ | ☐ | |
| N3 | Common Area Landscape Management | ☒ | ☐ | |
| N4 | BMP Maintenance | ☒ | ☐ | |
| N5 | Title 22 CCR Compliance (How development will comply) | ☒ | ☐ | |
| N6 | Local Industrial Permit Compliance | ☐ | ☒ | Not an industrial project |
| N7 | Spill Contingency Plan | ☐ | ☒ | No proposed hazardous materials onsite |
| N8 | Underground Storage Tank Compliance | ☐ | ☒ | No underground storage |
| N9 | Hazardous Materials Disclosure Compliance | ☐ | ☒ | No hazardous material onsite |
| N10 | Uniform Fire Code Implementation | ☐ | ☒ | Proposed facility does not propose to store toxic or highly toxic compressed gas. |
| N11 | Common Area Litter Control | ☒ | ☐ | |
| N12 | Employee Training | ☒ | ☐ | |
| N13 | Housekeeping of Loading Docks | ☐ | ☒ | No loading docks |
| N14 | Common Area Catch Basin Inspection | ☒ | ☐ | |
| N15 | Street Sweeping Private Streets and Parking Lots | ☒ | ☐ | |
| N16 | Retail Gasoline Outlets | ☐ | ☒ | No retail gas |

### IV.3.9 Structural Source Control BMPs

Fill out structural source control check box forms or provide a brief narrative explaining if Structural source controls were not used.

| Structural Source Control BMPs | | | | |
|---|---|---|---|---|
| **Identifier** | **Name** | **Check One** | | **If not applicable, state brief reason** |
| | | **Included** | **Not Applicable** | |
| S1 | Provide storm drain system stenciling and signage | ☒ | ☐ | |
| S2 | Design and construct outdoor material storage areas to reduce pollution introduction | ☐ | ☒ | No storage areas proposed |
| S3 | Design and construct trash and waste storage areas to reduce pollution introduction | ☒ | ☐ | |
| S4 | Use efficient irrigation systems & landscape design, water conservation, smart controllers, and source control | ☒ | ☐ | |
| S5 | Protect slopes and channels and provide energy dissipation | ☐ | ☒ | No slopes/channels |
| | Incorporate requirements applicable to individual priority project categories (from SDRWQCB NPDES Permit) | ☐ | ☒ | Santa Ana Region |
| S6 | Dock areas | ☐ | ☒ | No docks |
| S7 | Maintenance bays | ☐ | ☒ | No bays |
| S8 | Vehicle wash areas | ☐ | ☒ | No wash areas |
| S9 | Outdoor processing areas | ☐ | ☒ | No outdoor processing |
| S10 | Equipment wash areas | ☐ | ☒ | No wash areas |
| S11 | Fueling areas | ☐ | ☒ | No fueling areas |
| S12 | Hillside landscaping | ☐ | ☒ | No hillsides |
| S13 | Wash water control for food preparation areas | ☐ | ☒ | No food prep |
| S14 | Community car wash racks | ☐ | ☒ | No car washes |

# IV.4 ALTERNATIVE COMPLIANCE PLAN (IF APPLICABLE)

## IV.4.1 Water Quality Credits

Determine if water quality credits are applicable for the project. *Refer to Section 3.1 of the Model WQMP for description of credits and Appendix VI of the TGD for calculation methods for applying water quality credits.*

| **Description of Proposed Project** | | |
|---|---|---|
| Project Types that Qualify for Water Quality Credits (Select all that apply): | | |
| ☐Redevelopment projects that reduce the overall impervious footprint of the project site. | ☐Brownfield redevelopment, meaning redevelopment, expansion, or reuse of real property which may be complicated by the presence or potential presence of hazardous substances, pollutants or contaminants, and which have the potential to contribute to adverse ground or surface WQ if not redeveloped. | ☐ Higher density development projects which include two distinct categories (credits can only be taken for one category): those with more than seven units per acre of development (lower credit allowance); vertical density developments, for example, those with a Floor to Area Ratio (FAR) of 2 or those having more than 18 units per acre (greater credit allowance). |
| ☐ Mixed use development, such as a combination of residential, commercial, industrial, office, institutional, or other land uses which incorporate design principles that can demonstrate environmental benefits that would not be realized through single use projects (e.g. reduced vehicle trip traffic with the potential to reduce sources of water or air pollution). | ☐ Transit-oriented developments, such as a mixed use residential or commercial area designed to maximize access to public transportation; similar to above criterion, but where the development center is within one half mile of a mass transit center (e.g. bus, rail, light rail or commuter train station). Such projects would not be able to take credit for both categories, but may have greater credit assigned | ☐ Redevelopment projects in an established historic district, historic preservation area, or similar significant city area including core City Center areas (to be defined through mapping). |

| | | | | |
|---|---|---|---|---|
| ☐Developments with dedication of undeveloped portions to parks, preservation areas and other pervious uses. | ☐ Developments in a city center area. | ☐ Developments in historic districts or historic preservation areas. | ☐ Live-work developments, a variety of developments designed to support residential and vocational needs together – similar to criteria to mixed use development; would not be able to take credit for both categories. | ☐In-fill projects, the conversion of empty lots and other underused spaces into more beneficially used spaces, such as residential or commercial areas. |

| | |
|---|---|
| Calculation of Water Quality Credits (if applicable) | N/A |

## IV.4.2 Alternative Compliance Plan Information

Describe an alternative compliance plan (if applicable). Include alternative compliance obligations (i.e., gallons, pounds) and describe proposed alternative compliance measures. *Refer to Section 7.II 3.0 in the WQMP.*

N/A

# Section V Inspection/Maintenance Responsibility for BMPs

Fill out information in table below. Prepare and attach an Operation and Maintenance Plan. Identify the mechanism through which BMPs will be maintained. Inspection and maintenance records must be kept for a minimum of five years for inspection by the regulatory agencies. *Refer to Section 7.II 4.0 in the Model WQMP.*

| BMP INSPECTION/MAINTENANCE | | | |
|---|---|---|---|
| **BMP** | **Responsible Party(s)** | **Inspection/Maintenance Activities Required** | **Minimum Frequency of Activities** |
| **HYDROLOGIC SOURCE CONTROL BMPs** | | | |
| HSC-3 Street Trees | Owner | Conduct general inspection and maintenance monthly per routine landscaping maintenance activities. Trim trees as needed. Conduct bi-annual tree health evaluations. | After significant storm events and monthly with landscaping maintenance |
| **BIOTREATMENT BMPs** | | | |
| Proprietary Biotreatment | Owner | Comply with "Operations & Maintenance" by manufacturer. Attached with Final WQMP.<br><br>Inspect unit for accumulated debris and sediment and plant health; remove trash; trim vegetation.<br><br>Scarify media to promote infiltration. Add additional mulch | Before and after major storm events. |
| | | Replace 3" to 6" of media layer. | 10 years |
| **NON-STRUCTURAL SOURCE CONTROL BMPs** | | | |
| N1 Education for Property Owners, Tenants and Occupants | Owner | Educational materials will be provided to the owner. Materials shall include those shown in Section VII of this WQMP. | At project completion |
| N2 Activity Restrictions | Owner | Owner will follow activity restrictions to protect surface water quality. | Ongoing |

| | | | |
|---|---|---|---|
| N3 Common Area Landscape Management | Owner | Maintenance shall be consistent with City requirements; any fertilizer and/or pesticide usages shall be consistent with City and manufacturer guidelines for use of fertilizers and pesticides. Maintenance includes mowing, weeding, and debris removal. Trimming, replanting and replacement of mulch shall be performed on an as-needed basis. Trimmings, clippings, and other waste shall be properly disposed of off-site in accordance with local regulations. Materials temporarily stockpiled during maintenance activities shall be placed away from water courses and drain inlets. | Monthly and as needed |
| N4 BMP Maintenance | Owner | Maintenance of BMPs implemented at the project site shall be performed at the frequency prescribed in this WQMP. Records of inspections and BMP maintenance shall be maintained by the City and documented with the WQMP. | Ongoing |
| N11 Common Area Litter control | Owner | Litter patrol, violations investigation, reporting and other litter control activities shall be performed by the owner as needed and in conjunction with maintenance activities for common areas. | Ongoing |
| N12 Employee Training | Owner | Owner till train all current and new employees on maintenance procedures for all site BMPs | Ongoing |
| N14 Common Area Catch Basin Inspection | Owner | Catch basin inlets, area drains, curb-and-gutter systems and other drainage systems shall be inspected prior to October 1st of each year and after large storm events. If necessary, drains shall be cleaned prior to any succeeding rain events. 80% of private facilities shall be inspected and cleaned annually, with 100% of facilities inspected and maintained within a 2-year period. | Annually |
| N15 Street Sweeping | Owner | Parking lots and drive aisles will be swept and kept clear of debris. | Monthly |

| **STRUCTURAL SOURCE CONTROL BMPs** | | | |
|---|---|---|---|
| S1 Provide storm drain system stencilling and signage | Owner | As a part of the final civil engineering drawings, it will be required by the owner to stencil on all of the project's catch basins, where applicable in paved areas, the words, "No Dumping - Drains to Ocean." Storm drain stencils shall be inspected for legibility, at minimum, once prior to the storm season, no later than October 1st each year. Those determined to be illegible will be re-stencilled as soon as possible. | Annually |
| S3 – Trash & Waste Storage Areas | Owner | Trash areas shall be inspected to insure no leaks or spills from the trash containers. Drop inlet will be inspected and filters cleaned out. | Monthly/Quarterly (before/after major rain events) |
| S4 Use efficient irrigation systems & landscape design, water conservation, smart controllers, and source control | Owner | In conjunction with routine maintenance activities, verify that landscape design continues to function properly by adjusting properly to eliminate overspray to hardscape areas, and to verify that irrigation timing and cycle lengths are adjusted in accordance with water demands, given time of year, weather, day or night time temperatures based on system specifications and local climate patterns. | Monthly |

# Section VI Site Plan and Drainage Plan

## VI.1 SITE PLAN AND DRAINAGE PLAN

See Attachment A.

Include a site plan and drainage plan sheet set containing the following minimum information:

- Project location
- Site boundary
- Land uses and land covers, as applicable
- Suitability/feasibility constraints
- Structural BMP locations
- Drainage delineations and flow information
- Drainage connections
- BMP details

# Section VII Educational Materials

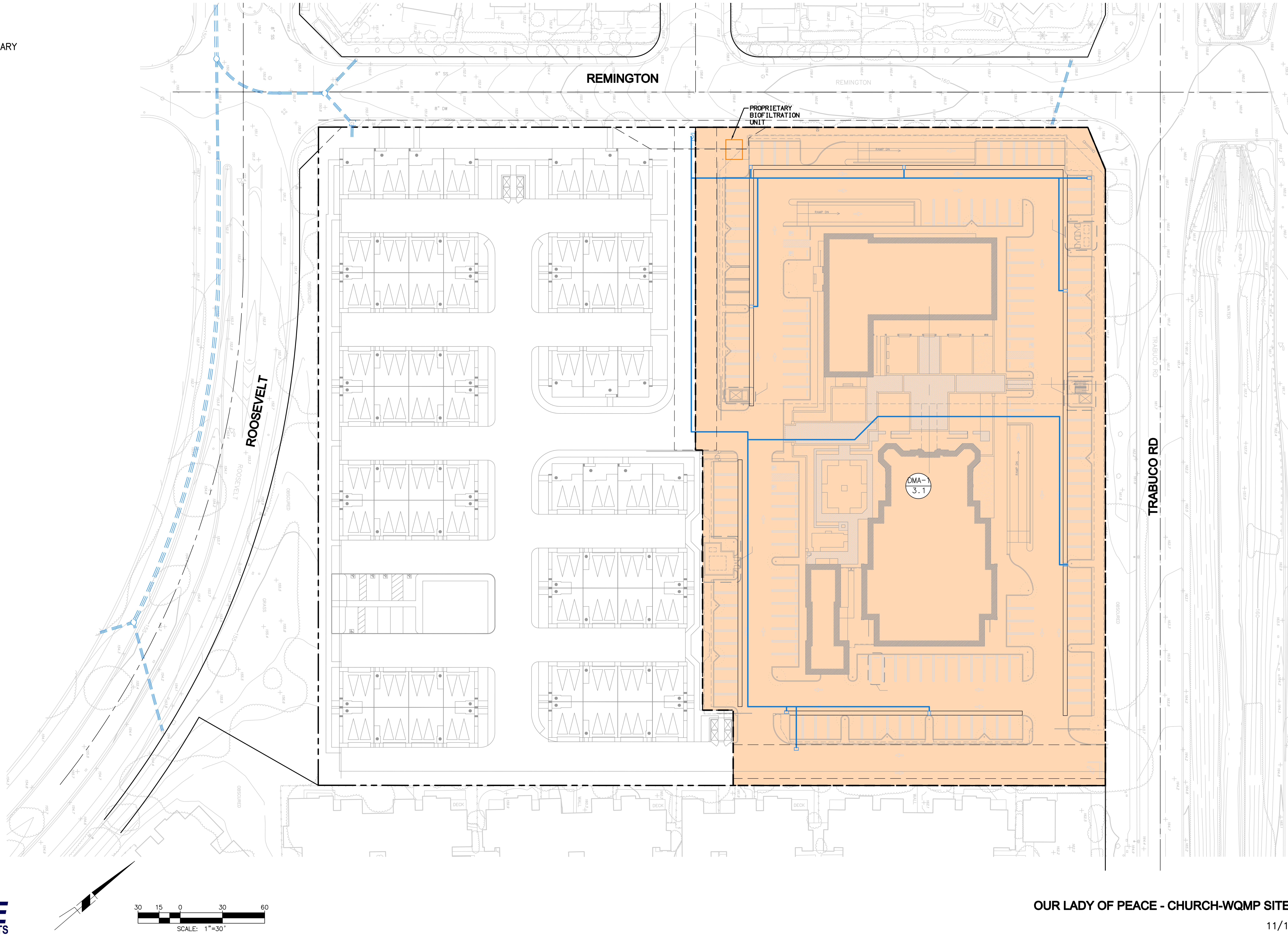
Refer to the Orange County Stormwater Program (ocwatersheds.com) for a library of materials available. For the copy submitted to the Permittee, only attach the educational materials specifically applicable to the project. Other materials specific to the project may be included as well and must be attached.

| Education Materials | | | |
|---|---|---|---|
| **Residential Material (http://www.ocwatersheds.com)** | **Check If Applicable** | **Business Material (http://www.ocwatersheds.com)** | **Check If Applicable** |
| The Ocean Begins at Your Front Door | ☒ | Tips for the Automotive Industry | ☐ |
| Tips for Car Wash Fund-raisers | ☐ | Tips for Using Concrete and Mortar | ☐ |
| Tips for the Home Mechanic | ☐ | Tips for the Food Service Industry | ☐ |
| Homeowners Guide for Sustainable Water Use | ☐ | Proper Maintenance Practices for Your Business | ☒ |
| Household Tips | ☐ | **Other Material** | **Check If Attached** |
| Proper Disposal of Household Hazardous Waste | ☐ | | |
| Recycle at Your Local Used Oil Collection Center (North County) | ☐ | | ☐ |
| Recycle at Your Local Used Oil Collection Center (Central County) | ☐ | | ☐ |
| Recycle at Your Local Used Oil Collection Center (South County) | ☐ | | ☐ |
| Tips for Maintaining a Septic Tank System | ☐ | | ☐ |
| Responsible Pest Control | ☒ | | ☐ |
| Sewer Spill | ☐ | | ☐ |
| Tips for the Home Improvement Projects | ☐ | | ☐ |
| Tips for Horse Care | ☐ | | ☐ |
| Tips for Landscaping and Gardening | ☒ | | ☐ |
| Tips for Pet Care | ☐ | | ☐ |
| Tips for Pool Maintenance | ☐ | | ☐ |
| Tips for Residential Pool, Landscape and Hardscape Drains | ☐ | | ☐ |
| Tips for Projects Using Paint | ☐ | | ☐ |

# Attachment A: Site Plan

## LEGEND

- DRAINAGE BOUNDARY
- STORM DRAIN
- EX STORM DRAIN
- DMA-1 3.1 AREA
- PROPRIETARY BIOFILTRATION UNIT





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**OUR LADY OF PEACE - CHURCH-WQMP SITE PLAN**

11/10/2025


Plotted: Nov 10, 2025 – 8:35pm by: A_Jaramillo Drawing Name: N:\20.151.000\DLV\REPORTS\PWQMP-CHURCH\OLP CHURCH-WQMP SITE PLAN.dwg

# Attachment B: Worksheets and Calculations

## Worksheet D: Capture Efficiency Method for Flow-Based BMPs

| | | | | |
|---|---|---|---|---|
| ***Step 1: Determine the design capture storm depth used for calculating volume*** | | | | |
| 1 | Enter the time of concentration, $T_c$ (min) (See **Appendix** IV.2) | $T_c$= | 8.90 | |
| 2 | Using Figure III.4, determine the design intensity at which the estimated time of concentration ($T_c$) achieves 80% capture efficiency, $I_1$ | $I_1$= | 0.23 | in/hr |
| 3 | Enter the effect depth of provided HSCs upstream, $d_{HSC}$ (inches) (Worksheet A) | $d_{HSC}$= | 0 | inches |
| 4 | Enter capture efficiency corresponding to $d_{HSC}$, $Y_2$ (Worksheet A) | $Y_2$= | 0 | % |
| 5 | Using Figure III.4, determine the design intensity at which the time of concentration ($T_c$) achieves the upstream capture efficiency($Y_2$), $I_2$ | $I_2$= | 0 | |
| 6 | Determine the design intensity that must be provided by BMP, $I_{design}= I_1 - I_2$ | $I_{design}$= | 0.23 | |
| ***Step 2: Calculate the design flowrate*** | | | | |
| 1 | Enter Project area tributary to BMP (s), *A* (acres) | A= | 3.1 | acres |
| 2 | Enter Project Imperviousness, *imp* (unitless) | imp= | 0.77 | |
| 3 | Calculate runoff coefficient, $C= (0.75 \times imp) + 0.15$ | C= | 0.73 | |
| 4 | Calculate design flowrate, $Q_{design}= (C \times i_{design} \times A)$ | $Q_{design}$= | 0.52 | cfs |
| ***Supporting Calculations*** 0.73 x 0.23in/hr x 3.1ac x 43,560 sqft/ac x 1hr/60min x 1min/60sec x 1ft/12in = 0.52 cfs | | | | |

Describe system:

| MODEL # | DIMENSIONS | WETLANDMEDIA SURFACE AREA (sq. ft.) | TREATMENT FLOW RATE (cfs) |
|---|---|---|---|
| MWS-L-8-20 | 9' x 21' | 252 | 0.577 |

Provide time of concentration assumptions:

```
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)      =         3.1  TC(MIN.) =        8.90
  EFFECTIVE AREA(ACRES) =       3.08  AREA-AVERAGED Fm(INCH/HR)=  0.03
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap = 0.100
  PEAK FLOW RATE(CFS)    =       4.44
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```

**Figure III.4. Capture Efficiency Nomograph for Off-line Flow-based Systems in Orange County**

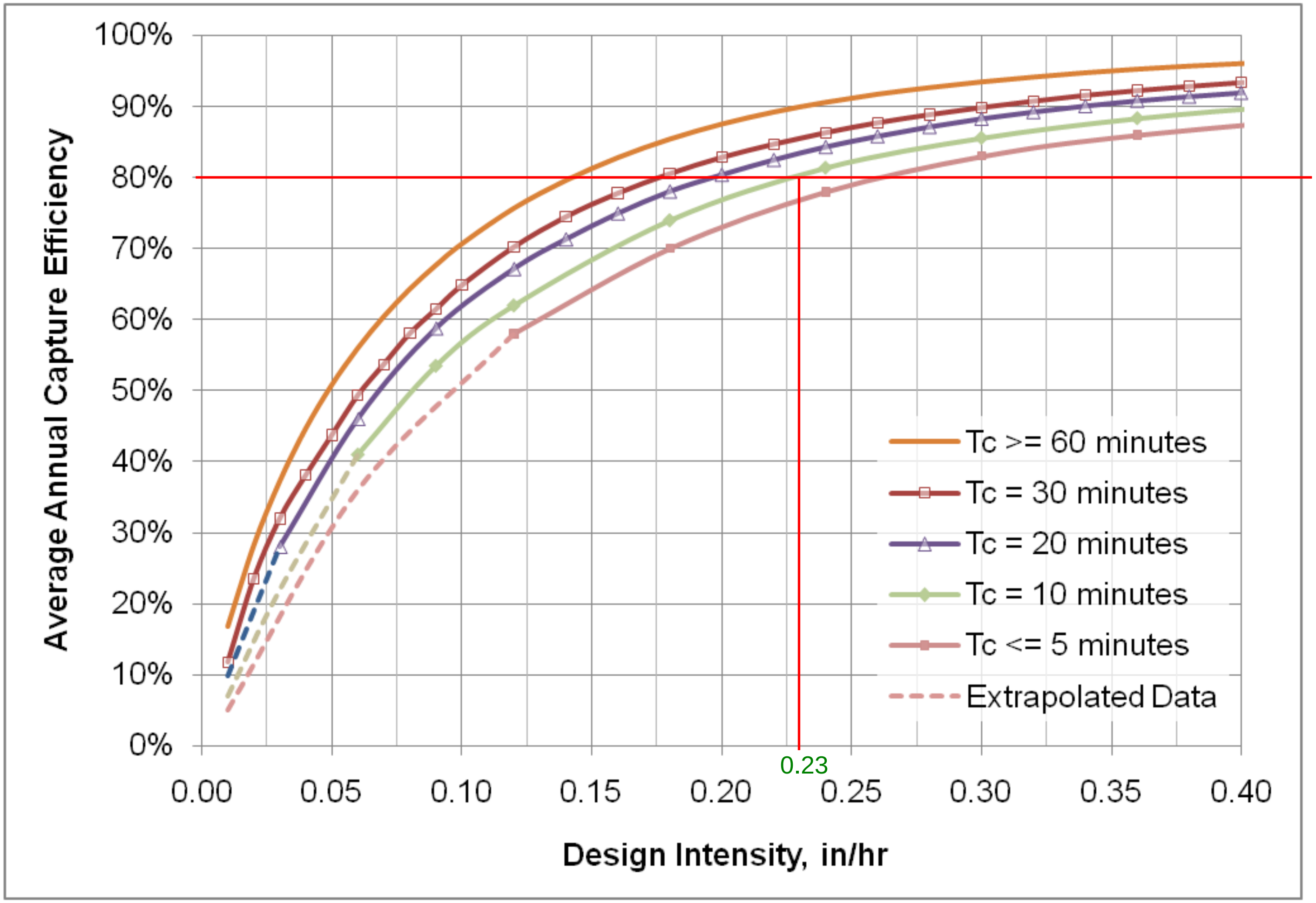


**Table X.8: Minimum Irrigated Area for Potential Partial Capture Feasibility**

| General Landscape Type | Conservation Design: $K_L$ = 0.35 | | | Active Turf Areas: $K_L$ = 0.7 | | |
|---|---|---|---|---|---|---|
| *Closest ET Station* | *Irvine* | *Santa Ana* | *Laguna* | *Irvine* | *Santa Ana* | *Laguna* |
| **Design Capture Storm Depth, inches** | **Minimum Required Irrigated Area per Tributary Impervious Acre for Potential Partial Capture, ac/ac** | | | | | |
| 0.60 | 0.66 | 0.68 | 0.72 | 0.33 | 0.34 | 0.36 |
| 0.65 | 0.72 | 0.73 | 0.78 | 0.36 | 0.37 | 0.39 |
| 0.70 | 0.77 | 0.79 | 0.84 | 0.39 | 0.39 | 0.42 |
| 0.75 | 0.83 | 0.84 | 0.90 | 0.41 | 0.42 | 0.45 |
| 0.80 | 0.88 | 0.90 | 0.96 | 0.44 | 0.45 | 0.48 |
| 0.85 | 0.93 | 0.95 | 1.02 | 0.47 | 0.48 | 0.51 |
| 0.90 | 0.99 | 1.01 | 1.08 | 0.49 | 0.51 | 0.54 |
| 0.95 | 1.04 | 1.07 | 1.14 | 0.52 | 0.53 | 0.57 |
| 1.00 | 1.10 | 1.12 | 1.20 | 0.55 | 0.56 | 0.60 |

**Worksheet J: Summary of Harvested Water Demand and Feasibility**

| | | | | |
|---|---|---|---|---|
| 1 | What demands for harvested water exist in the tributary area (check all that apply): | | | |
| 2 | Toilet and urinal flushing | | ☑ | |
| 3 | Landscape irrigation | | ☑ | |
| 4 | Other:______________________________ | | ☐ | |
| 5 | What is the design capture storm depth? (**Figure III.1**) | d | 0.8 | inches |
| 6 | What is the project size? | A | 3.06 | ac |
| 7 | What is the acreage of impervious area? | IA | 2.36 | ac |
| | **For projects with multiple types of demand ( toilet flushing, indoor demand, and/or other demand)** | | | |
| 8 | What is the minimum use required for partial capture? (**Table X.6**) | 650 | | gpd |
| 9 | What is the project estimated wet season total daily use? | Very low considering small landscape areas | | gpd |
| 10 | Is partial capture potentially feasible? (Line 9 > Line 8?) | No | | |
| | **For projects with only toilet flushing demand** | | | |
| 11 | What is the minimum TUTIA for partial capture? (**Table X.7**) | | | |
| 12 | What is the project estimated TUTIA? | | | |

**Worksheet J: Summary of Harvested Water Demand and Feasibility**

| | | | |
|---|---|---|---|
| 13 | Is partial capture potentially feasible? (Line 12 > Line 11?) | | |
| | For projects with only irrigation demand | | |
| 14 | What is the minimum irrigation area required based on conservation landscape design? ( Table X.8) | | ac |
| 15 | What is the proposed project irrigated area? (multiply conservation landscaping by 1; multiply active turf by 2) | | ac |
| 16 | Is partial capture potentially feasible? (Line 15 > Line 14?) | | |
| Provide supporting assumptions and citations for controlling demand calculation: | | | |

**Table X.6: Harvested Water Demand Thresholds for Minimum Partial Capture**

| Design Capture Storm Depth[1], inches | Wet Season Demand Required for Minimum Partial Capture[2], gpd per impervious acre |
|---|---|
| 0.60 | 490 |
| 0.65 | 530 |
| 0.70 | 570 |
| 0.75 | 610 |
| 0.80 | 650 |
| 0.85 | 690 |
| 0.90 | 730 |
| 0.95 | 770 |
| 1.00 | 810 |

1 - Based on isopluvial map (See XVI.1)

2 –Minimum Partial Capture is a performance standard whereby system performance exceeds 40 percent capture (See **Appendix** XIII) , such that the system must be considered for use even if it cannot achieve the full DCV.

# Attachment C: BMP Fact Sheets & Details

# BIO-5/BIO-7: PROPRIETARY BIOTREATMENT

**Category: Biotreatment with Partial Infiltration** (when accompanied by supplemental retention)

**Biotreatment with No Infiltration** (when used without supplemental retention)

Proprietary biotreatment BMPs are proprietary devices that are manufactured to treat stormwater. **Acceptance criteria for proprietary biotreatment BMPs are defined in Appendix J**. Proprietary BMPs that do not meet these acceptance criteria are not permitted. In addition, proprietary biotreatment BMPs must meet the definition of biofiltration in order to be used as LID biotreatment BMPs. There are two configurations of proprietary biotreatment, as explained in the following subsections.

**BIO-5: Proprietary Biotreatment with Enhanced Retention Configuration**

As standalone systems, proprietary biotreatment BMPs typically provide negligible volume reduction. To be used as a "biotreatment BMP with partial infiltration," these BMPs must be accompanied by a retention compartment. This could consist of several options:

- Permeable pavement upstream of the proprietary BMPl
- Shallow infiltration gallery or chambers downstream of the BMP, connected to underdrains.
- Proprietary biotreatment downstream of a cistern for harvest and use.
- Use of adequate hydrologic source controls in the watershed to meet volume reduction targets (see Sizing section of this Fact Sheet).
- Other configurations that are determined to be appropriate to maximize the feasible volume reduction for the DMA.

Guidance for retention compartments is provided in other fact sheets, such as INF-5 (Permeable Pavement) and INF-6 (Underground Infiltration).

**BIO-7: Standard Configuration without Supplemental Retention**

For conditions that do not require partial infiltration, volume retention is not a performance goal. Acceptable proprietary biotreatment BMPs may be used as standalone systems. Guidance related to complementary retention can be disregarded.

## Pollutant Removal Considerations

BMPs that meet the acceptance criteria in **Appendix J** are considered to provide adequate treatment for pollutants of concern. According to these critera, there are different levels of treatment certification needed for different pollutants of concern.

## Recommended Design Criteria and Considerations

| Design Criteria | Intent/Rationale |
|---|---|
| ☐ Sediment sources should be controlled prior to operation of the system. | Proprietary systems are susceptible to clogging similar to other BMPs. Systems should not be used in areas that will continue to receive elevated sediment loading following construction, such as from open space area. |
| ☐ When accompanied by infiltration compartments, the ponding should not be higher than the underdrain elevation of the proprietary BMP. | This is intended to ensure that the complementary retention compartment does not reduce the hydraulic capacity of the proprietary biotreatment BMP. |
| ☐ When accompanied by infiltration compartments, these infiltration BMPs must adhere to siting guidance found in the respective fact sheet for the BMP | Specific siting considerations apply to infiltration BMPs. |
| ☐ Proprietary biotreatment systems typically do not require separate pretreatment | These BMPs typically include integrated mechanisms for pretreatment. |
| ☐ Proprietary BMPs must be designed in a manner consistent with manufacturer recommendations and consistent with the design configuration that was tested as part of the BMP certification | Proprietary devices have device-specific design, installation, and maintenance details which must be followed for proper treatment results. |
| ☐ In right of way areas, plant selection should not impair traffic sightlines or vehicle access. | Vegetation must not be prohibitive for typical vehicular movement and parking access needs. |
| ☐ Manufacturer guidance on vegetation selection and establishment should be followed | Manufacturers have experience with plant survival in specific climates for the BMP-specific conditions. |

## Calculations and Sizing Method

Proprietary Biotreatment BMPs are flow-based BMPs. See **Appendix E** for acceptable sizing methods.

Supplemental retention elements (for BIO-5 configuration) should be sized for one of the following targets, where possible:

- Approximately 40 percent long term volume reduction.
- Retention storage provided for approximately one-third of the DCV.
- Infiltration footprint (collective of all infiltrating elements of the project design) meeting target defined in **Section E.4.2**.

## Construction Guidance

| | Construction Guidance | Intent/Rationale |
|---|---|---|
| ☐ | Plans should include a construction sequence for the BMP. Revisions proposed by the contractor should be reviewed by the engineer. The construction sequence should address erosion control, utilities, BMP installation, inspections, testing and certifications, vegetation, stabilization, and post-construction monitoring. | Construction sequencing is critical to avoid issues/damage and allow appropriate inspections, testing, and certifications to be performed. |
| ☐ | Provide for inspection of buried infrastructure (e.g., underdrain, filter course) before it is buried. | It is impractical to inspect buried elements once they are covered. |
| ☐ | Fully stabilize sources of sediment within the tributary area (i.e., no exposed soil) prior to placing the finished BMP into service. | Sediment loading can seriously impair the capacity of the BMP. |
| ☐ | Allow plants and mulch to stabilize for as long as practicable (preferably several months) prior to placing the finished BMP into service. | Stabilization of the system allows plants to mature before stressing the system with stormwater loading. |

## O&M Activities and Frequencies

| Activity | Frequency |
|---|---|
| **GENERAL INSPECTIONS** | |
| Remove trash and debris | Four times per year during wet season, including inspection just before the wet season and within 24 hours after at least two storm events ≥ 0.5 inches. |
| Identify excess erosion or scour | |
| Identify sediment accumulation that requires maintenance | |
| Inspect during storm event, when possible, to estimate treatment capacity and determine if premature bypass is occurring | |
| Evaluate plant health and need for corrective action | |
| Identify any needed corrective maintenance that will require site-specific planning or design | |
| **OPERATION AND MAINTENANCE** | |
| • O&M of proprietary BMPs must follow established manufacturer guidelines<br>• O&M of accompanying retention BMPs should follow the guidelines established in the associated fact sheet for that BMP. | |

Bio Clean

A Forterra Company

# Modular Wetlands® System Linear

## A Stormwater Biofiltration Solution

# OVERVIEW

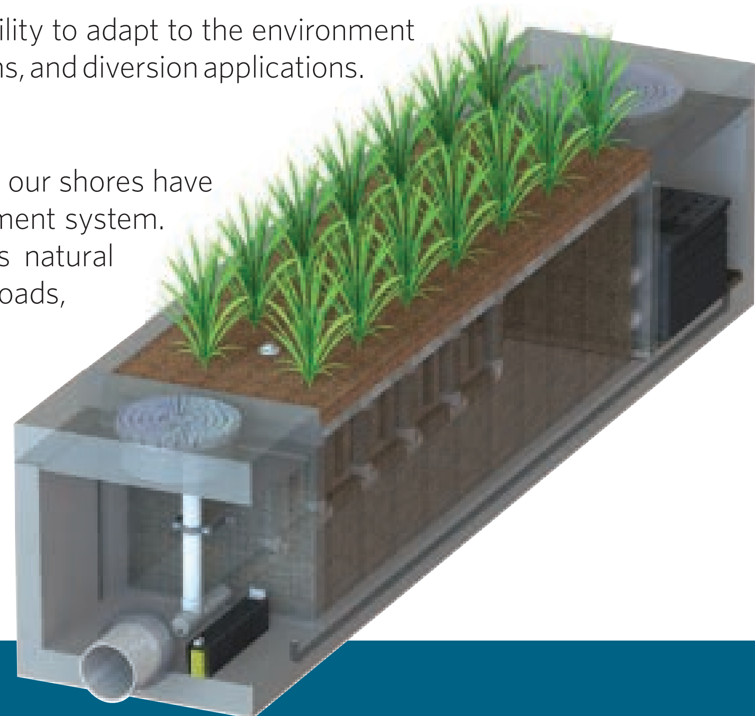
The Bio Clean Modular Wetlands® System Linear represents a pioneering breakthrough in stormwater technology as the only biofiltration system to utilize patented horizontal flow, allowing for a smaller footprint, higher treatment capacity, and a wide range of versatility. While most biofilters use little or no pretreatment, the Modular Wetlands® incorporates an advanced pretreatment chamber that includes separation and pre-filter cartridges. In this chamber, sediment and hydrocarbons are removed from runoff before entering the biofiltration chamber, reducing maintenance costs and improving performance.

Horizontal flow also gives the system the unique ability to adapt to the environment through a variety of configurations, bypass orientations, and diversion applications.

### The Urban Impact

For hundreds of years, natural wetlands surrounding our shores have played an integral role as nature's stormwater treatment system. But as cities grow and develop, our environment's natural filtration systems are blanketed with impervious roads, rooftops, and parking lots.

Bio Clean understands this loss and has spent years re-establishing nature's presence in urban areas, and rejuvenating waterways with the Modular Wetlands® System Linear.



## PERFORMANCE

The Modular Wetlands® continues to outperform other treatment methods with superior pollutant removal for TSS, heavy metals, nutrients, hydrocarbons, and bacteria. Since 2007 the Modular Wetlands® has been field tested on numerous sites across the country and is proven to effectively remove pollutants through a combination of physical, chemical, and biological filtration processes. In fact, the Modular Wetlands® harnesses some of the same biological processes found in natural wetlands in order to collect, transform, and remove even the most harmful pollutants.

| | | | | |
|---|---|---|---|---|
| 66% REMOVAL OF DISSOLVED ZINC | 69% REMOVAL OF TOTAL ZINC | 38% REMOVAL OF DISSOLVED COPPER | 64% REMOVAL OF TOTAL PHOSPHORUS | |
| 45% REMOVAL OF NITROGEN | 50% REMOVAL OF TOTAL COPPER | 95% REMOVAL OF MOTOR OIL | 67% REMOVAL OF ORTHO PHOSPHORUS | 85% REMOVAL OF TSS |

# APPROVALS

The Modular Wetlands® System Linear has successfully met years of challenging technical reviews and testing from some of the most prestigious and demanding agencies in the nation and perhaps the world. Here is a list of some of the most high-profile approvals, certifications, and verifications from around the country.

State of Washington TAPE GULD Basic Enhanced/Metals Phosphorus

**Washington State Department of Ecology TAPE Approved**
The MWS Linear is approved for General Use Level Designation (GULD) for Basic, Enhanced, and Phosphorus treatment at 1 gpm/ft$^2$ loading rate. The highest performing BMP on the market for all main pollutant categories.

CA

**California Water Resources Control Board, Full Capture Certification**
The Modular Wetlands® System is the first biofiltration system to receive certification as a full capture trash treatment control device.

VA

**Virginia Department of Environmental Quality, Assignment**
The Virginia Department of Environmental Quality assigned the MWS Linear the highest phosphorus removal rating for manufactured treatment devices to meet the new Virginia Stormwater Management Program (VSMP) regulation technical criteria.

MD

**Maryland Department of the Environment, Approved ESD**
Granted Environmental Site Design (ESD) status for new construction, redevelopment, and retrofitting when designed in accordance with the design manual.

MA

**MASTEP Evaluation**
The University of Massachusetts at Amherst – Water Resources Research Center issued a technical evaluation report noting removal rates up to 84% TSS, 70% total phosphorus, 68.5% total zinc, and more.

RI

**Rhode Island Department of Environmental Management, Approved BMP**
Approved as an authorized BMP and noted to achieve the following minimum removal efficiencies: 85% TSS, 60% pathogens, 30% total phosphorus, and 30% total nitrogen.

## ADVANTAGES

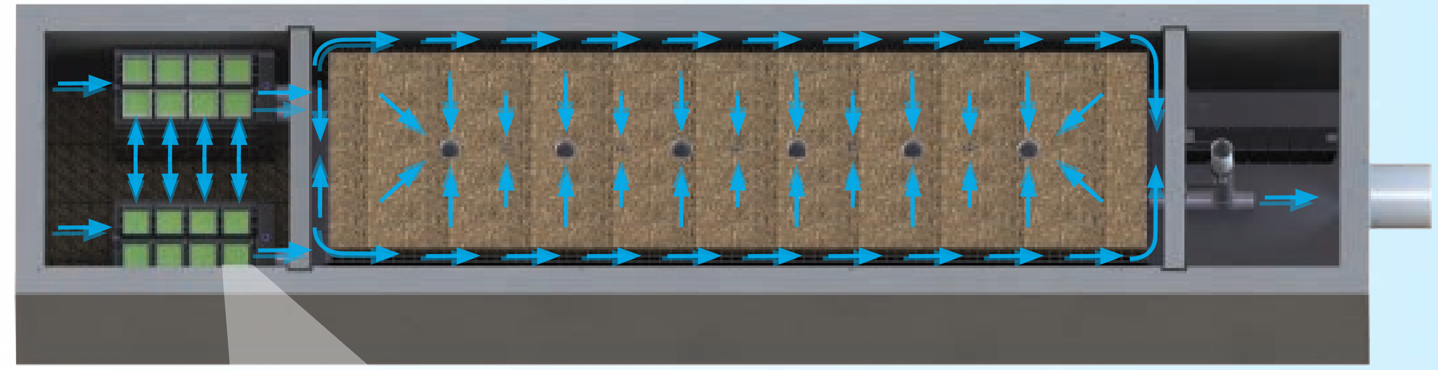
- HORIZONTAL FLOW BIOFILTRATION
- GREATER FILTER SURFACE AREA
- PRETREATMENT CHAMBER
- PATENTED PERIMETER VOID AREA
- FLOW CONTROL
- NO DEPRESSED PLANTER AREA
- AUTO DRAINDOWN MEANS NO MOSQUITO VECTOR

# OPERATION

The Modular Wetlands® System Linear is the most efficient and versatile biofiltration system on the market, and it is the only system with horizontal flow which:

- Improves performance
- Reduces footprint
- Minimizes maintenance

Figure 1 & Figure 2 illustrate the invaluable benefits of horizontal flow and the multiple treatment stages.



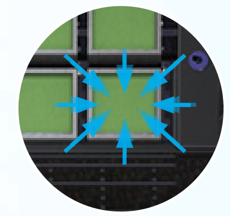


Figure 2, Top View

2x to 3x more surface area than traditional downward flow bioretention systems.

## 1 PRETREATMENT

### SEPARATION

- Trash, sediment, and debris are separated before entering the pre-filter cartridges
- Designed for easy maintenance access

### PRE-FILTER CARTRIDGES

- Over 25 sq. ft. of surface area per cartridge
- Utilizes BioMediaGREEN™ filter material
- Removes over 80% of TSS and 90% of hydrocarbons
- Prevents pollutants that cause clogging from migrating to the biofiltration chamber

## 2 BIOFILTRATION

### HORIZONTAL FLOW

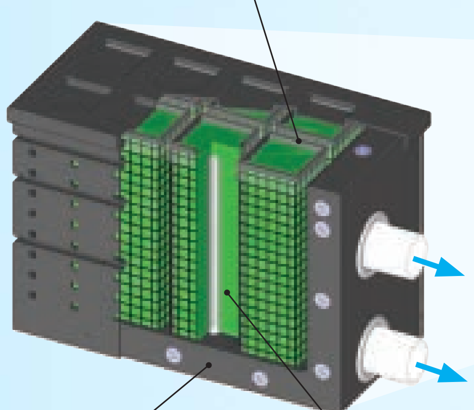
- Less clogging than downward flow biofilters
- Water flow is subsurface
- Improves biological filtration

### PATENTED PERIMETER VOID AREA

- Vertically extends void area between the walls and the WetlandMEDIA™ on all four sides
- Maximizes surface area of the media for higher treatment capacity

### WETLANDMEDIA

- Contains no organics and removes phosphorus
- Greater surface area and 48% void space
- Maximum evapotranspiration
- High ion exchange capacity and lightweight



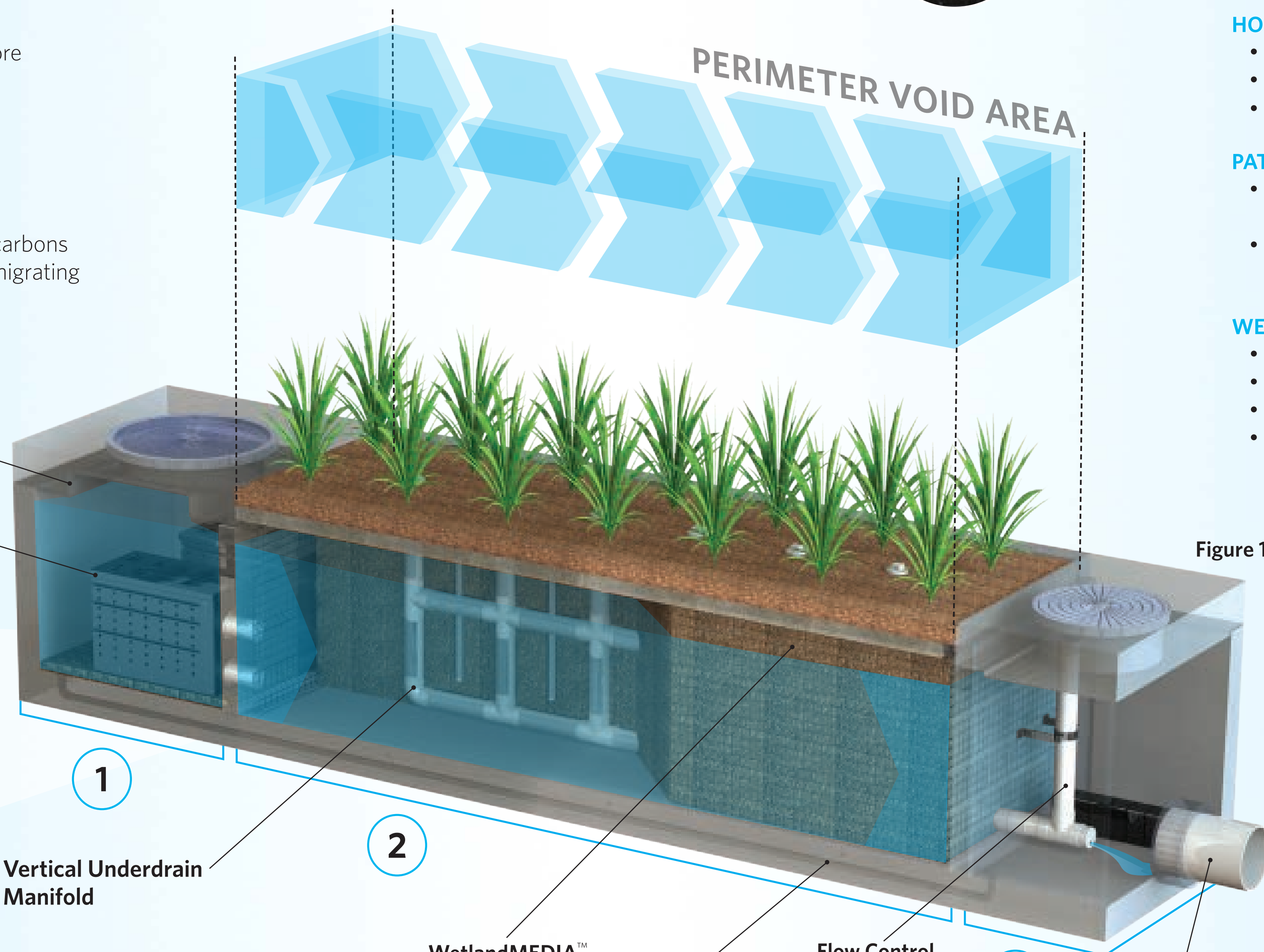




Figure 1

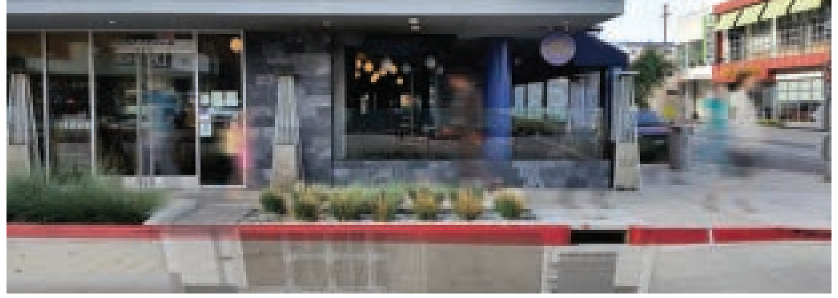
## 3 DISCHARGE

### FLOW CONTROL

- Orifice plate controls flow of water through WetlandMEDIA™ to a level lower than the media's capacity
- Extends the life of the media and improves performance

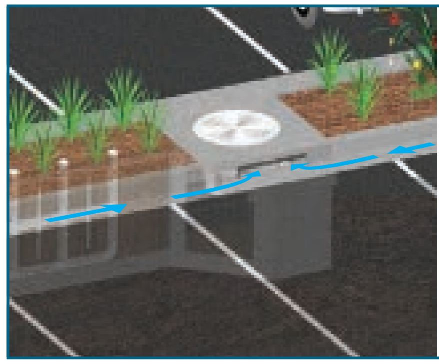
### DRAINDOWN FILTER

- The draindown is an optional feature that completely drains the pretreatment chamber
- Water that drains from the pretreatment chamber between storm events will be treated

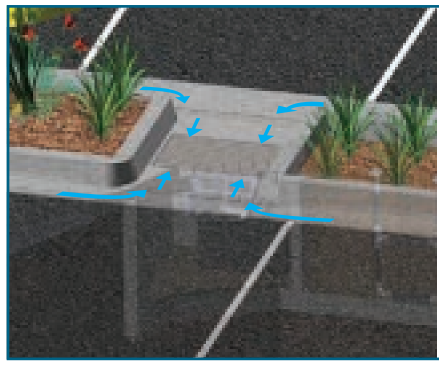
# CONFIGURATIONS

The Modular Wetlands® System Linear is the preferred biofiltration system of civil engineers across the country due to its versatile design. This highly versatile system has available "pipe-in" options on most models, along with built-in curb or grated inlets for simple integration into your storm drain design.



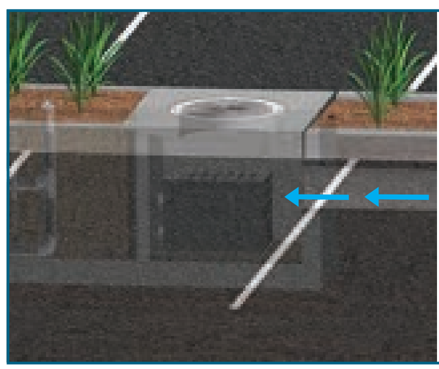
## CURB TYPE

The Curb Type configuration accepts sheet flow through a curb opening and is commonly used along roadways and parking lots. It can be used in sump or flow-by conditions. Length of curb opening varies based on model and size.



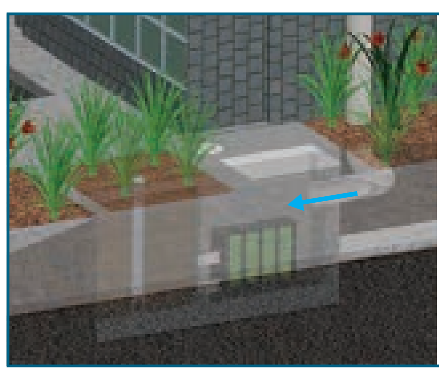
## GRATE TYPE

The Grate Type configuration offers the same features and benefits as the Curb Type but with a grated/drop inlet above the systems pretreatment chamber. It has the added benefit of allowing pedestrian access over the inlet. ADA-compliant grates are available to assure easy and safe access. The Grate Type can also be used in scenarios where runoff needs to be intercepted on both sides of landscape islands.



## VAULT TYPE

The system's patented horizontal flow biofilter is able to accept inflow pipes directly into the pretreatment chamber, meaning the Modular Wetlands® can be used in end-of-the-line installations. This greatly improves feasibility over typical decentralized designs that are required with other biofiltration/bioretention systems. Another benefit of the "pipe-in" design is the ability to install the system downstream of underground detention systems to meet water quality volume requirements.



## DOWNSPOUT TYPE

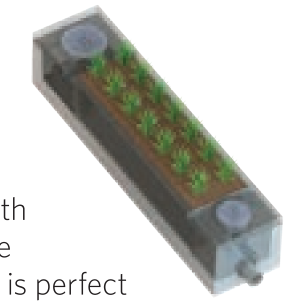
The Downspout Type is a variation of the Vault Type and is designed to accept a vertical downspout pipe from rooftop and podium areas. Some models have the option of utilizing an internal bypass, simplifying the overall design. The system can be installed as a raised planter, and the exterior can be stuccoed or covered with other finishes to match the look of adjacent buildings.

# ORIENTATIONS

### SIDE-BY-SIDE

The Side-By-Side orientation places the pretreatment and discharge chamber adjacent to one another with the biofiltration chamber running parallel on either side. This minimizes the system length, providing a highly compact footprint. It has been proven useful in situations such as streets with directly adjacent sidewalks, as half of the system can be placed under that sidewalk. This orientation also offers internal bypass options as discussed below.

### END-TO-END



The End-To-End orientation places the pretreatment and discharge chambers on opposite ends of the biofiltration chamber, therefore minimizing the width of the system to 5 ft. (outside dimension). This orientation is perfect for linear projects and street retrofits where existing utilities and sidewalks limit the amount of space available for installation. One limitation of this orientation is that bypass must be external.

# BYPASS

### INTERNAL BYPASS WEIR (SIDE-BY-SIDE ONLY)

The Side-By-Side orientation places the pretreatment and discharge chambers adjacent to one another allowing for integration of internal bypass. The wall between these chambers can act as a bypass weir when flows exceed the system's treatment capacity, thus allowing bypass from the pretreatment chamber directly to the discharge chamber.

### EXTERNAL DIVERSION WEIR STRUCTURE

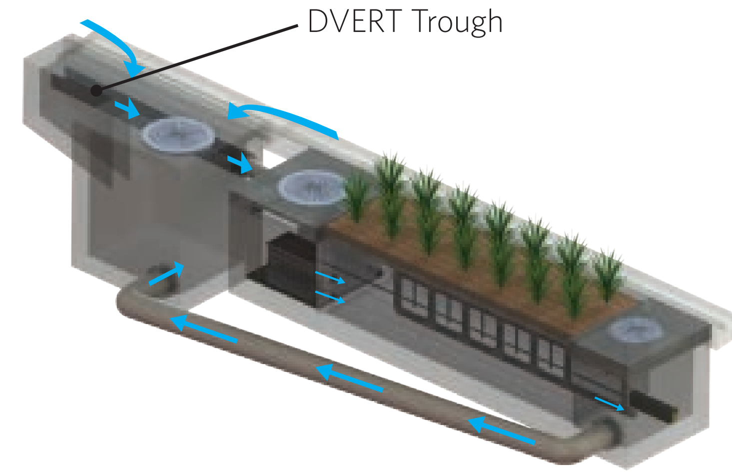
This traditional offline diversion method can be used with the Modular Wetlands® in scenarios where runoff is being piped to the system. These simple and effective structures are generally configured with two outflow pipes. The first is a smaller pipe on the upstream side of the diversion weir - to divert low flows over to the Modular Wetlands® for treatment. The second is the main pipe that receives water once the system has exceeded treatment capacity and water flows over the weir.

### FLOW-BY-DESIGN

This method is one in which the system is placed just upstream of a standard curb or grate inlet to intercept the first flush. Higher flows simply pass by the Modular Wetlands® and into the standard inlet downstream.

### DVERT LOW FLOW DIVERSION

This simple yet innovative diversion trough can be installed in existing or new curb and grate inlets to divert the first flush to the Modular Wetlands® via pipe. It works similar to a rain gutter and is installed just below the opening into the inlet. It captures the low flows and channels them over




to a connecting pipe exiting out the wall of the inlet and leading to the MWS Linear. The DVERT is perfect for retrofit and green street applications that allow the Modular Wetlands® to be installed anywhere space is available.

# SPECIFICATIONS

## FLOW-BASED DESIGNS

The Modular Wetlands® System Linear can be used in stand-alone applications to meet treatment flow requirements. Since the Modular Wetlands® is the only biofiltration system that can accept inflow pipes several feet below the surface, it can be used not only in decentralized design applications but also as a large central end-of-the-line application for maximum feasibility.

| MODEL # | DIMENSIONS | WETLANDMEDIA SURFACE AREA (sq. ft.) | TREATMENT FLOW RATE (cfs) |
|---|---|---|---|
| MWS-L-4-4 | 4' x 4' | 23 | 0.052 |
| MWS-L-4-6 | 4' x 6' | 32 | 0.073 |
| MWS-L-4-8 | 4' x 8' | 50 | 0.115 |
| MWS-L-4-13 | 4' x 13' | 63 | 0.144 |
| MWS-L-4-15 | 4' x 15' | 76 | 0.175 |
| MWS-L-4-17 | 4' x 17' | 90 | 0.206 |
| MWS-L-4-19 | 4' x 19' | 103 | 0.237 |
| MWS-L-4-21 | 4' x 21' | 117 | 0.268 |
| MWS-L-6-8 | 7' x 9' | 64 | 0.147 |
| MWS-L-8-8 | 8' x 8' | 100 | 0.230 |
| MWS-L-8-12 | 8' x 12' | 151 | 0.346 |
| MWS-L-8-16 | 8' x 16' | 201 | 0.462 |
| MWS-L-8-20 | 9' x 21' | 252 | 0.577 |
| MWS-L-8-24 | 9' x 25' | 302 | 0.693 |
| MWS-L-10-20 | 10' x 20' | 302 | 0.693 |

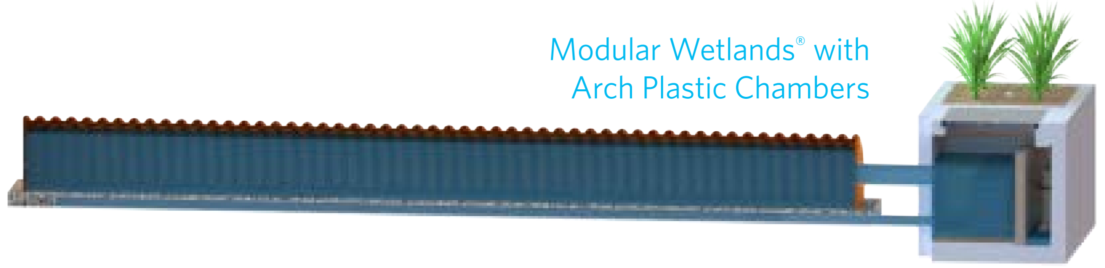
# VOLUME-BASED DESIGNS

## HORIZONTAL FLOW BIOFILTRATION ADVANTAGE

Modular Wetlands® with
Box Culvert Prestorage

The Modular Wetlands® System Linear offers a unique advantage in the world of biofiltration due to its exclusive horizontal flow design: Volume-Based Design. No other biofilter has the ability to be placed downstream of detention ponds, extended dry detention basins, underground storage systems and permeable paver reservoirs. The systems horizontal flow configuration and built-in orifice control allows it to be installed with just 6" of fall between inlet and outlet pipe for a simple connection to projects with shallow downstream tie-in points. In the example above, the Modular Wetlands® is installed downstream of underground box culvert storage. Designed for the water quality volume, the Modular Wetlands® will treat and discharge the required volume within local draindown time requirements.

Modular Wetlands® with
Arch Plastic Chambers



### DESIGN SUPPORT

Bio Clean engineers are trained to provide you with superior support for all volume sizing configurations throughout the country. Our vast knowledge of state and local regulations allow us to quickly and efficiently size a system to maximize feasibility. Volume control and hydromodification regulations are expanding the need to decrease the cost and size of your biofiltration system. Bio Clean will help you realize these cost savings with the Modular Wetlands®, the only biofilter than can be used downstream of storage BMPs.

## ADVANTAGES

- LOWER COST THAN FLOW-BASED DESIGN
- MEETS LID REQUIREMENTS
- BUILT-IN ORIFICE CONTROL STRUCTURE
- WORKS WITH DEEP INSTALLATIONS

# APPLICATIONS

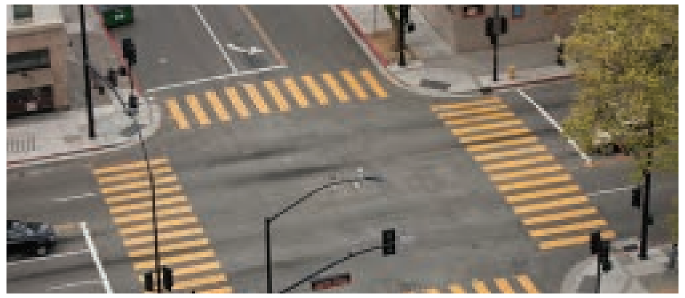
The Modular Wetlands® System Linear has been successfully used on numerous new construction and retrofit projects. The system's superior versatility makes it beneficial for a wide range of stormwater and waste water applications - treating rooftops, streetscapes, parking lots, and industrial sites.

**INDUSTRIAL**

Many states enforce strict regulations for discharges from industrial sites. The Modular Wetlands® has helped various sites meet difficult EPA-mandated effluent limits for dissolved metals and other pollutants.

**RESIDENTIAL**

Low to high density developments can benefit from the versatile design of the Modular Wetlands®. The system can be used in both decentralized LID design and cost-effective end-of-the-line configurations.

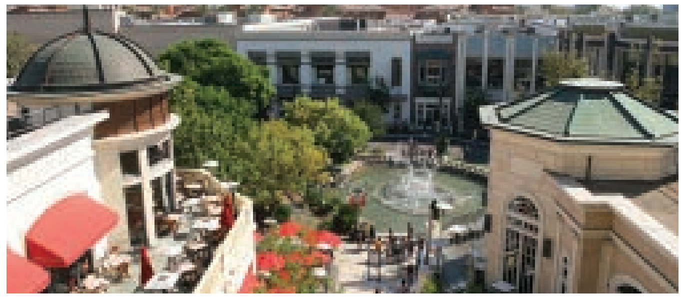


**STREETS**

Street applications can be challenging due to limited space. The Modular Wetlands® is very adaptable, and it offers the smallest footprint to work around the constraints of existing utilities on retrofit projects.

**PARKING LOTS**

Parking lots are designed to maximize space and the Modular Wetlands'® 4 ft. standard planter width allows for easy integration into parking lot islands and other landscape medians.



**COMMERCIAL**

Compared to bioretention systems, the Modular Wetlands® can treat far more area in less space, meeting treatment and volume control requirements.

**MIXED USE**

The Modular Wetlands® can be installed as a raised planter to treat runoff from rooftops or patios, making it perfect for sustainable "live-work" spaces.

**More applications include:**

• Agriculture • Reuse • Low Impact Development • Waste Water

# PLANT SELECTION

Abundant plants, trees, and grasses bring value and an aesthetic benefit to any urban setting, but those in the Modular Wetlands® System Linear do even more - they increase pollutant removal. What's not seen, but very important, is that below grade, the stormwater runoff/flow is being subjected to nature's secret weapon: a dynamic physical, chemical, and biological process working to break down and remove non-point source pollutants. The flow rate is controlled in the Modular Wetlands®, giving the plants more contact time so that pollutants are more successfully decomposed, volatilized, and incorporated into the biomass of the Modular Wetlands'® micro/macro flora and fauna.

A wide range of plants are suitable for use in the Modular Wetlands®, but selections vary by location and climate. View suitable plants by visiting biocleanenvironmental.com/plants.

# INSTALLATION

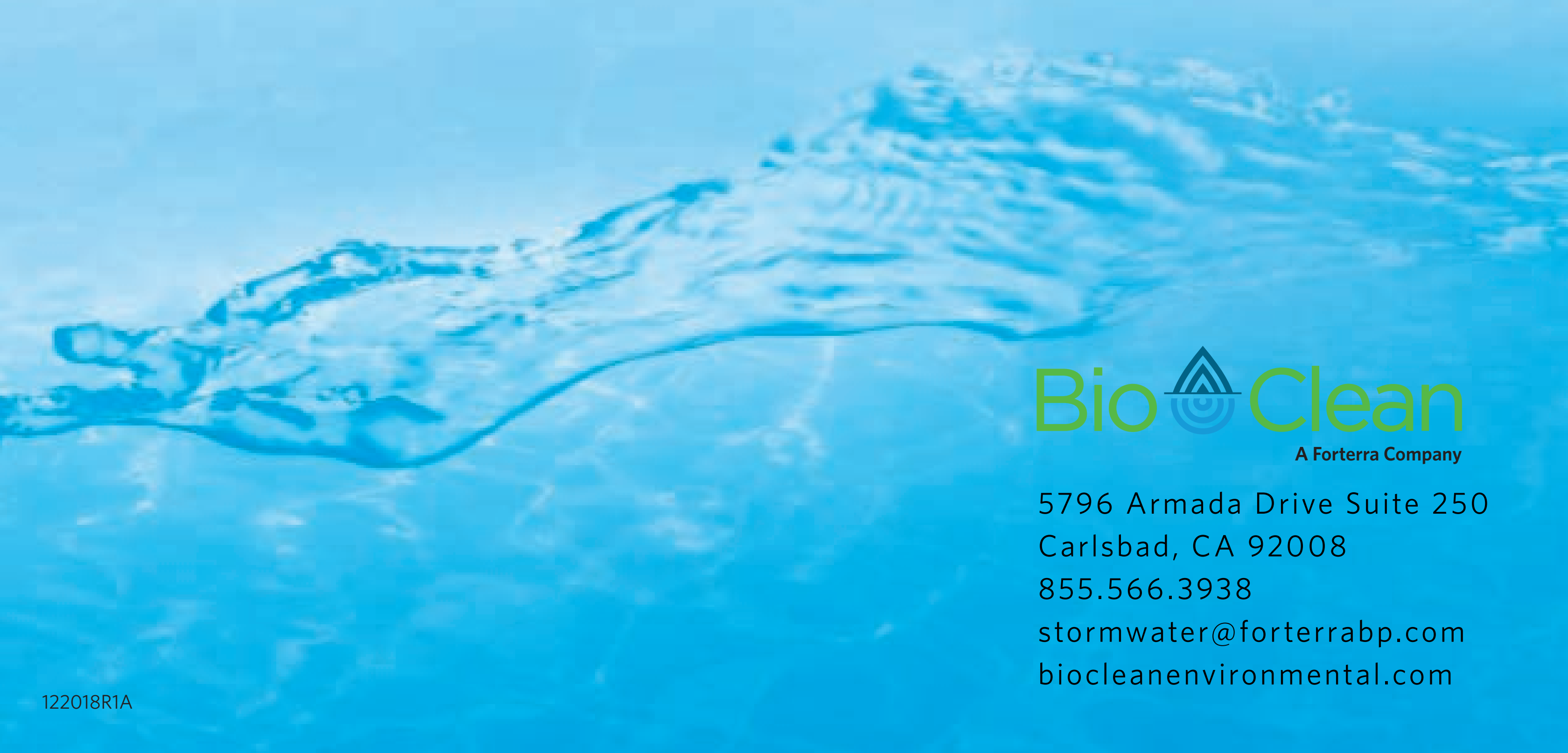

The Modular Wetlands® is simple, easy to install, and has a space-efficient design that offers lower excavation and installation costs compared to traditional tree-box type systems. The structure of the system resembles precast catch basin or utility vaults and is installed in a similar fashion.

The system is delivered fully assembled for quick installation. Generally, the structure can be unloaded and set in place in 15 minutes. Our experienced team of field technicians is available to supervise installations and provide technical support.

# MAINTENANCE

Reduce your maintenance costs, man hours, and materials with the Modular Wetlands®. Unlike other biofiltration systems that provide no pretreatment, the Modular Wetlands® is a self-contained treatment train which incorporates simple and effective pretreatment.

Maintenance requirements for the biofilter itself are almost completely eliminated, as the pretreatment chamber removes and isolates trash, sediments, and hydrocarbons. What's left is the simple maintenance of an easily accessible pretreatment chamber that can be cleaned by hand or with a standard vac truck. Only periodic replacement of low-cost media in the pre-filter cartridges is required for long-term operation, and there is absolutely no need to replace expensive biofiltration media.



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122018R1A

# Attachment D: Geotechnical Report

July 1, 2024 Project No. 23174-01

***Roman Catholic Bishop of Orange***
13280 Chapman Avenue
Garden Grove, CA 92840

***Subject:*** ***Preliminary Geotechnical Subsurface Evaluation, Proposed Church Development, 14010 Remington, Irvine, California***

In accordance with your request, LGC Geotechnical, Inc. has performed a geotechnical subsurface evaluation for the proposed church development located at 14010 Remington in the City of Irvine, California. This report summarizes the results of our background review, subsurface exploration, and geotechnical analyses of the data collected, and presents our findings, conclusions, and preliminary recommendations for the proposed church project.

If you should have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Respectfully,

***LGC Geotechnical, Inc.***

Barry Graham, CEG 2749
Project Geologist

CERTIFIED ENGINEERING GEOLOGIST BARRY GRAHAM No. 2749 STATE OF CALIFORNIA

Dennis Boratynec, GE 2770
Vice President

REGISTERED PROFESSIONAL ENGINEER DENNIS BORATYNEC No. 2770 GEOTECHNICAL STATE OF CALIFORNIA

Branden Petersen, EIT
Senior Staff Engineer

DJB/BPP/BPG/amm

Distribution: (1) Addressee (electronic copy, wet signed copies can be provided upon request)

## *TABLE OF CONTENTS*

## *TABLE OF CONTENTS (Cont'd)*



## *LIST OF TABLES, ILLUSTRATIONS, & APPENDICES*

### *Tables*



### *Figures*



### *Appendices*

# *1.0 INTRODUCTION*

## *1.1 Purpose and Scope of Services*

This report presents the results of our geotechnical subsurface evaluation for the proposed church development located at 14010 Remington in the City of Irvine, California (see Site Location Map, Figure 1). The purpose of our work was to collect subsurface data in order to confirm that the site can be developed from a geotechnical perspective. Our scope of services included:

- Review of pertinent readily available geotechnical information and geologic maps (Appendix A).
- Subsurface evaluation including excavation, sampling, and logging of four small-diameter hollow stem borings.
- Infiltration testing at two of the small-diameter hollow-stem borings.
- Laboratory testing of representative samples obtained during our subsurface evaluation (Appendix C).
- Geotechnical analysis and evaluation of the data obtained, including:
  - Suitability of the site for the proposed development from a geotechnical standpoint;
  - Description of the site geology, and subsurface soil and groundwater conditions;
  - Evaluation of the seismic conditions at the site, including seismic design criteria based on the 2022 California Building Code (CBC); and
  - Recommendations for remedial grading operations and site preparation.
- Preparation of this report presenting our findings, conclusions and recommendations with respect to the proposed site development.

## *1.2 Background/Project Description*

The approximately 3-acre site is bound to the north by Trabuco Road, to the west by Remington, to the south by a parking lot for the associated church and open space and east by a residential development. Review of historical aerial photographs suggests the site was undeveloped farmland prior to 1980. By 1985, Roosevelt was constructed but the site remained undeveloped until 2000. The church and associated parking lot was completed by 2002. The site has essentially remained unchanged since then.

The proposed development will consist of a new parish hall, plaza, church, rectory, parking lot and other associated improvements. We understand that the proposed church development will have a basement approximately 9 feet deep while the other structures will be at grade with relatively light building loads (column and wall loads assumed to be a maximum of approximately 30 kips and 3 kips per lineal foot, respectively). At this time, we do not have any design cut/fill information, but we assume maximum design cuts will be a maximum of approximately 10 feet (for basement) and design fills will be less than 5 feet.

The recommendations provided herein are based upon the estimated structural loading, potential grade changes, and expected layout information above. We understand that the project plans are currently being developed at this time; LGC Geotechnical should be provided with updated project plans (including grading, foundation, retaining/basement walls, etc.) and any changes to the assumed structural loads when they become available, in order to either confirm or modify the recommendations provided herein.

Note that an evaluation of the adjacent proposed residential site (LGC Geotechnical, 2024) was conducted at a similar time to the subject proposed church site. The limits of these sites can be found in Figure 2 - Boring Location Map. The laboratory test results from the adjacent residential evaluation were also included within this report.

## 1.3 *Subsurface Evaluation*

LGC Geotechnical performed a subsurface geotechnical evaluation of the site consisting of the excavation of 4 hollow-stem auger borings.

Four hollow-stem borings (HS-5, HS-6, I-3, and I-4) were drilled to depths ranging from approximately 10 to 70 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were excavated using a track-mounted drill rig equipped with 8-inch-diameter hollow-stem augers. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler generally obtained at 2.5 to 5-foot vertical increments. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch-tall brass rings. The SPT sampler and MCD sampler were driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. Bulk samples were also collected and logged at select depths for laboratory testing. At the completion of drilling, the borings were backfilled with the native soil cuttings and tamped. Some settlement of the backfill soils may occur over time.

The approximate locations of borings are shown on the Geotechnical Exploration Location Map (Figure 2). Boring logs are presented in Appendix B.

## 1.4 *Field Percolation Testing*

Two falling head field percolation tests (I-3 and I-4) were performed in the approximate locations indicated on our Geotechnical Map (Figure 2). Estimation of infiltration rates for the site was accomplished in general accordance with the guidelines set forth by the County of Orange County (2013). A 3-inch diameter perforated PVC pipe with filter sock was placed in the borehole, and the annulus was backfilled with gravel, including placement of approximately 2 inches of gravel at the bottom of the borehole. The infiltration wells were pre-soaked the day prior to testing. During the pre-test, if the water level drops more than 6 inches in 25 minutes for two consecutive readings, the test procedure for coarse-grained soils was followed. If the water level did not drop 6 inches in one or both pre-test readings, the procedure for fine-grained soils was followed. The procedure for coarse-grained soils requires performing the test for one hour and taking one reading every 10 minutes from a fixed reference point. The procedure for fine-grained soils requires performing the test for six hours and taking one reading every 30 minutes from a fixed reference point.

The pre-tests indicated the procedure for the fine-grained soils should be followed. The calculated infiltration is normalized relative to the three-dimensional flow that occurs within the field test to a one-dimensional flow out of the bottom of the boring only (i.e., "Porchet Method"). The measured infiltration rates (for feasibility purposes only) are provided in Table 1 below. Please note that infiltration is discussed in Section 4.8 and field data is provided in Appendix D.

Please note that the values provided in Table 1 do not include reduction factors associated with the test procedure, site variability, and long-term siltation plugging that are used to calculate the design infiltration rate.

***TABLE 1***

***Summary of Field Infiltration Testing***

| Infiltration Test No. | Approx. Depth Below Existing Grade (ft) | Measured Infiltration Rate* (in./hr.) |
|---|---|---|
| I-3 | 10 | 0.02 |
| I-4 | 10 | 0.01 |

*Measured Infiltration Rates Do Not Include Factor of Safety.

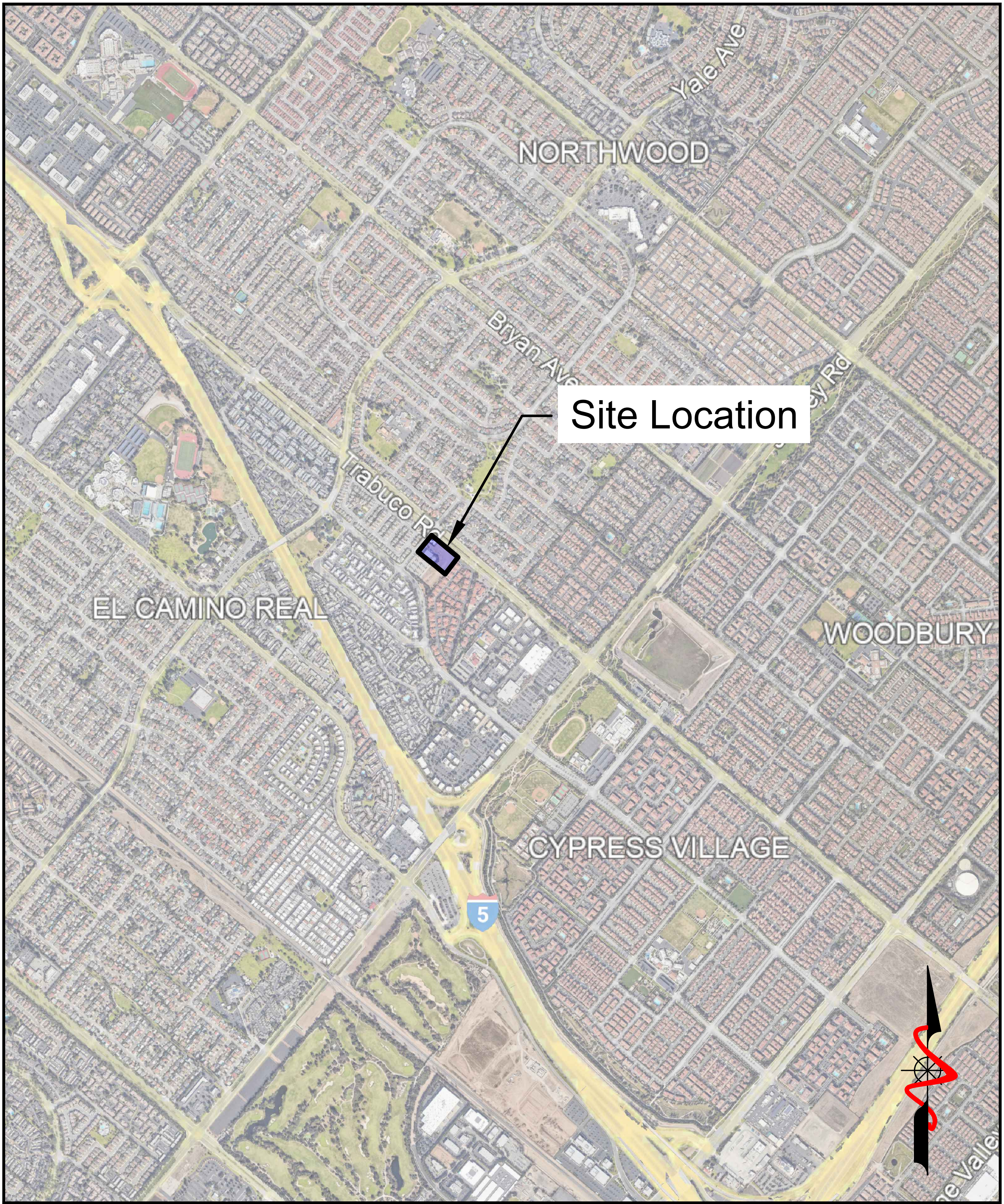
## *1.5 Laboratory Testing*

Laboratory testing was performed on representative soil samples obtained from our subsurface evaluation. Laboratory testing included in-situ moisture and density tests, fines content, expansion index, laboratory compaction, collapse and corrosion testing.

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 80 pounds per cubic foot (pcf) to 106 pcf, with an average of approximately 94 pcf. Field moisture contents ranged from approximately 1 percent to 24 percent, with an average of approximately 13 percent.
- Two samples were tested for fines content indicated a fines content (passing No. 200 sieve) ranging from 48 to 68 percent. According to the Unified Soils Classification System (USCS), the tested samples are classified as "coarse" and "fine-grained" soil.
- Two Expansion Index (EI) tests were performed. The result indicated an EI value ranging from 112 to 135, corresponding to "High" to "Very High" expansion potential.
- Two laboratory compaction tests of near surface samples indicated a maximum dry density ranging from 115.0 to 117.5 with an optimum moisture content ranging from 12.5 to 14.0 percent.
- Six swell/collapse tests were performed. The plots are provided in Appendix C.
- Corrosion testing indicated soluble sulfate content of 95 parts per million (ppm), chloride content of 60 ppm, and pH value of 7.90, and minimum resistivity of 720 ohm-cm.

A summary of the results is presented in Appendix C. The moisture and dry density test results are presented on the boring logs in Appendix B.




LGC Geotechnical, Inc.

**FIGURE 1**
**Site Location Map**

| | |
|---|---|
| PROJECT NAME | Our Lady of Peace, Irvine |
| PROJECT NO. | 23174-01 |
| ENG. / GEOL. | DJB |
| SCALE | Not to Scale |
| DATE | July 2024 |

# *2.0 <u>GEOTECHNICAL CONDITIONS</u>*

## *2.1 Regional Geology*

The subject site is located along the southeastern margin of the Los Angeles Basin, a large structural depression within the Peninsular Ranges geomorphic province of California. The southeastern portion of Los Angeles Basin includes the Tustin Plain, a shallow alluvial portion of the coastal plain. The site is located in the eastern portion of the Tustin Plain in the Irvine subbasin, generally composed of approximately 200 to 700 feet of unconsolidated to semi-consolidated Pliocene, Quaternary, and Holocene-age alluvial sediments. Soils within this zone consist predominantly of interbedded discontinuous lenses of clay, silt, sand and gravel.

Regional geologic mapping and local topographic expressions do not indicate the presence of large-scale landslides within or adjacent to the project area.

## *2.2 Site Geology and Generalized Subsurface Conditions*

Based on regional mapping (Morton, 2006) and our observations of the site, the subject site is underlain by Young Alluvial Fan Deposits (Qyf). In portions of the site, the young alluvial deposits are overlain by a thin veneer of older artificial fill that was placed during the previous development of the site. The depth of this older artificial fill was found to range from approximately 1 to 2 feet below existing grade during our evaluation but may be found at a deeper depth during construction, especially beneath the footprint of the existing church building. As indicated in our field explorations, site soils generally consisted of medium dense to dense clayey sands/silty sands and stiff to hard sandy clays.

It should be noted that borings are only representative of the location and time where/when they are performed, and varying subsurface conditions may exist outside of the performed locations. In addition, subsurface conditions can change over time. The soil descriptions provided should not be construed to mean that the subsurface profile is uniform, and that soil is homogenous within the project area. Descriptions of the subsurface conditions are presented on the boring and geotechnical trench logs located in Appendix B.

## *2.3 Groundwater*

Groundwater was encountered at a depth of approximately 54 feet below existing grade. Historic high groundwater is estimated to be greater than 40 feet below existing ground surface (CGS, 2001).

In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local seepage or during rainy seasons. Groundwater conditions below the site may be variable, depending on numerous factors including seasonal rainfall, local irrigation, and groundwater pumping, among others.

### *2.4 Faulting and Seismic Hazards*

Prompted by damaging earthquakes in California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. Their purpose was to prevent the construction of urban developments across the trace of active faults, resulting in the Alquist-Priolo Earthquake Fault Zoning Act. Earthquake Fault Zones have been delineated along the traces of active faults within California. Where developments for human occupation are proposed within these zones, the State requires detailed fault evaluations be performed so that engineering geologists can mitigate the hazards associated with active faulting by identifying the location of active faults and allowing for a setback from zones of previous ground rupture.

The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation. The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching, shallow ground rupture, soil liquefaction and dynamic settlement. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. Some of the major active nearby faults that could produce these secondary effects include the San Joaquin Hills, Whittier, Elsinore, Newport-Inglewood, among others (CGS, 2018). A discussion of these secondary effects is provided in the following sections.

#### *2.4.1 Liquefaction and Dynamic Settlement*

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that loose, saturated, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils, and cohesive soils exhibit low to negligible liquefaction potential. In general, cohesive soils are not considered susceptible to liquefaction. Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Furthermore, dynamic settlement of dry sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CDMG, 2001), the subject site is not located within a liquefaction hazard zone. Based on our evaluation, site soils are generally not susceptible to liquefaction due to the fine-grained nature of the on-site soils and low probability of groundwater in the upper 50 feet. Therefore, liquefaction potential is considered low.

### *2.4.2 Lateral Spreading*

Lateral spreading is a type of liquefaction induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move downslope towards a free face (such as a river channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Due to the site being relatively flat, low liquefaction potential and the lack of an adjacent "free face" to drive lateral spreading, the potential for lateral spreading is considered very low.

## *2.5 Seismic Design Criteria*

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2022 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the CBC. Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16. Representative site coordinates of latitude 33.700290 degrees north and longitude -117.768356 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations ($S_{MS}$ and $S_{M1}$) and adjusted design spectral response acceleration parameters ($S_{DS}$ and $S_{D1}$) for Site Class D are provided in Table 2 on the following page. Since site soils are Site Class D, additional adjustments are required to code acceleration response spectrums as outlined below and provided in ASCE 7-16. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

***TABLE 2***

***Seismic Design Parameters***

| **Selected Parameters from 2022 CBC, Section 1613 - Earthquake Loads** | **Seismic Design Values** | **Notes/Exceptions** |
|---|---|---|
| Distance to applicable faults classifies the site as a "Near-Fault" site. | | Section 11.4.1 of ASCE 7 |
| Site Class | D* | Chapter 20 of ASCE 7 |
| Ss (Risk-Targeted Spectral Acceleration for Short Periods) | 1.258g | From SEAOC, 2024 |
| $S_1$ (Risk-Targeted Spectral Accelerations for 1-Second Periods) | 0.450g | From SEAOC, 2024 |
| $F_a$ (per Table 1613.2.3(1)) | 1.0 | For Simplified Design Procedure of Section 12.14 of ASCE 7, $F_a$ shall be taken as 1.4 (Section 12.14.8.1) |
| $F_v$ (per Table 1613.2.3(2)) | 1.850 | Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7 |
| $S_{MS}$ for Site Class D [Note: $S_{MS} = F_aS_S$] | 1.258g | - |
| $S_{M1}$ for Site Class D [Note: $S_{M1} = F_vS_1$] | 0.833g | Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7 |
| $S_{DS}$ for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$] | 0.839g | - |
| $S_{D1}$ for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$] | 0.555g | Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7 |
| $C_{RS}$ (Mapped Risk Coefficient at 0.2 sec) | 0.940 | ASCE 7 Chapter 22 |
| $C_{R1}$ (Mapped Risk Coefficient at 1 sec) | 0.932 | ASCE 7 Chapter 22 |
| *Since site soils are Site Class D and S1 is greater than or equal to 0.2, the seismic response coefficient Cs is determined by Eq. 12.8-2 for values of T ≤ 1.5Ts and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for TL ≥ T > Ts, or Eq. 12.8-4 for T > TL. Refer to ASCE 7-16. Site Class F modified to Site Class D, seismic parameters only applicable for structure period ≤ 0.5 second, refer to discussion above. | | |

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.55 at a distance of approximately 14.75 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period (Design Earthquake) indicates that an earthquake magnitude of 6.56 at a distance of approximately 21.37 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2022 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean ($MCE_G$) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The $PGA_M$ for the site is equal to 0.576g (SEAOC, 2024). The design PGA is equal to 0.384g (2/3 of $PGA_M$).

## *2.6 __Oversized Material__*

Oversized materials (material larger than 8 inches in maximum dimension) are not anticipated to be encountered during site grading based on our subsurface evaluation. If encountered, recommendations are provided for appropriate handling of oversized materials in Appendix E.

## *2.7 __Expansion Potential__*

Based on the results of laboratory testing from our evaluation and from nearby sites, finished grade soils are anticipated to have a “High to Very High” expansion potential. Final expansion potential of site soils should be determined at the completion of grading.

## *2.8 __Soils Susceptible to Hydro-Collapse__*

Soils that are typically susceptible to hydro-collapse (or collapsible soils) are predominately sand and silt held in a loose honeycomb structure. This relatively loose honeycomb structure is held together by small amounts clay or calcium carbonate acting as a temporary (soluble) cementing agent. If the soil remains dry the soil maintains its structure, however the addition of water to the soil will greatly weaken the honeycomb structure and the soil can subsequently experience immediate collapses. This collapse results in rapid soil settlement and potential damage to any improvements which are located without the zone of influence of the collapsible soils.

Laboratory testing for hydro-collapse is typically performed per American Society for Testing and Materials (ASTM) Test Method D5333 or ASTM D 2435. The two most common categorizations of hydro-collapse potential are ASTM D5333 and the Naval Facilities Engineering Command (NFEC, 1986) are shown in Table 3A and 3B below. As indicated in Table 3B soils with collapse potential above 5 percent are considered “trouble”.

***TABLE 3A***

***Classification of Soil Collapsibility per ASTM D 5333***

| Collapse Index (Ie) | Collapse (%) | Collapse Potential |
|---|---|---|
| 0 | 0 | None |
| 0.001 - 0.02 | 0.1 - 2 | Slight |
| 0.021 - 0.60 | 2.1 - 6 | Moderate |
| 0.60 - 0.10 | 6 - 10 | Moderately Severe |
| > 0.10 | > 10 | Severe |

***TABLE 3B***

***Classification of Soil Collapsibility per NFEC, 1986***

| Collapse Potential (%) | Severity of Problem |
|---|---|
| 0-1 | No Problem |
| 1-5 | Moderate Trouble |
| 5-10 | Trouble |
| 10-20 | Severe Trouble |
| 20 | Very Severe Trouble |

A summary of the onsite soils collapse potential are provided in Table 4 on the following page. The measured collapse potential is up to 7.5 percent, with three samples indicating collapse potential of 5 percent or greater.

***TABLE 4***

***Summary of Hydro-Collapse Laboratory Test Results***

| Boring | Laboratory Measured Collapse (%) |
| --- | --- |
| HS-2 @ 10 feet | 7.5 |
| HS-2 @ 15 feet | 1.7 |
| HS-3 @ 10 feet | 6.2 |
| HS-5 @ 10 feet | 5.1 |

# *3.0 CONCLUSIONS*

Based on the results of our subsurface geotechnical evaluation, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided that the recommendations contained in the following sections are incorporated during site development. A summary of our geotechnical conclusions are as follows:

- In general, borings excavated at the subject site indicate primarily native soils consisting of clayey sand, silty sand and sandy clay to the maximum explored depth of approximately 70 feet below existing grade. Native alluvial materials were found locally overlain by a relatively thin veneer of old artificial fill associated with previous development of the site. The near-surface loose and compressible soils are not suitable for the planned improvements in their present condition (refer to Section 4.1).
- Groundwater was encountered at a depth of approximately 54 feet below existing grade. Historic high groundwater is estimated to be greater than 40 feet below existing ground surface (CGS, 2001).
- The subject study area is not located within a mapped State of California Earthquake Fault Zone, and based upon our review of published geologic mapping, no known active or potentially active faults are known to exist within or in the immediate vicinity of the site. Therefore, the potential for ground rupture as a result of faulting is considered very low.
- The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- The site is not located in a mapped liquefaction hazard zone. Based on our evaluation, site soils are generally not susceptible to liquefaction due to the fine-grained nature of the on-site soils and low probability of groundwater in the upper 50 feet. Therefore, liquefaction potential is considered low.
- Based on the results of preliminary laboratory testing, site soils are anticipated to have "High to Very High" expansion potential. Final design expansion potential must be determined at the completion of grading.
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order. We anticipate that soils generated from the excavations will be generally suitable for re-use as compacted fill, provided they are relatively free of rocks larger than 8 inches in dimension, construction debris, and significant organic material.
- Oversized materials (greater than 8 inches in maximum dimension) are not likely to be encountered during site grading. If encountered, recommendations are provided for appropriate handling of oversized materials in Appendix E.
- Due to their high clay content and expansion potential, the onsite soils are not suitable for backfill of site retaining/basement walls. Therefore, import of sandy soils meeting project recommendations will be required.
- Field testing resulted in a measured infiltration rate ranging from 0.01 to 0.02 inches per hour. The measured infiltration rates do not include a factor of safety. Discussion regarding infiltration is provided in Section 4.8.
- Some of the site soils are susceptible to hydro-collapse. To mitigate this condition, the soils most susceptible to collapse should be temporarily removed and recompacted as fill during the grading operations.

- Pre-soaking of the subgrade for building slabs (and flatwork) will be required due to site expansive soils. The duration and process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 2 to 3 weeks).

# *4.0 RECOMMENDATIONS*

The following recommendations are to be considered preliminary and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the owner.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2022 CBC requirements. With regard to the possible occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an "acceptable level." The "acceptable level" of risk is defined by the California Code of Regulations as "that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project" [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvement may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions but cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

The geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual as-graded conditions.

## *4.1 Site Earthwork*

Rough grading shall include remedial earthwork grading and placement of engineered compacted fill to design grades. Geotechnical recommendations for precise grading and construction of the proposed new improvements will be provided, as necessary.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the 2022 CBC/City of Irvine requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede those included in Appendix E. The following recommendations may be revised within future grading plan review reports or based on the actual conditions encountered during site grading.

### *4.1.1 Site Preparation*

Prior to grading, areas to be developed should undergo complete demolition, removal of all vegetation, and clearing of pavements, existing utilities, foundation and slab elements from the site. Vegetation, debris from previous land use and excessive organic material should be removed and properly disposed of offsite. Holes resulting from removals of buried obstructions, which extend below proposed remedial and/or finish grades, should be replaced with suitable compacted fill material. If the demolition contractor removes subsurface utilities below the proposed remedial grading depth we recommend the excavations either be left open until grading operations begin or properly compacted to the depth of remedial grading.

If cesspools or septic systems are encountered, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. Any encountered wells should be properly abandoned in accordance with regulatory requirements.

### 4.1.2 Removal Depths and Limits

In order to provide a relatively uniform bearing condition for the planned improvements, we recommend the near-surface potentially compressible soils and soils highly susceptible to collapse be temporarily removed and recompacted as engineered fill. If any undocumented artificial fill is encountered within the influence of the proposed building pads, the soils should be removed to competent native materials.

In order to promote more uniform soil conditions, soils shall be temporarily removed and recompacted to a minimum depth of approximately 12 feet below existing grade or 3 feet below the bottom of proposed foundations, whichever is deeper. Additionally, existing undocumented fill and unsuitable topsoil encountered within the building footprints should be temporarily removed and recompacted as compacted fill. Where space is available, the envelope for removal and recompaction should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade or 5 feet beyond the edges of the proposed building improvements, whichever is larger.

Local conditions may be encountered during excavation that could require additional over-excavation beyond the above noted minimum in order to obtain an acceptable subgrade. The actual depths and lateral extents of grading will be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. Removal areas and areas to be over-excavated should be accurately staked in the field by the Project Surveyor.

### 4.1.3 Temporary Excavations

Temporary excavations should be performed in accordance with project plans, specifications, and applicable Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. Based on our field investigation, the majority of site soils are anticipated to be OSHA Type “B” soils (refer to the attached boring logs). Isolated lenses of sandy soils are present and should be considered susceptible to caving. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Where proposed improvements will be adjacent to property lines, the potential for impacting existing offsite improvements may be reduced by performing "ABC" slot cuts while performing earthwork removal and recompaction. "ABC" slot cuts are defined as excavations perpendicular to sensitive property boundaries that are divided into multiple "slots" of equal width. If slots are labeled A, B, C, A, B, C, etc., then all "A" slots can be excavated at the same time but must be backfilled before all "B" slots can be excavated, etc. Any given slot should be backfilled immediately with properly compacted fill to finish grade prior to excavation of the adjacent two slots. Please note some isolated sand lenses which are potentially susceptible to caving are present at the site. Recommendations for slot cut dimensions should be evaluated during grading. Protection of the existing offsite improvements during grading is the responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a minimum distance equivalent to a 1:1(horizontal to vertical) projection from the bottom of the excavation or 5 feet, whichever is greater. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

It should be noted that any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation. If requested, temporary shoring parameters will be provided.

#### *4.1.4 <u>Removal Bottoms and Subgrade Preparation</u>*

In general, removal bottoms, over-excavation bottoms and areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition (generally within optimum and 2 percent above optimum moisture content), and re-compacted per project recommendations.

Removal bottoms, over-excavation bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement.

#### *4.1.5 <u>Material for Fill</u>*

From a geotechnical perspective, the onsite soils are generally considered suitable for use as general compacted fill, provided they are screened of significant organic materials, construction debris and any oversized material (8 inches in greatest dimension).

From a geotechnical viewpoint, import soils for general fill (i.e., non-retaining/basement wall backfill) should consist of clean, granular soils of Medium expansion potential (expansion index 90 or less based on ASTM D4829). Import for retaining/basement wall backfill should meet the criteria outlined in the paragraph below. Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of three working days prior to any planned importation.

Retaining/basement wall backfill should consist of imported sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per ASTM Test Method D1140 (or ASTM D6913/D422) and a “Very Low” expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches in maximum dimension.

Aggregate base should conform to the requirements of Section 200-2 of the most recent version of the Standard Specifications for Public Works Construction (“Greenbook”) for untreated base materials and/or City of Irvine requirements.

The placement of demolition materials in compacted fill is acceptable from a geotechnical viewpoint provided the demolition material is broken up into pieces not larger than typically used for aggregate base (approximately 2 to 4 inches in maximum dimension) and well blended into fill soils with essentially not resulting voids. Demolition material placed in fills must be free of construction debris (wood, organics, etc.) and reinforcing steel. If you elect to incorporate asphalt concrete fragments into the fill materials, approval from an environmental viewpoint and/or local agency may be required and is not the purview of the geotechnical consultant. From our previous experience, we recommend that asphalt concrete fragments be exported or limited to fill areas within planned street or parking areas (i.e., not within building pad areas).

### *4.1.6 Placement and Compaction of Fills*

Material to be placed as fill should be brought to near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM D1557). Moisture conditioning of site soils will be required in order to achieve adequate compaction. Drying and/or mixing the very moist soils will be required prior to reusing the materials in compacted fills. Soils are also present that will require additional moisture in order to achieve the required compaction.

The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in compacted thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing by LGC Geotechnical. Oversized material as previously defined should be removed from site fills.

During backfill of excavations, the fill should be properly benched into firm and competent soils of temporary backcut slopes as it is placed in lifts.

Aggregate base material should be compacted to a minimum of 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction per ASTM D1557 at near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content).

#### *4.1.7 <u>Trench and Retaining/Basement Wall Backfill and Compaction</u>*

The onsite materials may generally be suitable as trench backfill provided the soils are screened of organic matter and rocks greater than 6 inches in diameter. Trench backfill should be compacted in uniform lifts (generally not exceeding 12 inches in compacted thickness) by mechanical means to at least 90 percent relative compaction (per ASTM Test Method D1557). A representative from LGC Geotechnical should observe and test the backfill to verify compliance with the project recommendations.

Some manufacturers have restrictions for the type of soil in contact with pipes. Manufacturer specifications for soils surrounding the pipe should be reviewed and approved by the pipe designer for use prior to construction. The use of open-graded rock as backfill must include means of reducing migration of fines of adjacent materials into the void spaces. This can be done by entirely wrapping the open-graded rock in a filter fabric or introducing an excavatable flowable fill to the rock.

Compactive efforts must be made for all backfill soil types except for flowable fill. There must be room between the trench side walls and the springline of the pipe for mechanical compaction equipment to get adequate compaction of the soil under the haunches of the pipe. If mechanical compaction may result in damage to pipes, clean sand having a Sand Equivalent (SE) greater than or equal to 20 (per California Test Method [CTM] 217) can be used to shade the pipes. Sand backfill should be densified by jetting or flooding and then tamped to ensure adequate compaction. Sand grains should be from a natural source with rounded shape. Manufactured sand from crushed rock or recycled material is not suitable for jetting/flooding as the grains are typically angular in shape and do not densify well enough with these methods.

Retaining/basement wall backfill should consist of approved imported sandy soils as outlined in preceding Section 4.1.5. The onsite soils are not suitable for backfill of site retaining/basement walls based on their fines content and expansion potential. Therefore, import of sandy soils meeting project recommendations will be required. For the lateral limits of select sandy backfill refer to Figures 3 and 4. Retaining/basement wall backfill soils should be compacted in relatively uniform thin lifts to at least 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining/basement wall backfill materials should not be permitted.

A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project recommendations.

#### *4.1.8 Shrinkage and Subsidence*

Allowance in the earthwork volumes budget should be made for an estimated 5 to 15 percent reduction in volume of near-surface (upper approximate 12 feet) soils. It should be stressed that these values are only estimates and that an actual shrinkage factor would be extremely difficult to predetermine. The effective change in volume of onsite soils will depend primarily on the type of compaction equipment, method of compaction used onsite by the contractor, and accuracy of the topographic survey.

Due to the combined variability in topographic surveys, inability to precisely model the removals and variability of on-site near-surface conditions, it is our opinion that the site will not balance at the end of grading. If importing/exporting a large volume of soils is not considered feasible or economical, we recommend a balance area be designated onsite that can fluctuate up or down based on the actual volume of soil. We recommend a "balance" area that can accommodate on the order of 5 percent (plus or minus) of the total grading volume be considered.

### *4.2 Preliminary Foundation Recommendations*

Provided that the remedial grading recommendations provided herein are implemented, the site may be considered suitable for the support of the proposed structures using either a post-tensioned foundation or alternative rigid foundation system designed to resist the impacts of expansive soils. Site soils are anticipated to be of High to Very High expansion potential (EI of greater than 90 per ASTM D4829). This must be verified based on as-graded conditions. Please note that the following foundation recommendations are <u>preliminary</u> and must be confirmed by LGC Geotechnical at the completion of project plans (i.e., foundation, grading and site layout plans) as well as completion of earthwork. Recommended soil bearing and estimated static settlement are provided in Section 4.3.

#### *4.2.1 Provisional Post-Tensioned Foundation Design Parameters*

The geotechnical parameters provided herein may be used for post-tensioned slab foundations with a deepened perimeter footing or a post-tensioned mat slab. These parameters have been determined in general accordance with the Post-Tensioning Institute (PTI) Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, referenced in Chapter 18 of the 2022 CBC. In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural designer/architect. Other types of stiff slabs may be used in place of the CBC post-tensioned slab design provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the CBC/PTI method.

Our design parameters are based on our experience with similar projects, test results onsite, and the anticipated nature of the soil (with respect to expansion potential).

Please note that implementation of our recommendations will not eliminate foundation movement (and related distress) should the moisture content of the subgrade soils fluctuate. It is the intent of these recommendations to help maintain the integrity of the proposed structures and reduce (not eliminate) movement, based upon the anticipated site soil conditions. Should future owners and/or property maintenance personnel not properly maintain the areas surrounding the foundation, for example by overwatering, then we anticipate for highly expansive soils the maximum differential movement of the perimeter of the foundation to the center of the foundation to be on the order of a couple of inches. Soils of lower expansion potential are anticipated to show less movement.

***TABLE 5***

***Preliminary Post-Tensioned Foundation Design Parameters***

| **Parameter** | **PT Slab with Perimeter Footing** | **PT Mat with Thicken ed Edge** |
|---|---|---|
| Expansion Potential | Very High | Very High |
| Thornthwaite Moisture Index | -20 | -20 |
| Constant Soil Suction | PF 3.9 | PF 3.9 |
| Center Lift<br>Edge moisture variation distance, $e_m$<br>Center lift, $y_m$ | <br>6.1 feet<br>1.0 inch | <br>6.1 feet<br>1.2 inch |
| Edge Lift<br>Edge moisture variation distance, $e_m$<br>Edge lift, $y_m$ | <br>3.4 feet<br>2.5 inches | <br>3.4 feet<br>2.5 inches |
| Modulus of Subgrade Reaction, k (assuming presoaking as indicated below) | 100 pci | 100 pci |
| Minimum perimeter footing/thickened edge embedment below finish grade | 30 inches | 6 inches |
| Minimum Slab Thickness | 6 inches[2] | 10 inches[2] |
| Presoak | 140% of Opt.<br>30 inches | 140% of Opt.<br>30 inches |

1. Expansion index is for preliminary design purposes. Further evaluation is needed at the completion of grading.
2. Recommendations for foundation reinforcement and slab thickness are ultimately the purview of the foundation engineer/structural engineer based upon geotechnical criteria and structural engineering considerations.
3. Recommendations for vapor retarders below slabs are also the purview of the foundation engineer/structural engineer and should be provided in accordance with applicable code requirements.

#### *4.2.2 Provisional Conventional Foundation Design Parameters*

Conventional foundations may be designed in accordance with the Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2022 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 45
- Climatic Rating: Cw = 15
- Reinforcement: Per structural designer
- Minimum Footing Depth: 30 inches below lowest adjacent grade.
- Moisture-condition (presoak) slab subgrade to 140% of optimum moisture content to a minimum depth of 30 inches prior to trenching.

The recommended moisture content should be maintained up to the time of concrete placement.

#### *4.2.3 Shallow Foundation Maintenance*

The geotechnical parameters provided herein assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage and adequately maintained so that ponding, which causes significant moisture changes below the foundation, does not occur. Our recommendations do not account for excessive irrigation and/or incorrect landscape design. Plants should only be provided with sufficient irrigation for life and not overwatered to saturate subgrade soils. Sunken planters placed adjacent to the foundation should either be designed with an efficient drainage system or liners to prevent moisture infiltration below the foundation or into the basement. Some lifting of the perimeter foundation beam should be expected even with properly constructed planters.

In addition to the factors mentioned above, future owners/property management personnel should be made aware of the potential negative influences of trees and/or other large vegetation. Roots that extend near the vicinity of foundations can cause distress to foundations. Future owners (and the owner's landscape architect) should not plant trees/large shrubs closer to the foundations than a distance equal to half the mature height of the tree or 20 feet, whichever is more conservative unless specifically provided with root barriers to prevent root growth below the building foundation.

It is the owner's responsibility to perform periodic maintenance during hot and dry periods to ensure that adequate watering has been provided to keep soil from separating or pulling back from the foundation. Future owners and property management personnel should be informed and educated regarding the importance of maintaining a constant level of soil-moisture. The owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing potentially expansive soils to become too dry. Expansive soils can undergo shrinkage during drying, and swelling during the rainy winter season, or when irrigation is resumed. This can result in distress to building structures and hardscape improvements. The builder should provide these recommendations to future owners and property management personnel.

#### *4.2.4 Slab Underlayment Guidelines*

The following is for informational purposes only since slab underlayment (e.g., moisture retarder, sand or gravel layers for concrete curing and/or capillary break) is unrelated to the geotechnical performance of the foundation and thereby not the purview of the geotechnical consultant. Post-construction moisture migration should be expected below the foundation. The foundation engineer/architect should determine whether the use of a capillary break (sand or gravel layer), in conjunction with the vapor retarder, is necessary or required by code. Sand layer thickness and location (above and/or below vapor retarder) should also be determined by the foundation engineer/architect.

### *4.3 Soil Bearing and Lateral Resistance*

Subterranean Basement at least 9 feet below Grade: Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 3,000 pounds per square foot (psf) may be used for the design of footings having a minimum width of 18 inches and minimum embedment of 18 inches below the lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment and 150 psf for each additional foot of foundation width to a maximum of 4,000 psf.

At-Grade Structures: Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment and 150 psf for each additional foot of foundation width to a maximum value of 2,500 psf.

A mat foundation a minimum of 6 inches below lowest adjacent grade may be designed for an allowable soil bearing pressure of 1,200 psf. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated are for total dead loads and frequently applied live loads and may be increased by ⅓ for short duration loading (i.e., wind or seismic loads).

In utilizing the above-mentioned allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be 1-inch or less. Differential settlement may be taken as half of the total settlement (i.e., ½-inch over a horizontal span of 40 feet).

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.25 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 200 psf per foot of depth (or pcf) to a maximum of 2,000 psf may be used for the sides of footings poured against properly compacted fill. Allowable passive pressure may be increased to 265 pcf (maximum of 2,650 psf) for short duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively. The structural designer should incorporate appropriate factors of safety and/or load factors in their design.

### 4.4 *Lateral Earth Pressures and Retaining/Basement Wall Design Considerations*

The following may be used for design of site retaining/basement walls. It is our understanding that basement walls up to approximately 9 feet are proposed beneath the church building. Lateral earth pressures are provided as equivalent fluid unit weights, in psf per foot of depth (or pcf). These values do not contain an appreciable factor of safety, so the retaining/basement wall designer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

The following lateral earth pressures are presented in Table 6 below, for approved imported free draining, clean granular (sandy) soils with a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D-421/422) and a "Very Low" expansion potential (EI of 20 or less per ASTM D4829). The site soils are not suitable for retaining/basement wall backfill due to their fines content and expansion index. Import of soils meeting the criteria outlined above will need to be used for retaining/basement wall backfill soil. The wall designer should clearly indicate on the retaining/basement wall plans the required imported sandy soil backfill criteria. These preliminary findings should be confirmed during grading.

***TABLE 6***

***Lateral Earth Pressures – Approved Imported Sandy Soils***

| **Conditions** | **Equivalent Fluid Unit Weight (pcf)** | **Equivalent Fluid Unit Weight (pcf)** |
|---|---|---|
| | **Level Backfill** | **2:1 Sloped Backfill** |
| | **Approved Sandy Soils** | **Approved Sandy Soils** |
| Active | 35 | 55 |
| At-Rest | 55 | 70 |

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. This would include basement walls and 90-degree corners of retaining walls. Such walls should be designed for "at-rest." The equivalent fluid pressure values assume free-draining conditions and a drainage system will be installed and maintained to prevent the build-up of hydrostatic pressures. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Retaining/basement wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, subsurface water issues for soils in front of the retaining or potential water intrusion into the basement, the perforated subdrain pipe should be located as low as possible behind the retaining/basement wall. The outlet pipe should be sloped to drain to a suitable outlet. In general, we do not recommend retaining/basement wall outlet pipes be connected to area drains. If subdrains are connected to area drains, special care should be taken to maintain these drains. Typical conventional retaining/basement wall drainage is shown on Figures 3 and 4, respectively. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Waterproofing and subdrain outlet systems are not the purview of the geotechnical consultant.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining/basement wall designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed retaining/basement wall footing will surcharge the proposed retaining/basement wall. In addition to the recommended earth pressure, retaining/basement walls adjacent to streets should be designed to resist a uniform lateral pressure of 85 pounds per square foot (psf) due to normal street vehicle traffic, if applicable. Uniform lateral surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.45 and 0.3 may be used for at-rest and active conditions, respectively. The retaining/basement wall designer should contact the geotechnical consultant for any required geotechnical input in estimating surcharge loads.

The retaining wall/basement wall designer may use a seismic lateral earth pressure increment of 10 pcf. This increment should be applied in addition to the provide static lateral earth pressure using a triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). Per Section 1803.5.12 of the 2022 CBC, the seismic lateral earth pressure is applicable to structures assigned to Seismic Design Category D through F for retaining wall structures supporting more than 6 feet of backfill height. This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California and the Naval Civil Engineering Laboratory (Lew, et al, 2010 & NCEL, 1993).

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.3. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining/basement walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

### *4.5 <u>Corrosivity to Concrete and Metal</u>*

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils to buried concrete and metal facilities. We therefore present the results of our testing with regard to corrosion for the use of the client and other consultants, as they determine necessary.

Corrosion testing of near-surface bulk samples indicated a soluble sulfate content value of 95 ppm (less than 0.01 percent), a chloride content of 60 ppm, pH of 7.90, and a minimum resistivity of 720 ohm-centimeters. Based on Caltrans Corrosion Guidelines (2021), soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 2,000 ppm (0.2 percent) or greater. Based on the test results, soils are not considered corrosive using Caltrans criteria. Note that based on minimum resistivity the soils are considered severely corrosive to metallic improvements. If improvements that may be susceptible to corrosion are proposed, it is recommended that further evaluation by a corrosion engineer be performed.

Based on our laboratory test results of representative site soil samples and our experience, onsite soils should be considered as having a severity categorization of "moderate" and are designated class "S1" per ACI 318, Table 19.3.1.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, Table 19.3.2.1 using the "S1" sulfate classification.

Laboratory testing may need to be performed at the completion of grading by the project corrosion engineer to further evaluate the as-graded soil corrosivity characteristics. Accordingly, revision of the corrosion potential may be needed, should future test results differ substantially from the conditions reported herein. The client and/or other members of the development team should consider this during the design and planning phase of the project and formulate an appropriate course of action.

### *4.6 <u>Preliminary Asphalt Concrete Pavement Sections</u>*

For the purposes of these preliminary recommendations, we have selected a preliminary design R-value of 5 and calculated pavement sections for Traffic Indices of 5.0, 5.5 and 6.0. R-value testing of the street subgrade will need to be performed to confirm our preliminary testing results/assumptions once the streets have been graded to finish subgrade elevations and the final Traffic Index is determined by the Civil Engineer or City Engineer. We are not responsible for selecting a design Traffic Index. It is our understanding that the City of Irvine requires a minimum pavement section of 4.2 inches of asphalt over 6 inches of base.

***<u>TABLE 7</u>***

***<u>Preliminary Pavement Sections</u>***

| **Assumed Traffic Index** | 5.0 | 5.5 | 6.0 |
|---|---|---|---|
| **R -Value Subgrade** | 5 | 5 | 5 |
| **AC Thickness** | 4.2 | 4.2 inches | 4.2 inches |
| **AB Thickness** | 8.0 inches | 10.0 inches | 12.5 inches |

Due to anticipated construction traffic prior to the completion of the project, we recommend that the total thickness (base course and capping course) of AC be placed at essentially the same time. Construction traffic loading on only the base course of the AC will increase the potential for pavement distress. It should be noted that construction traffic such as concrete trucks will likely exceed traffic loading after completion of construction.

Increasing the thickness of asphalt or adding additional base material will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Earthwork recommendations regarding aggregate base and subgrade are provided in the previous Section "Site Earthwork" and the related sub-sections of this report.

### *4.7 Nonstructural Concrete Flatwork*

Nonstructural concrete flatwork (such as walkways, private drives, patio slabs, etc.) has a potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete may be designed in accordance with the minimum guidelines outlined in Table 8 on the following page. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

***TABLE 8***

***Nonstructural Concrete Flatwork Guidelines***

| | Sidewalks (≤6 feet wide) | Patios/Entryways / Walkways (adjacent to Buildings or flatwork >6 feet wide) | City Sidewalk Curb and Gutters |
|---|---|---|---|
| Minimum Thickness (in.) | 4 (nominal) | 5 (full) | City/Agency Standard |
| Presaturation | 140% of optimum m/c to 12 inches | 140% of optimum m/c to 24 inches | City/Agency Standard |
| Reinforcement | — | No. 3 at 24 inches on centers | City/Agency Standard |
| Thickened Edge (in.) | — | — | City/Agency Standard |
| Crack Control Joints | Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness | Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness | City/Agency Standard |
| Maximum Joint Spacing | 5 feet | 6 feet | City/Agency Standard |

## *4.8 Subsurface Water Infiltration*

Recent regulatory changes have occurred that mandate that storm water be infiltrated below grade into subsurface soils rather than collected in a conventional storm drain system. Typically, a combination of methods are implemented to reduce surface water runoff and increase infiltration including; permeable pavements/pavers for roadways and walkways, directing surface water runoff to grass-lined swales, retention areas, and/or drywells, etc.

It should be noted that collecting and concentrating surface water for the purpose of intentionally infiltrating below grade, conflicts with the geotechnical engineering objective of directing surface water away from slopes, structures and other improvements. The geotechnical stability and integrity of a site is reliant upon appropriately handling surface water. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement.

The results of our field infiltration testing indicate the observed 1-D infiltration rate for I-1 and I-2 (not including required factors of safety for design) was 0.01 to 0.02 inches per hour. The design infiltration rate is thereby equal to the Observed Infiltration Rate provided in Table 1 (inches per hour) divided by the design factor of safety. The design factor of safety must be a minimum of 2.0 but may be increased at the discretion of the design engineer (County of Orange, 2013). Per the County of Orange infiltration guidelines (2013), infiltration of stormwater is not required when the factored infiltration rate (measured infiltration rate with safety factors applied) is less than 0.3 inches per hour in the vicinity of the BMPs.

Based on the results of field infiltration testing indicating essentially zero infiltration and soils with high fines content (silts and clays), we strongly recommend against the intentional infiltration of stormwater into the subsurface soils.

### *4.9 Control of Surface Water and Drainage Control*

From a geotechnical perspective, we recommend that compacted finished grade soils adjacent to proposed structures be sloped away from the proposed structures and towards an approved drainage device or unobstructed swale. Where possible, we recommend concrete or asphalt be placed around the outer 5-10 feet of the buildings. Drainage swales, wherever feasible, should not be constructed within 5 feet of buildings. Where lot and building geometry necessitates that drainage swales be routed closer than 5 feet to structural foundations, we recommend the use of area drains together with drainage swales. Drainage swales used in conjunction with area drains should be designed by the project civil engineer so that a properly constructed and maintained system will prevent ponding within 5 feet of the foundation. Code compliance of grades is not the purview of the geotechnical consultant.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Overwatering must be avoided.

### *4.10 Geotechnical Plan Review*

Project plans (grading, foundation, retaining/basement wall, etc.) should be reviewed by this office prior to construction to verify that our geotechnical recommendations have been incorporated. Additional or modified geotechnical recommendations may be required based on the proposed layout.

### *4.11 Geotechnical Observation and Testing*

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC Geotechnical. Geotechnical observation and testing is required per Section 1705 of the 2022 California Building Code (CBC).

Geotechnical observation and/or testing should be performed by LGC Geotechnical at the following stages:

- During grading (removal bottoms, fill placement, etc.);
- During retaining/basement wall backfill and compaction;
- During utility trench backfill and compaction;
- After presoaking building pad and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- Preparation of pavement subgrade and placement of aggregate base;
- After building and wall footing excavation and prior to placement of steel reinforcement and/or concrete; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

## *5.0 LIMITATIONS*

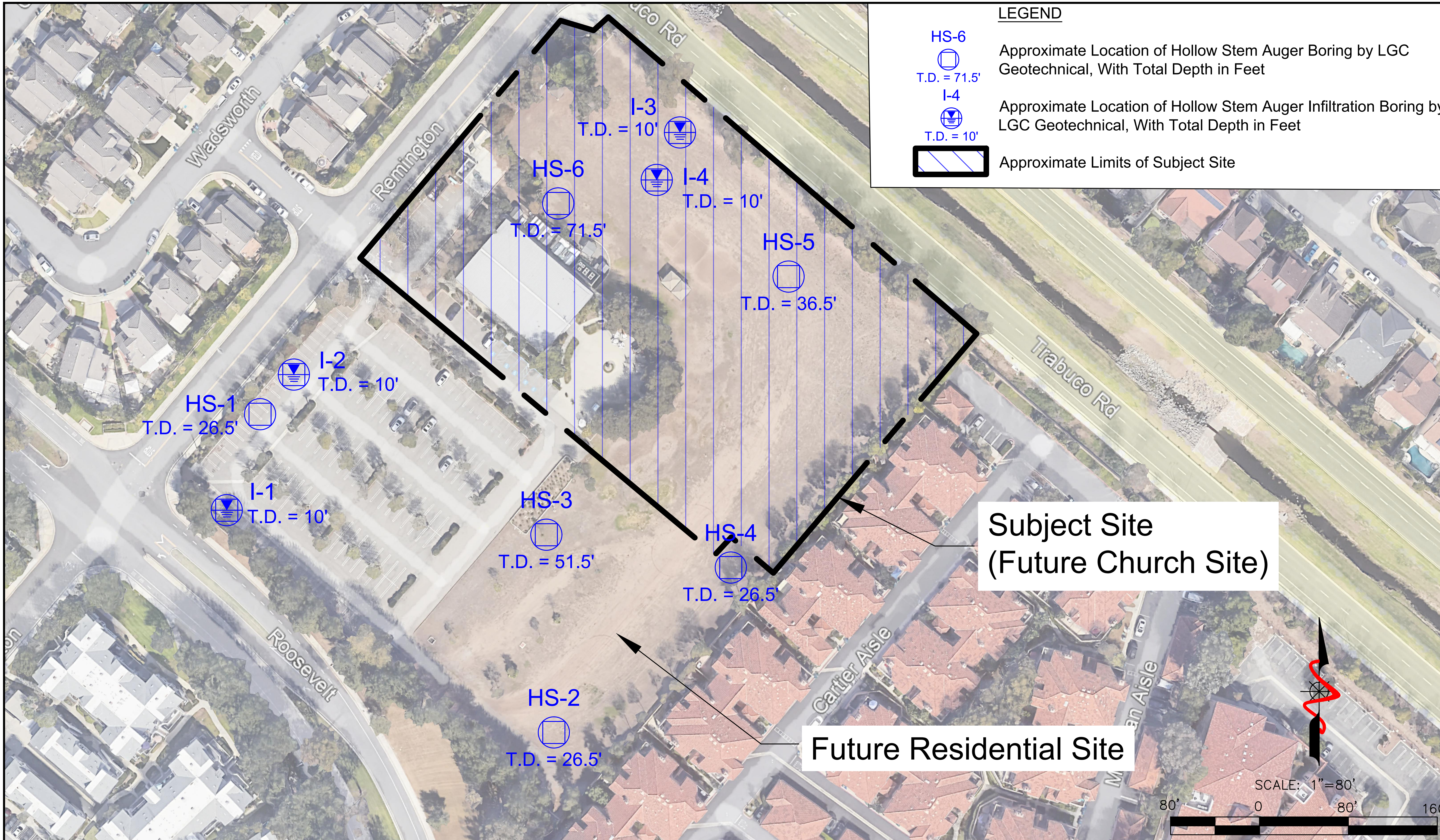
Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is based on data obtained from limited observations of the site, which have been extrapolated to characterize the site. While the scope of services performed is considered suitable to adequately characterize the site geotechnical conditions relative to the proposed development, no practical evaluation can completely eliminate uncertainty regarding the anticipated geotechnical conditions in connection with a subject site. Variations may exist and conditions not observed or described in this report may be encountered during grading and construction.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants (at a minimum the civil engineer, structural engineer, landscape architect) and incorporated into their plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

The findings of this report are valid as of the present date. However, changes in the conditions of a site can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. The findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site. This report is intended exclusively for use by the client, any use of or reliance on this report by a third party shall be at such party's sole risk.

In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and modification.




LGC Geotechnical, Inc.

**LGC Geotechnical, Inc.**
**131 Calle Iglesia, Ste. 200**
**San Clemente, CA 92672**
**TEL (949) 369-6141 FAX (949) 369-6142**

**FIGURE 2**
**Boring Location Map**

| | |
|---|---|
| PROJECT NAME | Our Lady of Peace, Irvine |
| PROJECT NO. | 23174-01 |
| ENG. / GEOL. | DJB / BPG |
| SCALE | 1" = 80' |
| DATE | July 2024 |

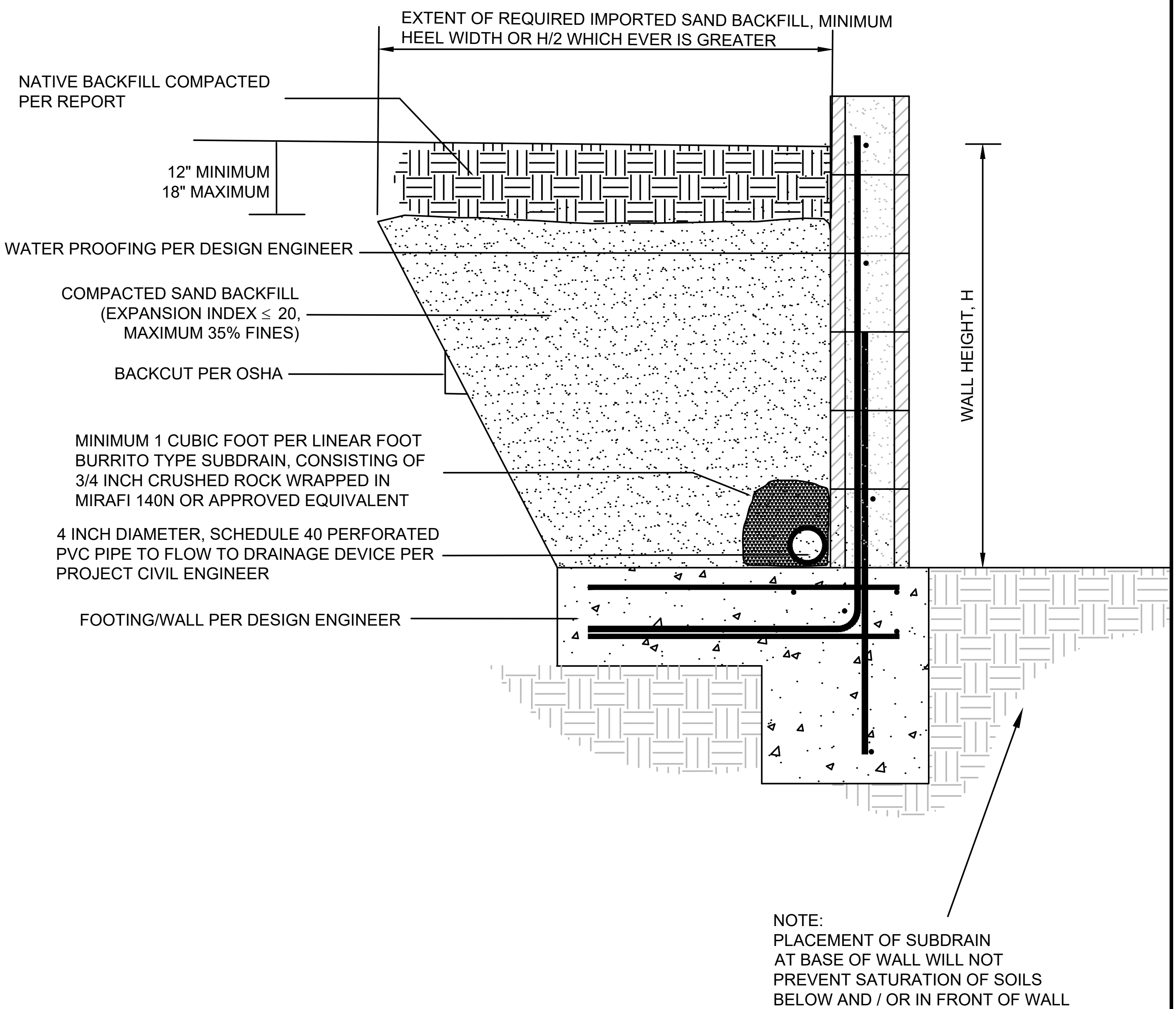



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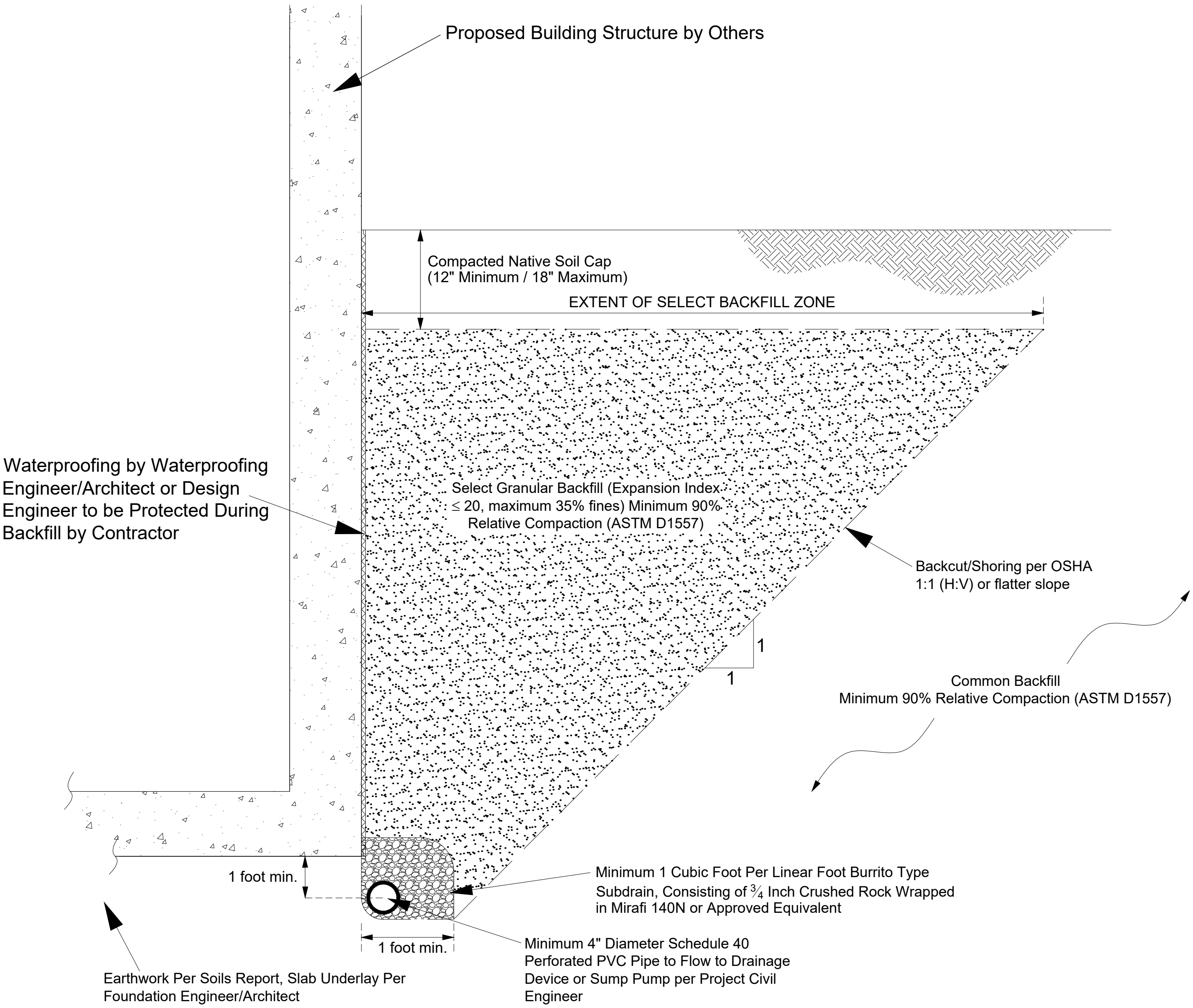
**FIGURE 3**
***Retaining Wall Backfill Detail***

| | |
|---|---|
| PROJECT NAME | Our Lady of Peace, Irvine |
| PROJECT NO. | 23174-01 |
| ENG. / GEOL. | DJB |
| SCALE | Not to Scale |
| DATE | July 2024 |




NOTE : Granular backfill may be substituted with 3/4 inch crushed rock wrapped in filter fabric. Crushed rock must be compacted in lifts. Geotechnical drainage detail will not prevent water seepage into structure, it is only to prevent build up of water pressure against wall.

LGC Geotechnical, Inc.

**FIGURE 4**

**Basement Subterranean Structure Backfill/Subdrain Detail**

| | |
|---|---|
| PROJECT NAME | Our Lady of Peace, Irvine |
| PROJECT NO. | 23174-01 |
| ENG. / GEOL. | DJB / BPG |
| SCALE | Not to Scale |
| DATE | July 2024 |

# *Appendix A*
# *References*

## *APPENDIX A*

## ***References***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# *Appendix B*
# *Boring Logs*

# Geotechnical Boring Log Borehole HS-5

| | |
|---|---|
| **Date:** 2/23/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 6" |
| **Elevation of Top of Hole:** 161' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 1 of 2 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| 160 | 0 | | | | | | | @0' - Grass | |
| | | B-1 | | | | | | **Young Axial Channel Deposits (Qya):** | |
| | | | R-1 | 4 13 31 | 100.4 | 20.9 | CL | @ 2.5' - Sandy CLAY: dark brown, very moist, hard, traces of roots | |
| 155 | 5 | | R-2 | 11 11 11 | 79.5 | 9.3 | SM | @ 5' - Silty SAND: pale brown, moist, medium dense | |
| | | | R-3 | 13 21 24 | 118.6 | 9.8 | SC | @ 7.5' - Clayey SAND: brown, moist, dense | |
| 150 | 10 | | R-4 | 13 12 10 | 81.6 | 9.8 | CL-ML | @ 10' - Silty CLAY: olive brown, slightly moist, very stiff | CO |
| 145 | 15 | | SPT-1 | 4 4 4 | | 7.9 | SM | @ 15' - Silty SAND: brown, moist, medium dense | |
| 140 | 20 | | R-5 | 8 17 18 | 120.1 | 11.8 | SC | @ 20' - Clayey SAND: dark brown, moist, medium dense | |
| 135 | 25 | | SPT-2 | 3 5 7 | | 28.0 | CL | @ 25' - Sandy CLAY: dark brown, very moist, very stiff | |
| | 30 | | | | | | | | |

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

**SAMPLE TYPES:**
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

**TEST TYPES:**
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 10/20/2022

# Geotechnical Boring Log Borehole HS-5

| | |
|---|---|
| **Date:** 2/23/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 6" |
| **Elevation of Top of Hole:** ~161' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 2 of 2 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| 130 | 30 | | R-6 | 8 13 18 | 108.8 | 5.4 | SC | @ 30' - Clayey SAND: brown, slightly moist, medium dense | |
| 125 | 35 | | SPT-3 | 4 6 7 | | 12.1 | | @ 35' - Clayey SAND: brown, moist, medium dense | |
| 120 | 40 | | | | | | | Total Depth = 36.5'<br>No Groundwater Encountered<br>Backfilled with Cuttings on 2/23/24 | |
| 115 | 45 | | | | | | | | |
| 110 | 50 | | | | | | | | |
| 105 | 55 | | | | | | | | |
| | 60 | | | | | | | | |

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- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

# Geotechnical Boring Log Borehole HS-6

| | |
|---|---|
| **Date:** 2/23/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 6" |
| **Elevation of Top of Hole:** 158' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 1 of 3 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | @0' - Grass | MD EI |
| | | | | | | | | **Young Axial Channel Deposits (Qya):** | |
| 155 | | | R-1 | 7 10 11 | 97.5 | 24.2 | CL | @ 2.5' - Sandy CLAY: brown, very moist, very stiff, scattered rootlets | |
| | 5 | B-1 | R-2 | 6 11 18 | 109.1 | 19.1 | | @ 5' - Sandy CLAY: brown, very moist, very stiff | |
| 150 | | | R-3 | 5 12 17 | 89.3 | 18.6 | SC | @ 7.5' - Clayey SAND: brown, very moist, medium dense | |
| | 10 | | R-4 | 12 14 16 | 112.9 | 15.7 | CL | @ 10' - Sandy CLAY: brown, moist, very stiff | |
| 145 | | | | | | | | | |
| | 15 | | SPT-1 | 3 5 6 | | 12.6 | SC | @ 15' - Clayey SAND: reddish brown, moist, medium dense | |
| 140 | | | | | | | | | |
| | 20 | | R-5 | 11 16 26 | 108.9 | 3.9 | SM-CL | @ 20' - Silty SAND to Sandy CLAY: brown, slightly moist, dense/hard, traces of gravel | |
| 135 | | | | | | | | | |
| | 25 | | SPT-2 | 4 4 6 | | 13.4 | SC | @ 25' - Clayey SAND: brown, moist, medium dense | |
| 130 | | | | | | | | | |
| | 30 | | | | | | | | |

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**SAMPLE TYPES:**
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- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

**TEST TYPES:**
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- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 10/20/2022

# Geotechnical Boring Log Borehole HS-6

| | |
|---|---|
| **Date:** 2/23/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 6" |
| **Elevation of Top of Hole:** ~158' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map |  |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 30 | | R-6 | 8 8 10 | 88.3 | 19.7 | ML | @ 30' - Sandy SILT: pale brown, very moist, medium dense | |
| 125 | | | | | | | | | |
| | 35 | | SPT-3 | 4 5 6 | | 9.6 | SC | @ 35' - Clayey SAND: brown, moist, medium dense | |
| 120 | | | | | | | | | |
| | 40 | | R-7 | 7 12 20 | 109.4 | 13.3 | | @ 40' - Clayey SAND: brown, moist, medium dense | |
| 115 | | | | | | | | | |
| | 45 | | SPT-4 | 5 7 6 | | 12.9 | | @ 45' - Clayey SAND: brown, moist, medium dense | |
| 110 | | | | | | | | | |
| | 50 | | R-8 | 6 7 12 | 89.9 | 30.7 | CL | @ 50' - Sandy CLAY: brown, very moist, very stiff | |
| 105 | | | | | | | | | |
| | 55 | | SPT-5 | 2 2 5 | | 22.0 | | @ 55' - Sandy CLAY: brown, wet, stiff | |
| 100 | | | | | | | | | |
| | 60 | | | | | | | | |

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

**SAMPLE TYPES:**
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

**TEST TYPES:**
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- MD MAXIMUM DENSITY
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- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

# Geotechnical Boring Log Borehole HS-6

| | |
|---|---|
| **Date:** 2/23/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 6" |
| **Elevation of Top of Hole:** ~158' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 3 of 3 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 60 | | R-9 | 8 13 15 | 92.7 | 31.3 | CL | @ 60' - Sandy CLAY: brown, wet, very stiff | |
| 95 | | | | | | | | | |
| | 65 | | SPT-6 | 2 3 6 | | 21.8 | | @ 65' - Sandy CLAY: brown, wet, stiff | |
| 90 | | | | | | | | | |
| | 70 | | R-10 | 9 13 22 | 102.2 | 24.0 | | @ 70' - Sandy CLAY: brown, wet, very stiff | |
| 85 | | | | | | | | Total Depth = 71.5'<br>Groundwater encountered at 54'<br>Backfilled with Cuttings on 2/23/24 | |
| | 75 | | | | | | | | |
| 80 | | | | | | | | | |
| | 80 | | | | | | | | |
| 75 | | | | | | | | | |
| | 85 | | | | | | | | |
| 70 | | | | | | | | | |
| | 90 | | | | | | | | |

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**SAMPLE TYPES:**
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- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

**TEST TYPES:**
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- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

# Geotechnical Boring Log Borehole I-3

| | |
|---|---|
| **Date:** 2/23/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 8" |
| **Elevation of Top of Hole:** ~160' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 1 of 1 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | @0' - Grass | |
| | | | | | | | | **Young Axial Channel Deposits (Qya):** | |
| | | | R-1 | 6 19 19 | 87.2 | 14.4 | SC | @ 2.5' - Clayey SAND: brown, very moist, dense, traces of roots | |
| 155 | 5 | | R-2 | 11 18 23 | 87.5 | 13.8 | | @ 5' - Clayey SAND: pale brown, very moist, dense | |
| | | | R-3 | 9 12 17 | 88.1 | 11.7 | CL | @ 8.5' - Sandy CLAY: brown, moist, very stiff | -#200 |
| 150 | 10 | | | | | | | Total Depth = 10'<br>No Groundwater Encountered<br>3" Perforated Pipe with Filter Sock and Gravel Installed<br>Pipe Removed and Backfilled with Cuttings on 2/26/2024 | |
| 145 | 15 | | | | | | | | |
| 140 | 20 | | | | | | | | |
| 135 | 25 | | | | | | | | |
| 130 | 30 | | | | | | | | |

LGC Geotechnical, Inc.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

**SAMPLE TYPES:**
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

**TEST TYPES:**
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

# Geotechnical Boring Log Borehole I-4

| | |
|---|---|
| **Date:** 2/23/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 8" |
| **Elevation of Top of Hole:** ~159' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 1 of 1 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | @0' - Grass | |
| | | | | | | | | **Young Axial Channel Deposits (Qya):** | |
| | | | R-1 | 6 9 13 | 103.9 | 15.0 | CL | @ 2.5' - Sandy CLAY: dark brown, moist, very stiff, traces of roots. | |
| 155 | 5 | | R-2 | 12 14 14 | 74.6 | 14.6 | SC | @ 5' - Clayey SAND: pale brown, very moist, medium dense. | |
| 150 | | | R-3 | 19 32 41 | 118.6 | 12.4 | CL | @ 8.5' - Sandy CLAY: dark brown, moist, hard. | |
| | 10 | | | | | | | Total Depth = 10'<br>No Groundwater Encountered<br>3" Perforated Pipe with Filter Sock and Gravel Installed<br>Pipe Removed and Backfilled with Cuttings on 2/26/2024 | |
| 145 | 15 | | | | | | | | |
| 140 | 20 | | | | | | | | |
| 135 | 25 | | | | | | | | |
| 130 | 30 | | | | | | | | |

Geotechnical, Inc.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

**SAMPLE TYPES:**
B BULK SAMPLE
R RING SAMPLE (CA Modified Sampler)
G GRAB SAMPLE
SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

**TEST TYPES:**
DS DIRECT SHEAR
MD MAXIMUM DENSITY
SA SIEVE ANALYSIS
S&H SIEVE AND HYDROMETER
EI EXPANSION INDEX
CN CONSOLIDATION
CR CORROSION
AL ATTERBERG LIMITS
CO COLLAPSE/SWELL
RV R-VALUE
-#200 % PASSING # 200 SIEVE

# *Appendix C*
# *Laboratory Test Results*

## *APPENDIX C*

## ***Laboratory Testing Procedures and Test Results***

The laboratory testing program was formulated towards providing data relating to the relevant engineering properties of the soils with respect to residential construction. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

Moisture and Density Determination Tests: Moisture content (ASTM D2216) and dry density determinations (ASTM D2937) were performed on relatively undisturbed samples obtained from the test borings. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Grain Size Distribution/Fines Content: Representative samples were dried, weighed and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve (ASTM D1140). Where applicable, the portion retained on the No. 200 sieve and dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D6913 (sieve).

| **Sample Location** | **Description** | **% Passing # 200 Sieve** |
|---|---|---|
| I-1 @ 8.5 feet | Clayey Sand | 48 |
| I-3 @ 8.5 feet | Sandy Clay | 68 |

Expansion Index: The expansion potential of selected samples was evaluated by the Expansion Index Test, Standard ASTM D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch-thick by 4-inch-diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below.

| **Sample Location** | **Expansion Index** | **Expansion Potential*** |
|---|---|---|
| HS-3 @ 1-5 feet | 112 | High |
| HS-6 @ 1-5 feet | 135 | Very High |

* ASTM D4829

Maximum Density Tests: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM D1557. The results of these tests are presented in the table below:

*APPENDIX C (Cont'd)*

***Laboratory Testing Procedures and Test Results***

| Sample Location | Sample Description | Maximum Dry Density (pcf) | Optimum Moisture Content (%) |
|---|---|---|---|
| HS-3 @ 1-5 feet | Dark Grayish Brown Clay | 117.5 | 12.5 |
| HS-6 @ 1-5 feet | Dark Brown Sandy Clay | 115.0 | 14.0 |

Collapse/Swell Potential: Six collapse tests were performed per ASTM D4546. A sample (2.4 inches in diameter and 1-inch in height) was placed in a consolidometer and loaded to the approximate in-situ effective stress. The curve is presented in this Appendix.

Chloride Content: Chloride content was tested in accordance with Caltrans Test Method (CTM) 422. The results are presented below.

| Sample Location | Chloride Content, ppm |
|---|---|
| HS-3 @ 1-5 feet | 60 |

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM 417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below.

| Sample Location | Sulfate Content (ppm) | Sulfate Exposure Class * |
|---|---|---|
| HS-3 @ 1-5 feet | 95 | S0 |

*Based on ACI 318R-14, Table 19.3.1.1

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with CTM 643 and standard geochemical methods. The results are presented in the table below.

| Sample Location | pH | Minimum Resistivity (ohms-cm) |
|---|---|---|
| HS-3 @ 1-5 feet | 7.90 | 720 |

## ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

**ASTM D 4546**

| | | | | | |
|---|---|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala | Date: | 05/20/24 |
| Project No.: | 23174-01 | Checked By: | J. Ward | Date: | 05/22/24 |
| Boring No.: | HS-1 | Sample Type: | Ring | | |
| Sample No.: | R-4 | Depth (ft.) | 10.0 | | |
| Sample Description: | Brown lean clay (CL) | | | | |

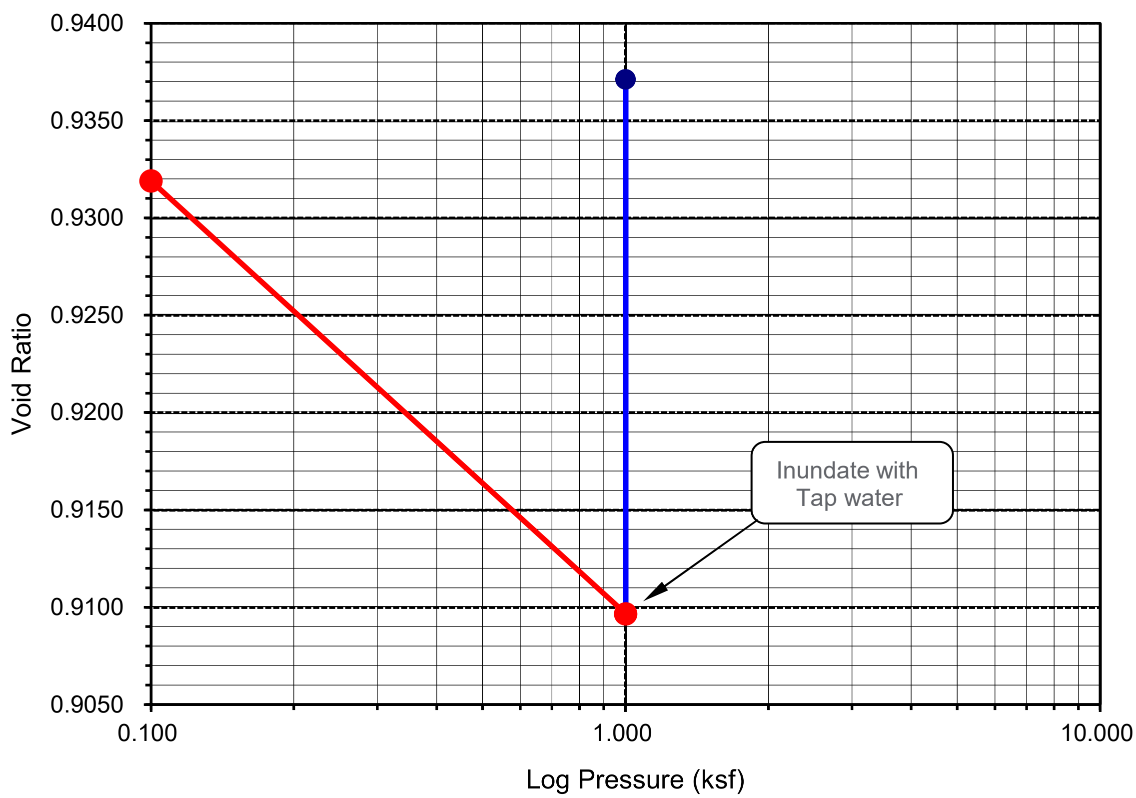
| | |
|---|---|
| Initial Dry Density (pcf): | 87.2 |
| Initial Moisture (%): | 22.94 |
| Initial Length (in.): | 1.0000 |
| Initial Dial Reading: | 0.1187 |
| Diameter(in): | 2.415 |

| | |
|---|---|
| Final Dry Density (pcf): | 87.3 |
| Final Moisture (%) : | 31.6 |
| Initial Void ratio: | 0.9335 |
| Specific Gravity(assumed): | 2.70 |
| Initial Saturation (%) | 66.4 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.1195 | 0.9992 | 0.00 | -0.08 | 0.9319 | -0.08 |
| 1.000 | 0.1325 | 0.9862 | 0.15 | -1.38 | 0.9097 | -1.23 |
| H2O | 0.1183 | 1.0004 | 0.15 | 0.04 | 0.9371 | 0.19 |

**Percent Swell (+) / Settlement (-) After Inundation = 1.44**

Void Ratio - Log Pressure Curve



# ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
## ASTM D 4546

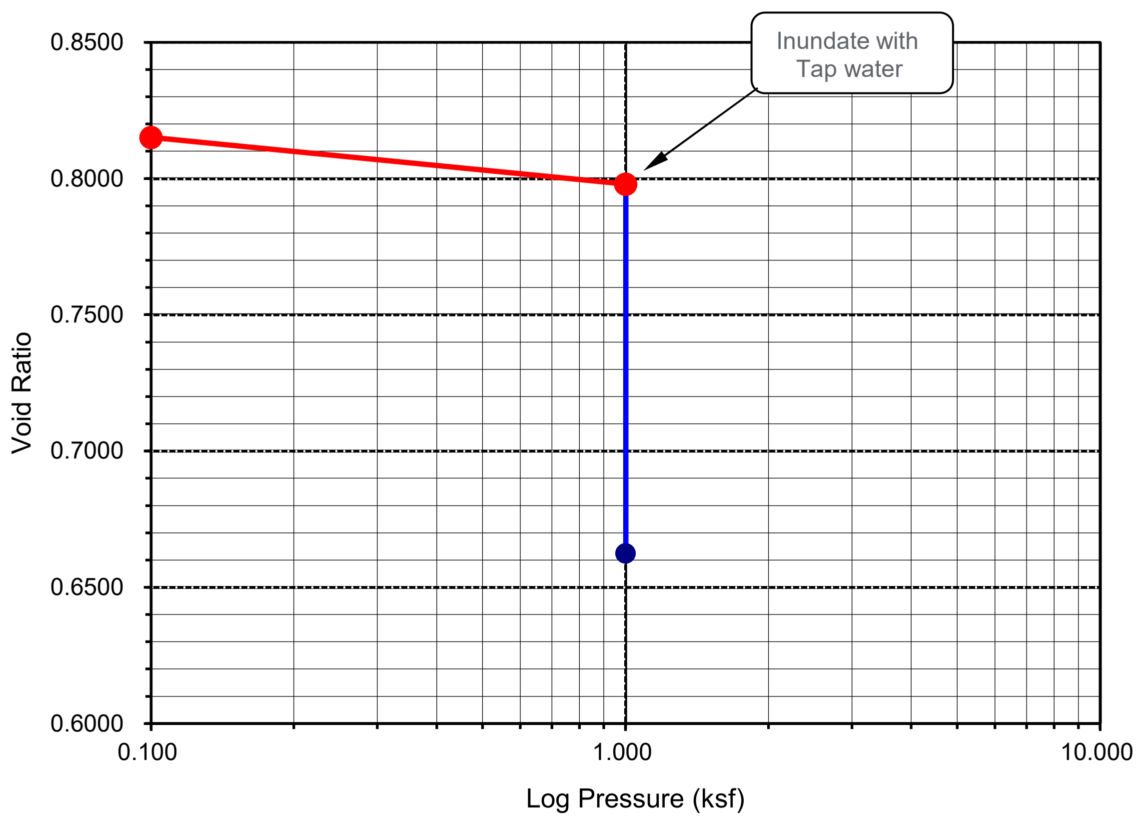
| | | | | | |
|---|---|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala | Date: | 05/21/24 |
| Project No.: | 23174-01 | Checked By: | J. Ward | Date: | 05/23/24 |
| Boring No.: | HS-2 | Sample Type: | Ring | | |
| Sample No.: | R-4 | Depth (ft.) | 10.0 | | |
| Sample Description: | Brown silty clay (CL-ML) | | | | |

| | | | |
|---|---|---|---|
| Initial Dry Density (pcf): | 92.3 | Final Dry Density (pcf): | 101.7 |
| Initial Moisture (%): | 7.54 | Final Moisture (%) : | 20.8 |
| Initial Length (in.): | 1.0000 | Initial Void ratio: | 0.8270 |
| Initial Dial Reading: | 0.1122 | Specific Gravity(assumed): | 2.70 |
| Diameter(in): | 2.415 | Initial Saturation (%) | 24.6 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.1187 | 0.9935 | 0.00 | -0.65 | 0.8151 | -0.65 |
| 1.000 | 0.1296 | 0.9826 | 0.15 | -1.74 | 0.7980 | -1.59 |
| H2O | 0.2038 | 0.9084 | 0.15 | -9.16 | 0.6624 | -9.01 |

**Percent Swell (+) / Settlement (-) After Inundation = -7.54**

Void Ratio - Log Pressure Curve



# ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

**ASTM D 4546**

| | | | | | |
|---|---|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala | Date: | 03/13/24 |
| Project No.: | 23174-01 | Checked By: | J. Ward | Date: | 03/21/24 |
| Boring No.: | HS-2 | Sample Type: | Ring | | |
| Sample No.: | R-5 | Depth (ft.) | 15.0 | | |
| Sample Description: | Brown silty sand (SM) | | | | |

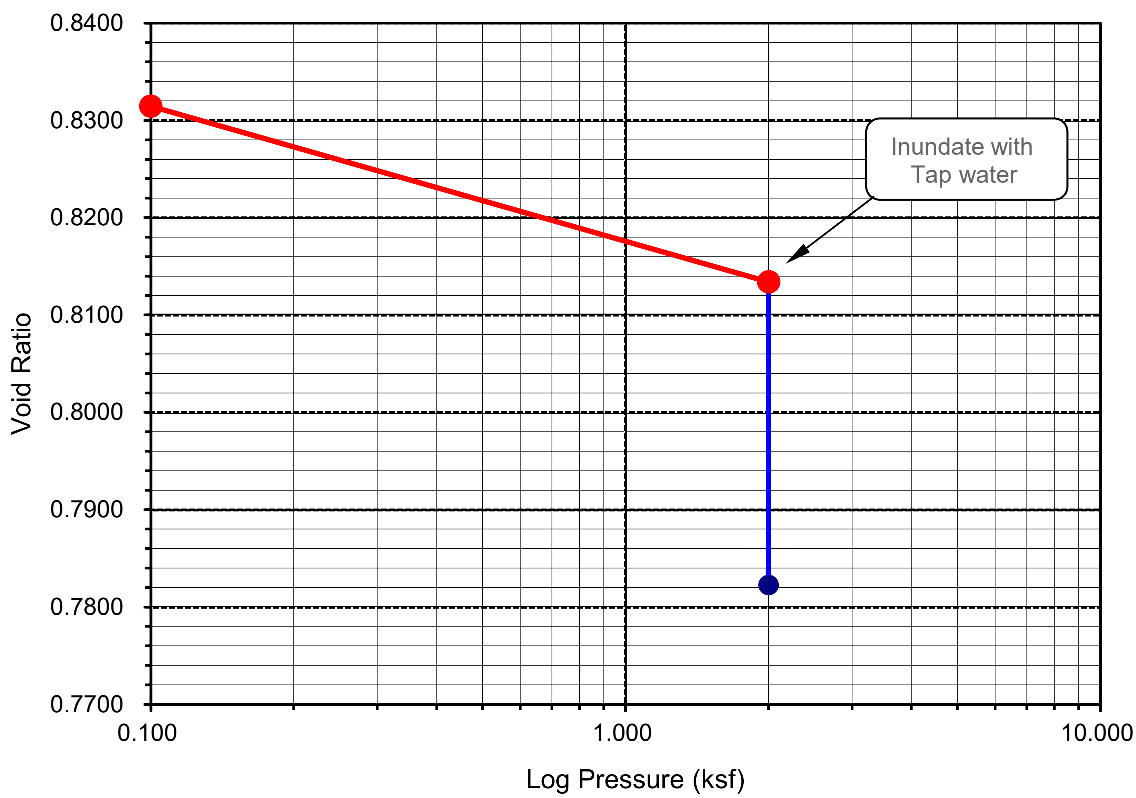
| | |
|---|---|
| Initial Dry Density (pcf): | 92.0 |
| Initial Moisture (%): | 8.87 |
| Initial Length (in.): | 1.0000 |
| Initial Dial Reading: | 0.0680 |
| Diameter(in): | 2.415 |

| | |
|---|---|
| Final Dry Density (pcf): | 95.9 |
| Final Moisture (%) : | 22.0 |
| Initial Void ratio: | 0.8325 |
| Specific Gravity(assumed): | 2.70 |
| Initial Saturation (%) | 28.8 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.06855 | 0.9995 | 0.00 | -0.06 | 0.8315 | -0.06 |
| 2.000 | 0.0849 | 0.9831 | 0.65 | -1.69 | 0.8134 | -1.04 |
| H2O | 0.1019 | 0.9661 | 0.65 | -3.39 | 0.7823 | -2.74 |

**Percent Swell (+) / Settlement (-) After Inundation = -1.72**

Void Ratio - Log Pressure Curve



# ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

**ASTM D 4546**

| | | | | | |
|---|---|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala | Date: | 05/20/24 |
| Project No.: | 23174-01 | Checked By: | J. Ward | Date: | 05/22/24 |
| Boring No.: | HS-3 | Sample Type: | Ring | | |
| Sample No.: | R-4 | Depth (ft.) | 10.0 | | |
| Sample Description: | Brown lean clay with sand (CL)s | | | | |

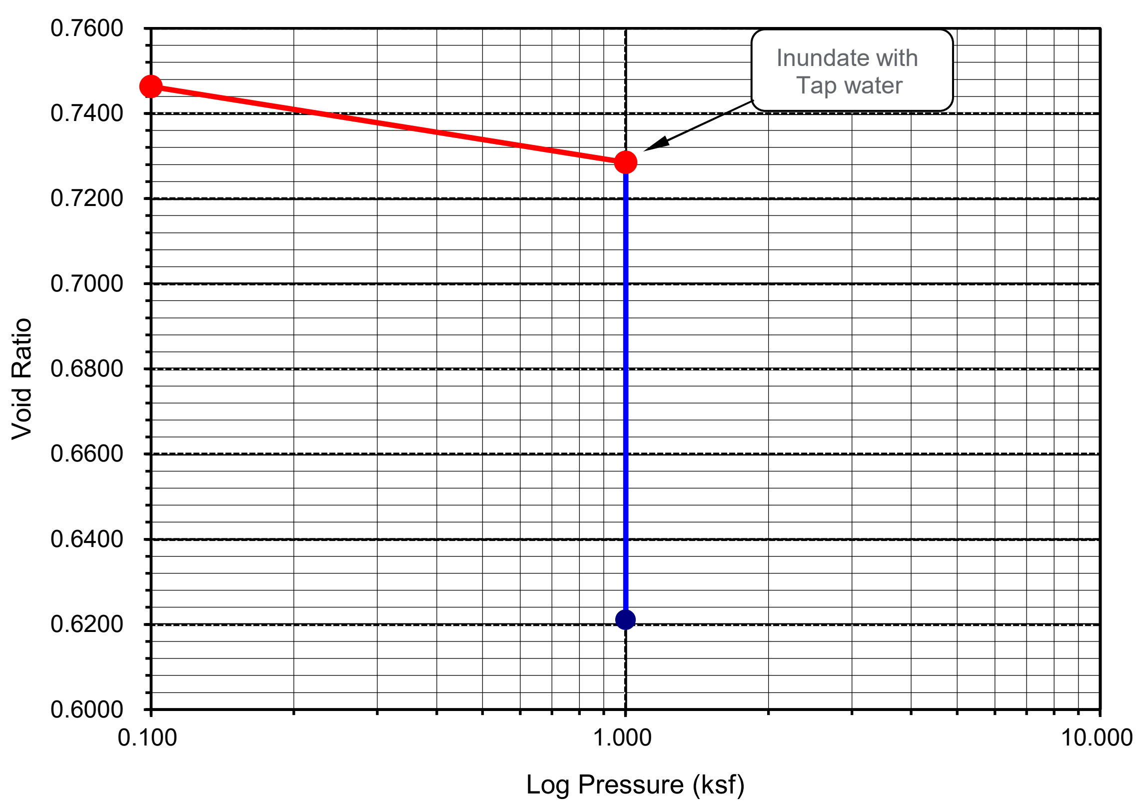
| | |
|---|---|
| Initial Dry Density (pcf): | 96.5 |
| Initial Moisture (%): | 5.37 |
| Initial Length (in.): | 1.0000 |
| Initial Dial Reading: | 0.0967 |
| Diameter(in): | 2.415 |

| | |
|---|---|
| Final Dry Density (pcf): | 104.3 |
| Final Moisture (%) : | 19.1 |
| Initial Void ratio: | 0.7469 |
| Specific Gravity(assumed): | 2.70 |
| Initial Saturation (%) | 19.4 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.0970 | 0.9997 | 0.00 | -0.03 | 0.7464 | -0.03 |
| 1.000 | 0.1087 | 0.9880 | 0.15 | -1.20 | 0.7285 | -1.05 |
| H2O | 0.1702 | 0.9265 | 0.15 | -7.35 | 0.6211 | -7.20 |

**Percent Swell (+) / Settlement (-) After Inundation = -6.22**

Void Ratio - Log Pressure Curve



# ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

**ASTM D 4546**

| | | | | | |
|---|---|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala | Date: | 05/21/24 |
| Project No.: | 23174-01 | Checked By: | J. Ward | Date: | 05/23/24 |
| Boring No.: | HS-4 | Sample Type: | Ring | | |
| Sample No.: | R-4 | Depth (ft.) | 10.0 | | |
| Sample Description: | Brown lean clay (CL) | | | | |

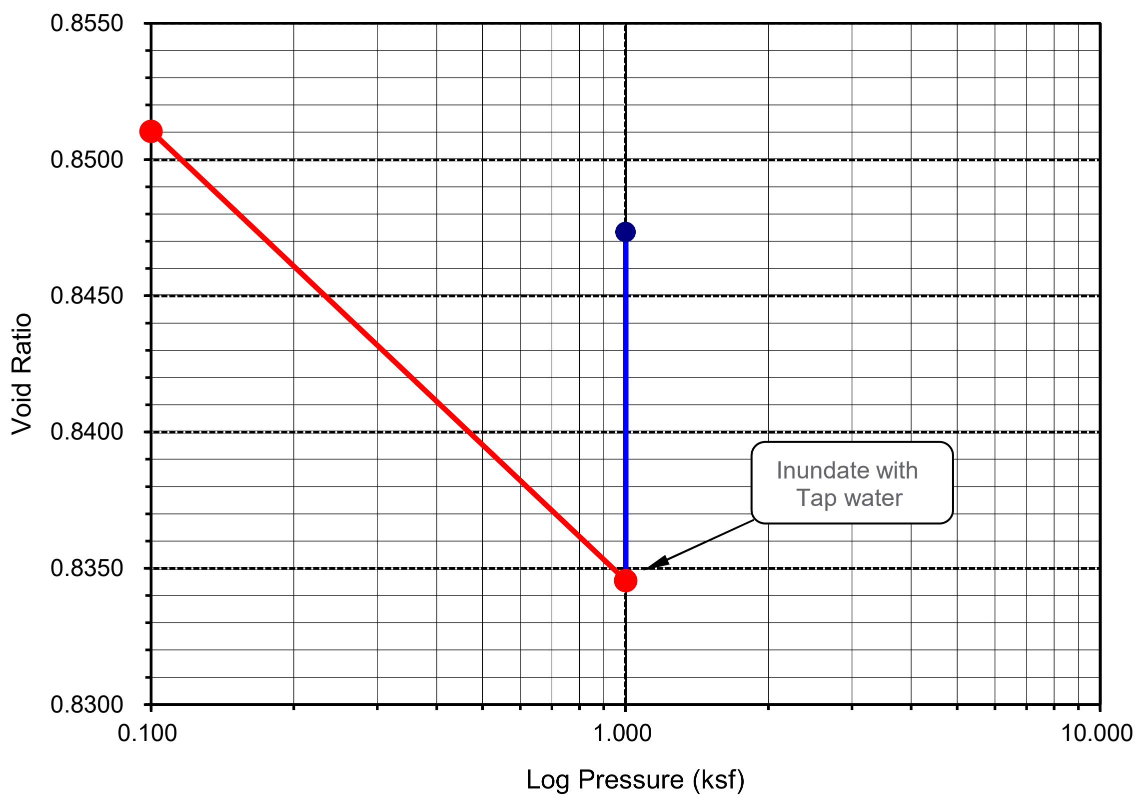
| | |
|---|---|
| Initial Dry Density (pcf): | 91.0 |
| Initial Moisture (%): | 13.63 |
| Initial Length (in.): | 1.0000 |
| Initial Dial Reading: | 0.0961 |
| Diameter(in): | 2.415 |

| | |
|---|---|
| Final Dry Density (pcf): | 91.5 |
| Final Moisture (%) : | 24.3 |
| Initial Void ratio: | 0.8522 |
| Specific Gravity(assumed): | 2.70 |
| Initial Saturation (%) | 43.2 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.0967 | 0.9994 | 0.00 | -0.06 | 0.8510 | -0.06 |
| 1.000 | 0.1071 | 0.9890 | 0.15 | -1.10 | 0.8346 | -0.95 |
| H2O | 0.1002 | 0.9959 | 0.15 | -0.41 | 0.8473 | -0.26 |

**Percent Swell (+) / Settlement (-) After Inundation = 0.70**

Void Ratio - Log Pressure Curve



# ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

**ASTM D 4546**

| | | | |
|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala |
| Project No.: | 23174-01 | Checked By: | J. Ward |
| Boring No.: | HS-5 | Sample Type: | Ring |
| Sample No.: | R-4 | Depth (ft.) | 10.0 |
| Sample Description: | Olive brown silty clay (CL-ML) | | |

Date: 03/13/24
Date: 03/21/24

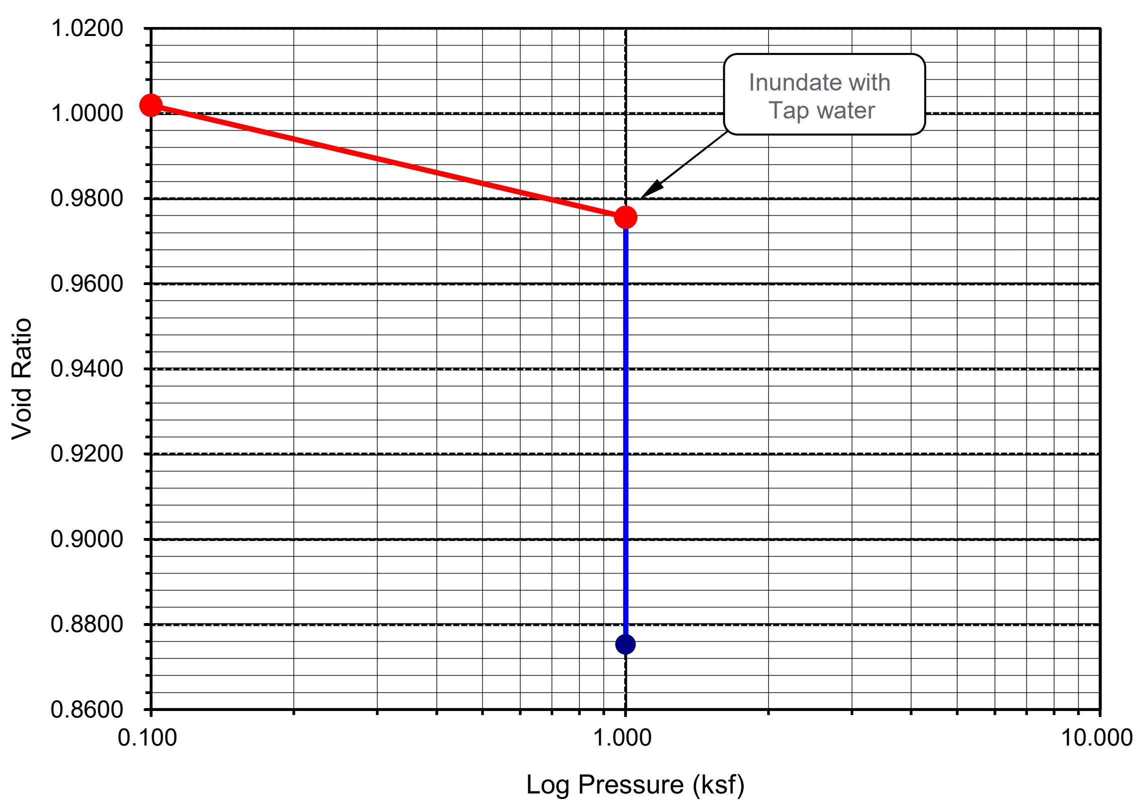
| | |
|---|---|
| Initial Dry Density (pcf): | 84.2 |
| Initial Moisture (%): | 12.07 |
| Initial Length (in.): | 1.0000 |
| Initial Dial Reading: | 0.1519 |
| Diameter(in): | 2.415 |

| | |
|---|---|
| Final Dry Density (pcf): | 90.2 |
| Final Moisture (%) : | 25.5 |
| Initial Void ratio: | 1.0029 |
| Specific Gravity(assumed): | 2.70 |
| Initial Saturation (%) | 32.5 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.1524 | 0.9995 | 0.00 | -0.05 | 1.0019 | -0.05 |
| 1.000 | 0.1670 | 0.9849 | 0.15 | -1.51 | 0.9757 | -1.36 |
| H2O | 0.2171 | 0.9348 | 0.15 | -6.52 | 0.8753 | -6.37 |

**Percent Swell (+) / Settlement (-) After Inundation = -5.08**

Void Ratio - Log Pressure Curve



# *Appendix D*
# *Infiltration Test Data*

# Infiltration Test Data Sheet

**LGC Geotechnical, Inc**

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

**Project Name:** Shea - Our Lady of Peace, Irvine
**Project Number:** 23174-01
**Date:** 2/26/2024
**Boring Number:** I-3

| Test hole dimensions (if circular) | |
|---|---|
| Boring Depth (feet)*: | 10 |
| Boring Diameter (inches): | 8 |
| Pipe Diameter (inches): | 3 |

*measured at time of test

| Test pit dimensions (if rectangular) | |
|---|---|
| Pit Depth (feet): | |
| Pit Length (feet): | |
| Pit Breadth (feet): | |

Minimum test Head ($D_o$):

(What the sounder tape should read) Boring Depth - (5 x Boring Radius) 8.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

### *Pre-Test (Sandy Soil Criteria)**

| Trial No. | Start Time (24:HR) | Stop Time (24:HR) | Time Interval (min) | Initial Depth to Water (feet) | Final Depth to Water (feet) | Total Change in Water Level (feet) | Greater Than or Equal to 0.5 feet (yes/no) |
|---|---|---|---|---|---|---|---|
| 1 | 7:15 | 7:40 | 25.0 | 6.88 | 6.91 | 0.03 | Yes |
| 2 | 7:42 | 8:07 | 25.0 | 6.81 | 6.84 | 0.03 | Yes |

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight, and then obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25 inches

### *Main Test Data*

| Trial No. | Start Time (24:HR) | Stop Time (24:HR) | Time Interval, Dt (min) | Initial Depth to Water, $D_o$ (feet) | Final Depth to Water, $D_f$ (feet) | Change in Water Level, DD (feet) | Measured Infiltration Rate(in/hr) |
|---|---|---|---|---|---|---|---|
| 1 | 8:04 | 8:34 | 30.0 | 6.84 | 6.87 | 0.03 | 0.04 |
| 2 | 8:36 | 9:06 | 30.0 | 6.87 | 6.89 | 0.02 | 0.02 |
| 3 | 9:08 | 9:38 | 30.0 | 6.85 | 6.87 | 0.02 | 0.02 |
| 4 | 9:40 | 10:10 | 30.0 | 6.87 | 6.9 | 0.03 | 0.04 |
| 5 | 10:12 | 10:42 | 30.0 | 6.91 | 6.94 | 0.03 | 0.04 |
| 6 | 10:45 | 11:15 | 30.0 | 6.82 | 6.85 | 0.03 | 0.04 |
| 7 | 11:18 | 11:48 | 30.0 | 6.85 | 6.87 | 0.02 | 0.02 |
| 8 | 11:50 | 12:20 | 30.0 | 6.87 | 6.89 | 0.02 | 0.02 |
| 9 | 12:22 | 12:52 | 30.0 | 6.86 | 6.88 | 0.02 | 0.02 |
| 10 | 12:54 | 13:24 | 30.0 | 6.84 | 6.86 | 0.02 | 0.02 |
| 11 | 13:26 | 13:56 | 30.0 | 6.86 | 6.88 | 0.02 | 0.02 |
| 12 | 13:58 | 14:28 | 30.0 | 6.88 | 6.90 | 0.02 | 0.02 |
| | | | | | | **Measured Infiltration Rate (No Factor of Safety)** | **0.02** |

***Sketch:***

***Notes:***

Based on Guidelines from: South Orange County 9/28/2017

Spreadsheet Revised on: 10/30/2019

LGC Geotechnical, Inc.

# Infiltration Test Data Sheet

**LGC Geotechnical, Inc**

**131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141**

**Project Name:** Shea - Our Lady of Peace, Irvine
**Project Number:** 23174-01
**Date:** 2/26/2024
**Boring Number:** I-4

| Test hole dimensions (if circular) | |
|---|---|
| Boring Depth (feet)*: | 10 |
| Boring Diameter (inches): | 8 |
| Pipe Diameter (inches): | 3 |

*measured at time of test

| Test pit dimensions (if rectangular) | |
|---|---|
| Pit Depth (feet): | |
| Pit Length (feet): | |
| Pit Breadth (feet): | |

Minimum test Head ($D_o$):

(What the sounder tape should read) Boring Depth - (5 x Boring Radius) 8.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

***Pre-Test (Sandy Soil Criteria)****

| Trial No. | Start Time (24:HR) | Stop Time (24:HR) | Time Interval (min) | Initial Depth to Water (feet) | Final Depth to Water (feet) | Total Change in Water Level (feet) | Greater Than or Equal to 0.5 feet (yes/no) |
|---|---|---|---|---|---|---|---|
| 1 | 7:17 | 7:42 | 25.0 | 7.04 | 7.06 | 0.02 | Yes |
| 2 | 7:44 | 8:09 | 25.0 | 7.06 | 7.08 | 0.02 | Yes |

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight, and then obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25 inches

***Main Test Data***

| Trial No. | Start Time (24:HR) | Stop Time (24:HR) | Time Interval, Dt (min) | Initial Depth to Water, $D_o$ (feet) | Final Depth to Water, $D_f$ (feet) | Change in Water Level, DD (feet) | Measured Infiltration Rate(in/hr) |
|---|---|---|---|---|---|---|---|
| 1 | 8:11 | 8:41 | 30.0 | 7.05 | 7.08 | 0.03 | 0.04 |
| 2 | 8:43 | 9:13 | 30.0 | 7.01 | 7.03 | 0.02 | 0.03 |
| 3 | 9:15 | 9:45 | 30.0 | 7.03 | 7.05 | 0.02 | 0.03 |
| 4 | 9:47 | 10:17 | 30.0 | 7.02 | 7.03 | 0.01 | 0.01 |
| 5 | 10:10 | 10:49 | 39.0 | 7.03 | 7.05 | 0.02 | 0.02 |
| 6 | 10:51 | 11:21 | 30.0 | 7.05 | 7.07 | 0.02 | 0.03 |
| 7 | 11:23 | 11:53 | 30.0 | 7.00 | 7.02 | 0.02 | 0.03 |
| 8 | 11:55 | 12:25 | 30.0 | 7.02 | 7.03 | 0.01 | 0.01 |
| 9 | 12:28 | 12:58 | 30.0 | 7.03 | 7.05 | 0.02 | 0.03 |
| 10 | 13:00 | 13:30 | 30.0 | 7.05 | 7.06 | 0.01 | 0.01 |
| 11 | 13:32 | 14:02 | 30.0 | 7.05 | 7.06 | 0.01 | 0.01 |
| 12 | 14:04 | 14:34 | 30.0 | 7.06 | 7.07 | 0.01 | 0.01 |
| | | | | | | **Measured Infiltration Rate (No Factor of Safety)** | **0.01** |

***Sketch:***

***Notes:***

Based on Guidelines from: South Orange County 9/28/2017

Spreadsheet Revised on: 10/30/2019

LGC Geotechnical, Inc.

# *Appendix E*
# *General Earthwork and Grading Specifications for Rough Grading*

## *General Earthwork and Grading Specifications for Rough Grading*

### *1.0 General*

#### *1.1 Intent*

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

#### *1.2 The Geotechnical Consultant of Record*

Prior to commencement of work, the owner shall employ a qualified Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to confirm that the attained level of compaction is being accomplished as specified. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

#### *1.3 The Earthwork Contractor*

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the project plans and specifications. The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of “equipment” of work and the estimated quantities of daily earthwork

contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the
Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate personnel will be available for observation and testing. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified. It is the contractor's sole responsibility to provide proper fill compaction.

## *2.0 Preparation of Areas to be Filled*

### *2.1 Clearing and Grubbing*

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

### *2.2 Processing*

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be over-excavated as specified in the following section. Scarification shall continue until soils are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

### *2.3 Over-excavation*

In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by the Geotechnical Consultant during grading.

### *2.4 Benching*

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

### *2.5 Evaluation/Acceptance of Fill Areas*

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

## *3.0 Fill Material*

### *3.1 General*

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

### *3.2 Oversize*

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

### *3.3 Import*

If importing of fill material is required for grading, proposed import material shall meet the requirements of the geotechnical consultant. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

## *4.0 Fill Placement and Compaction*

### *4.1 Fill Layers*

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

### *4.2 Fill Moisture Conditioning*

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

### *4.3 Compaction of Fill*

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

### *4.4 Compaction of Fill Slopes*

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

### *4.5 Compaction Testing*

Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

### *4.6 Frequency of Compaction Testing*

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

### *4.7 Compaction Test Locations*

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than
5 feet apart from potential test locations shall be provided.

## *5.0 Subdrain Installation*

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

## *6.0 Excavation*

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

## *7.0 Trench Backfills*

***7.1*** The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

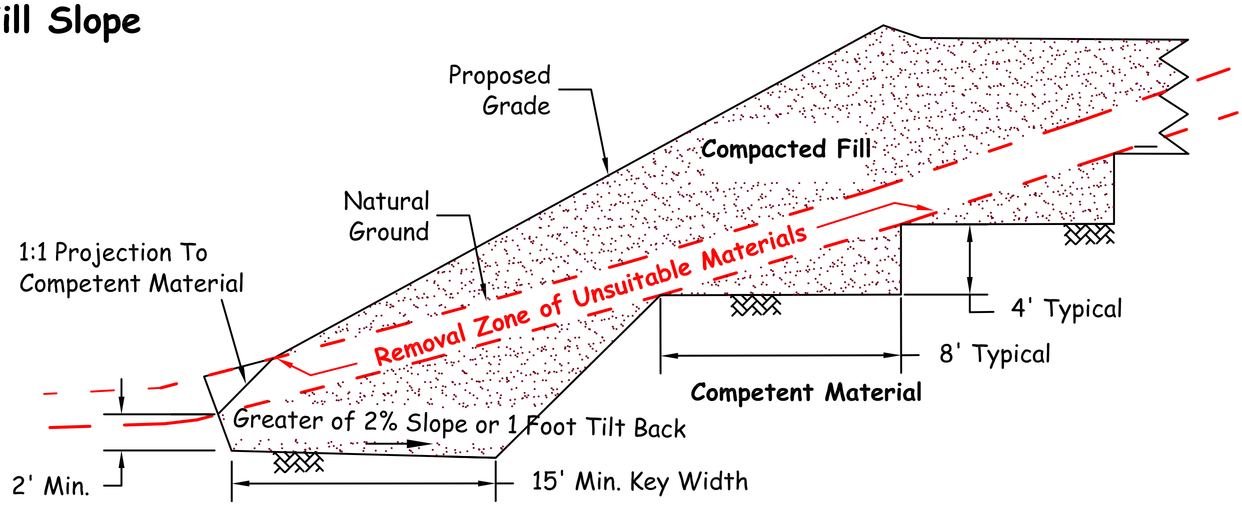
***7.2*** All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

***7.3*** The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

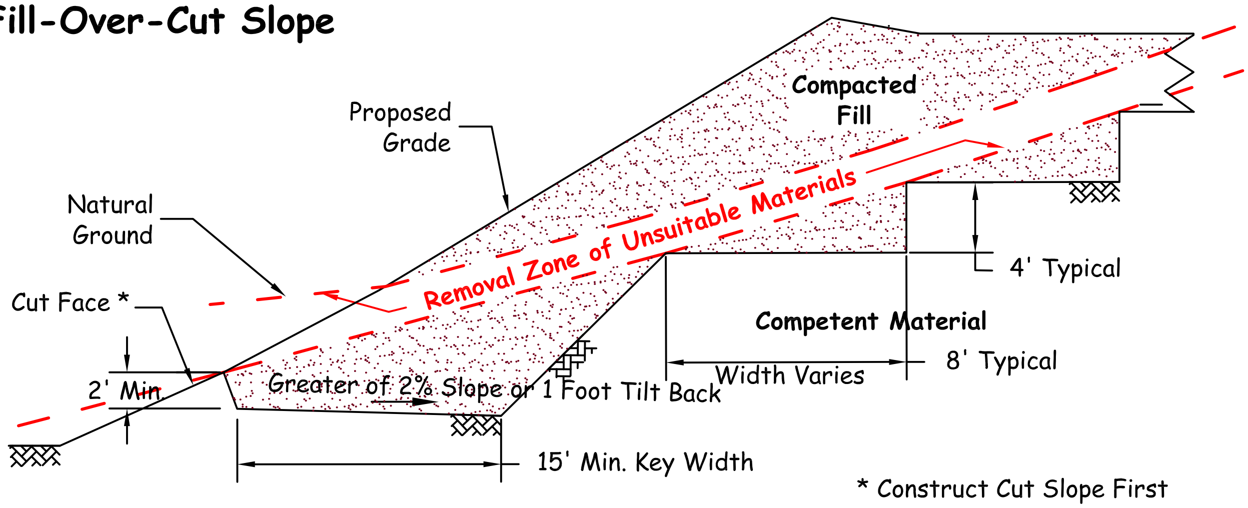
***7.4*** The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

***7.5*** Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

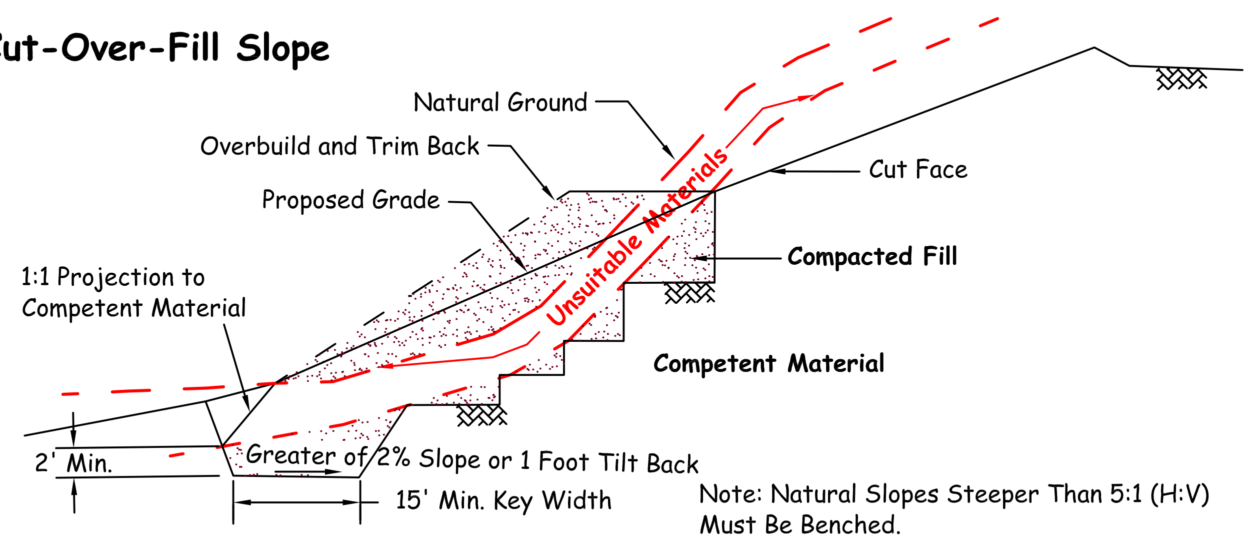
## Fill Slope




## Fill-Over-Cut Slope




## Cut-Over-Fill Slope

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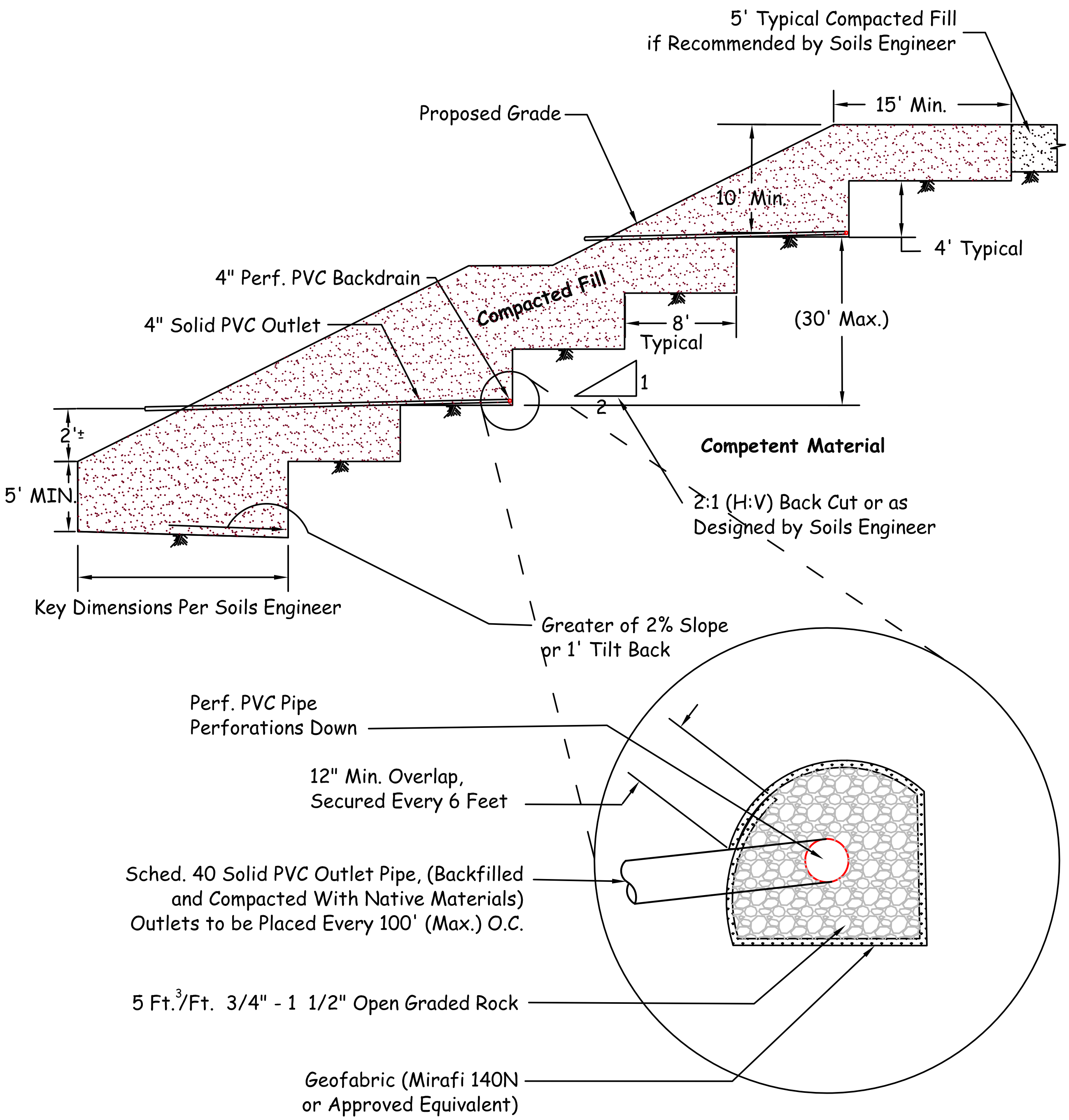
# KEYING AND BENCHING

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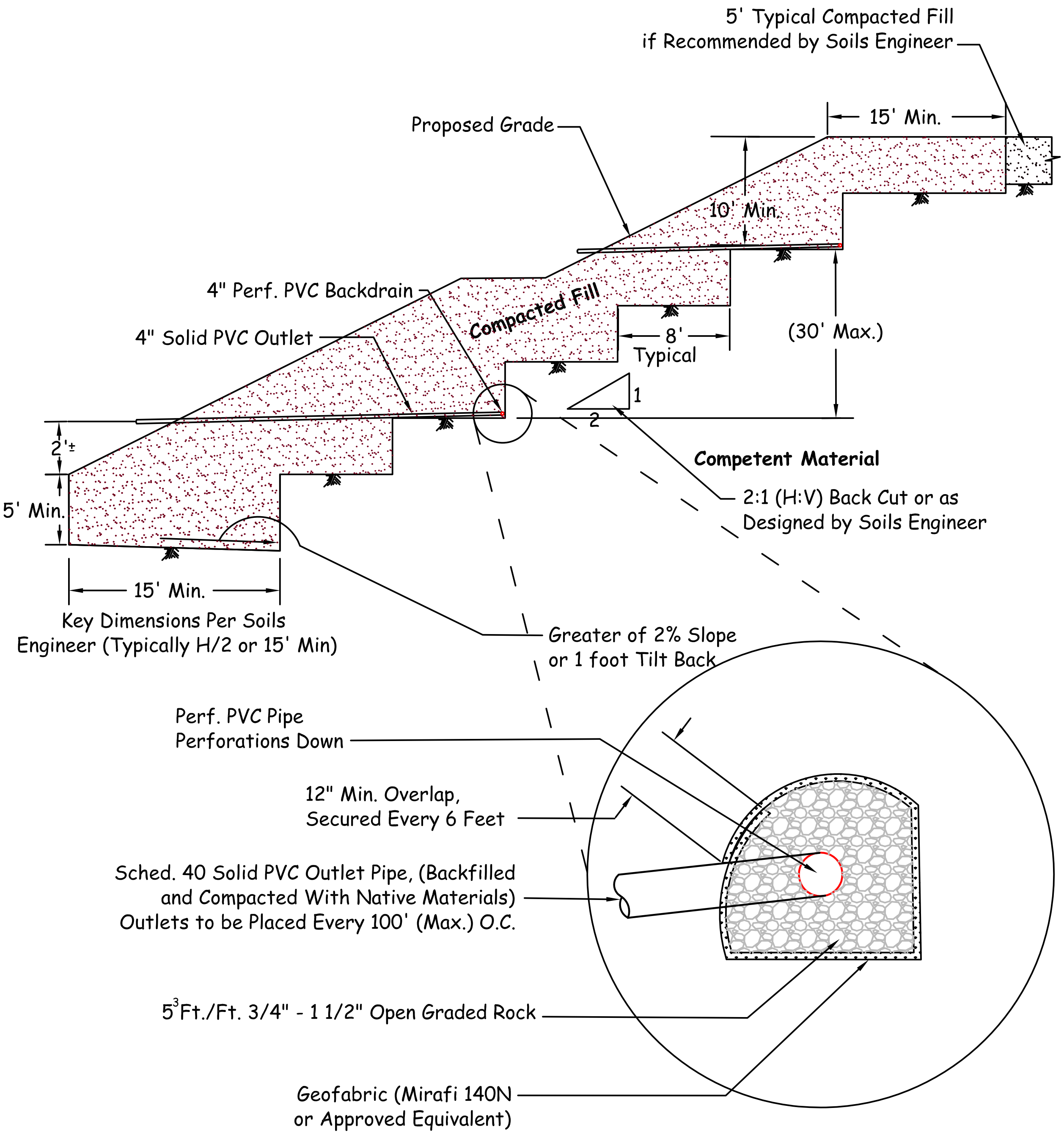
# TYPICAL BUTTRESS DETAIL

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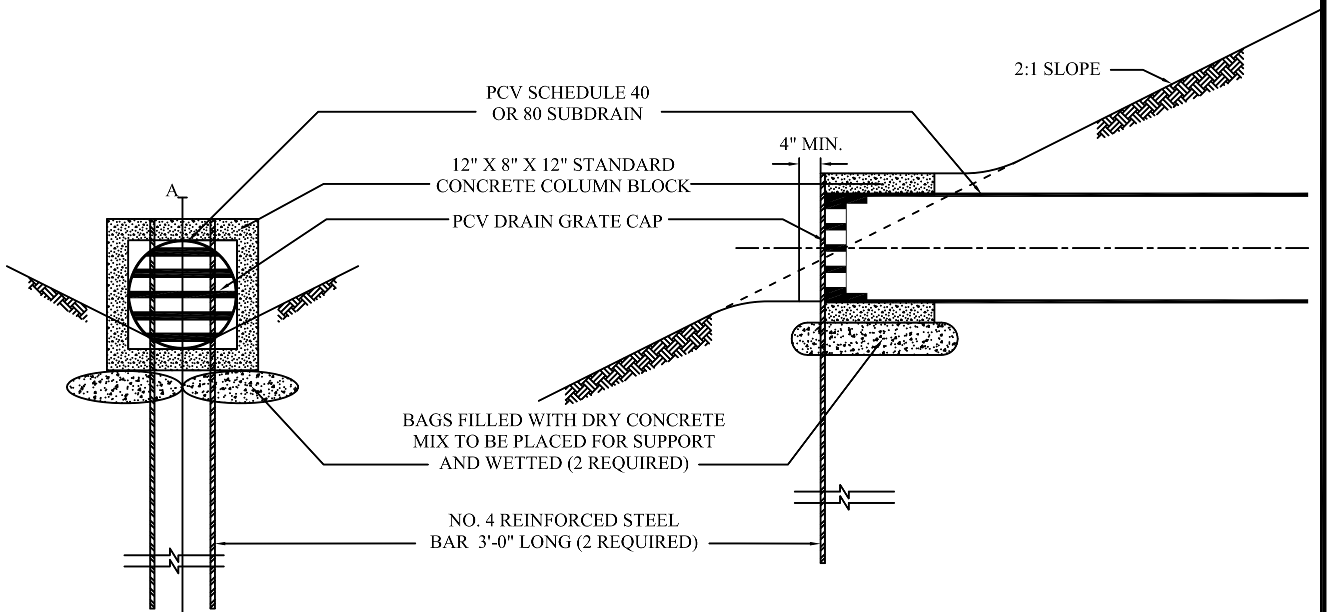
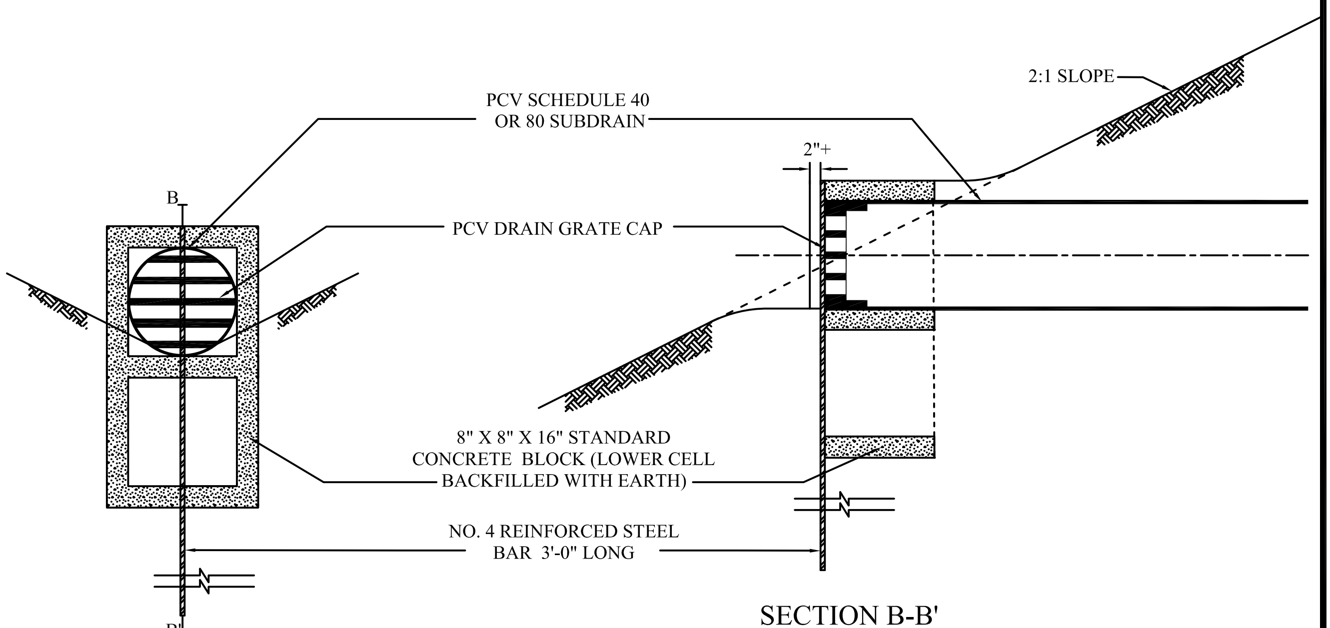
# TYPICAL STABILIZATION FILL DETAIL

# SUBDRAIN OUTLET MARKER -6" & 8" PIPE




SECTION A-A'

# SUBDRAIN OUTLET MARKER -4" PIPE




SECTION B-B'

NOT TO SCALE

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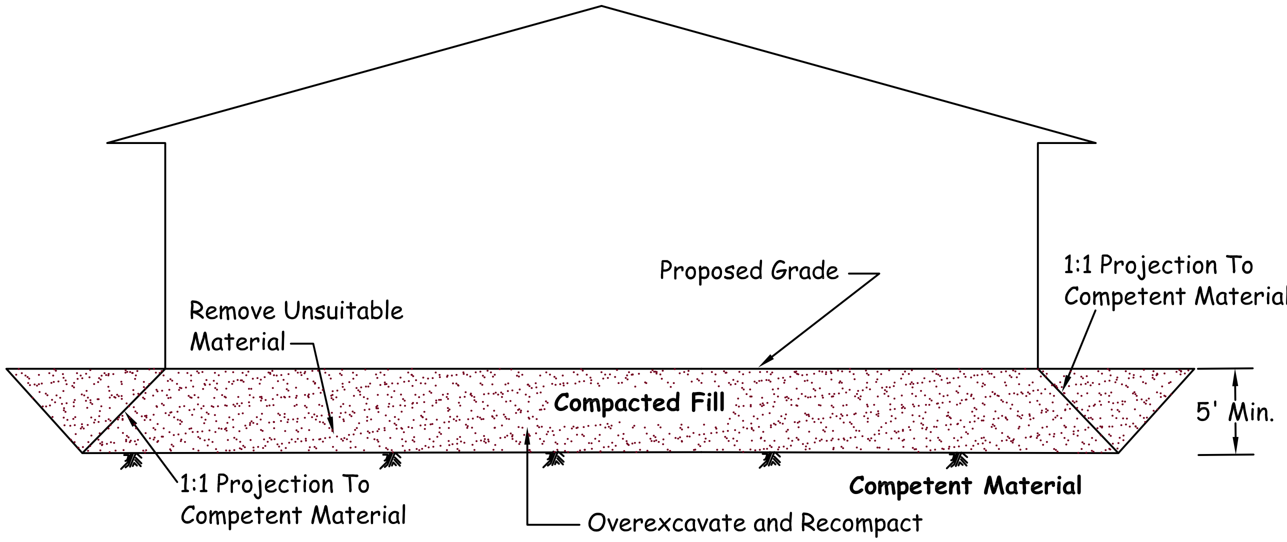
**SUBDRAIN OUTLET MARKER DETAIL**

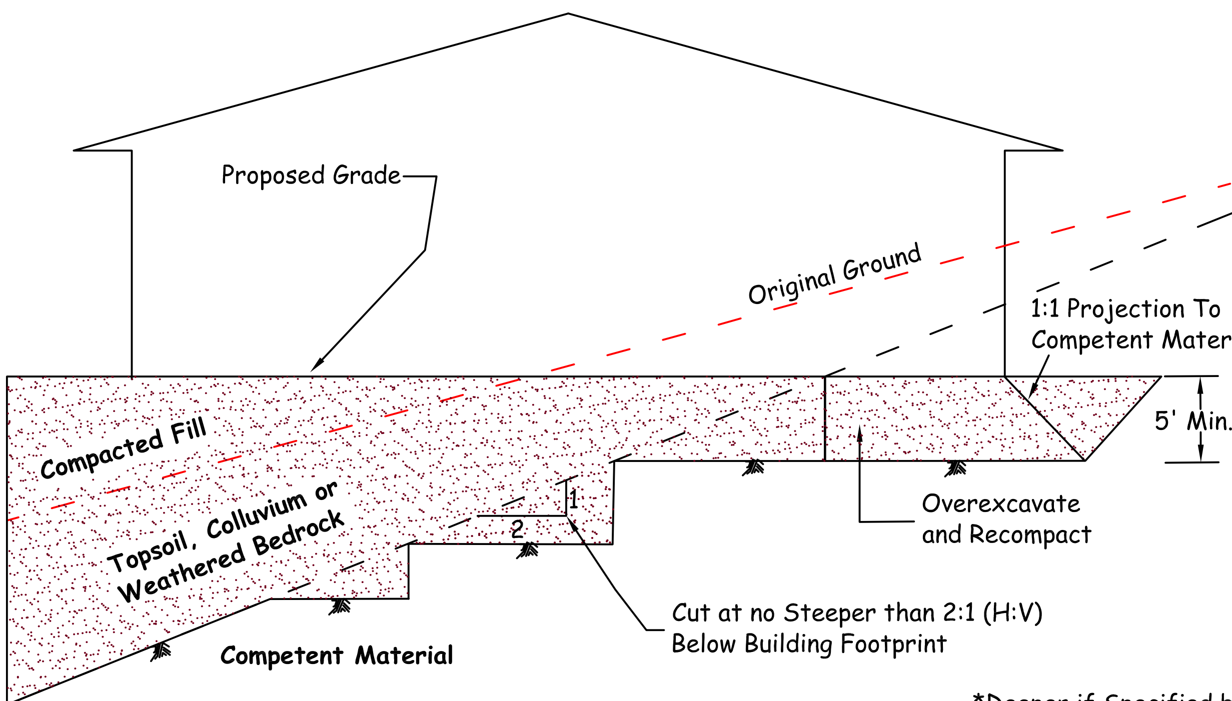
## Cut Lot
(Exposing Unsuitable Soils at Design Grade)




Note 1: Removal Bottom Should be Graded With Minimum 2% Fall Towards Street or Other Suitable Area (as Determined by Soils Engineer) to Avoid Ponding Below Building

Note 2: Where Design Cut Lots are Excavated Entirely Into Competent Material, Overexcavation May Still be Required for Hard-Rock Conditions or for Materials With Variable Expansion Characteristics.

## Cut/Fill Transition Lot




*Deeper if Specified by
Soils Engineer

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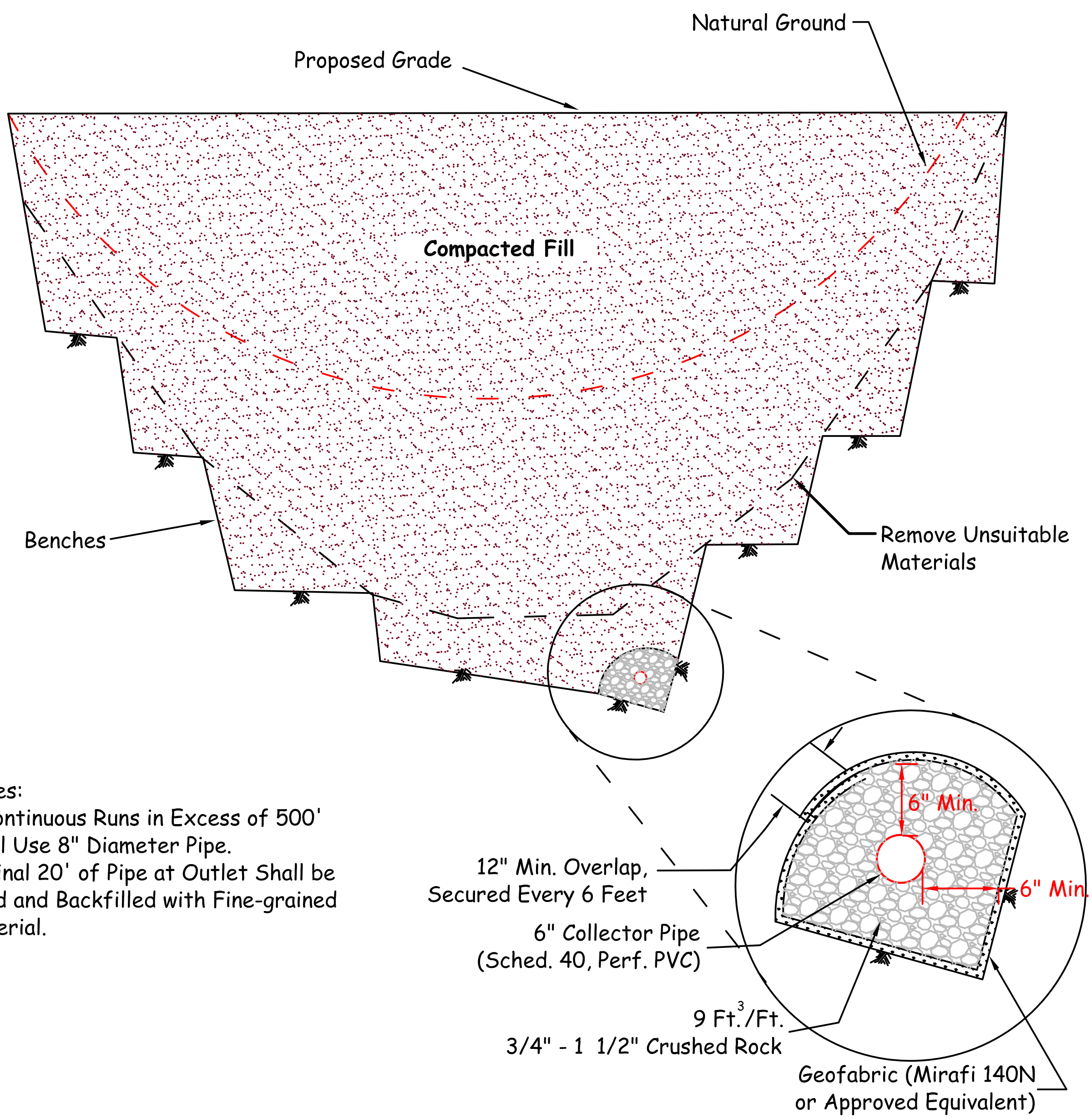
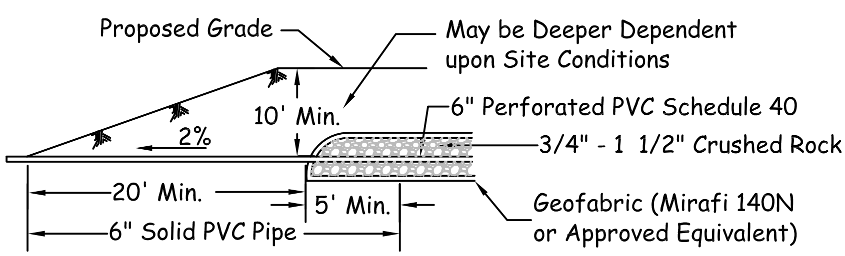
# CUT AND TRANSITION LOT OVEREXCAVATION DETAIL




Notes:
1) Continuous Runs in Excess of 500' Shall Use 8" Diameter Pipe.
2) Final 20' of Pipe at Outlet Shall be Solid and Backfilled with Fine-grained Material.

## Proposed Outlet Detail

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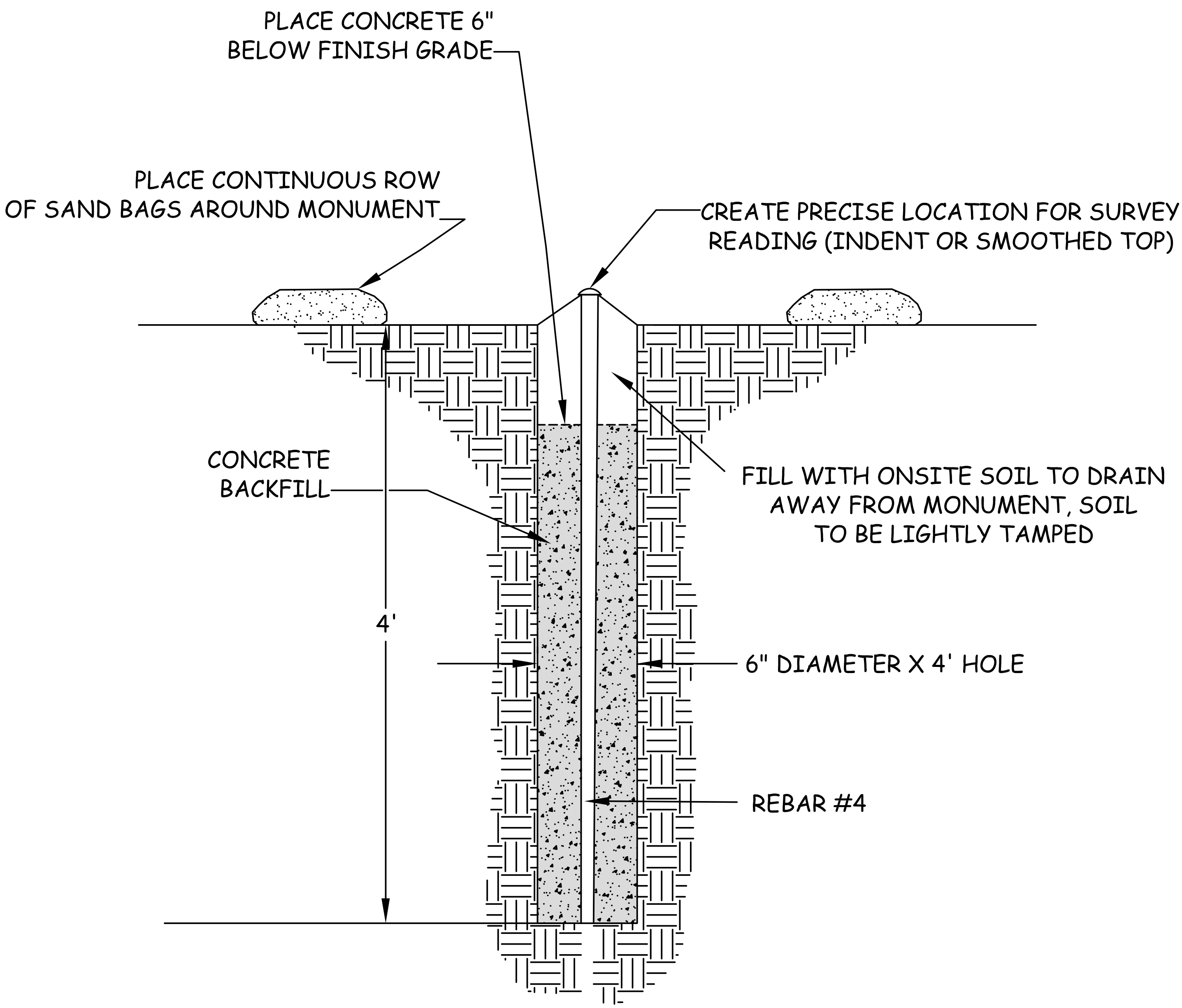
# CANYON SUBDRAINS




NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS

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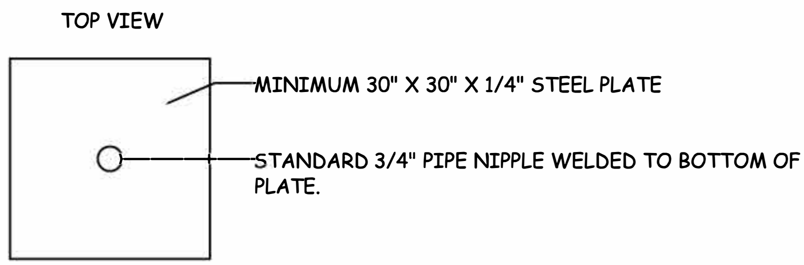
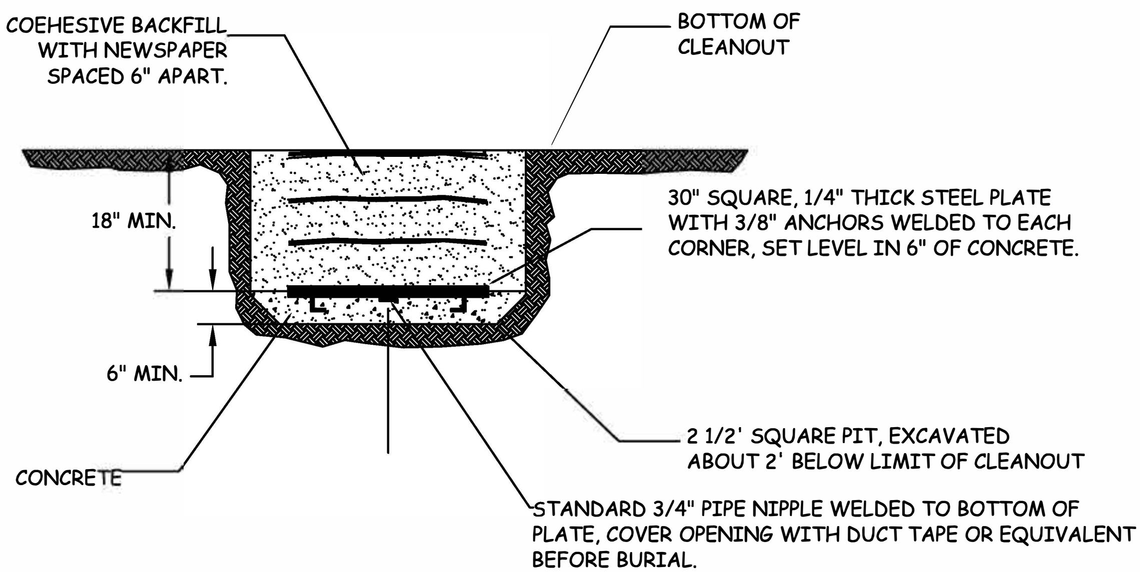
# TYPICAL SURFACE SETTLEMENT MONUMENT




1. SURVEY FOR HORIZONTAL AND VERTICAL LOCATION TO NEAREST .01 INCH PRIOR TO BACKFILL USING KNOW LOCATIONS THAT WILL REMAIN INTACT DURING THE DURATION OF THE MONITORING PROGRAM. KNOW POINTS EXPLICITELY NOT ALLOWED ARE THOSE LOCATED ON FILL OR THAT WILL BE DESTROYED DURING GRADING.
2. IN THE EVENT OF DAMAGE TO SETTLEMENT PLATE DURING GRADING, CONTRACTOR SHALL IMMEDIATELY NOTIFY THE GEOTECHNICAL ENGINEER AND SHALL BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.
3. DRILL TO RECOVER AND ATTACH RISER PIPE.

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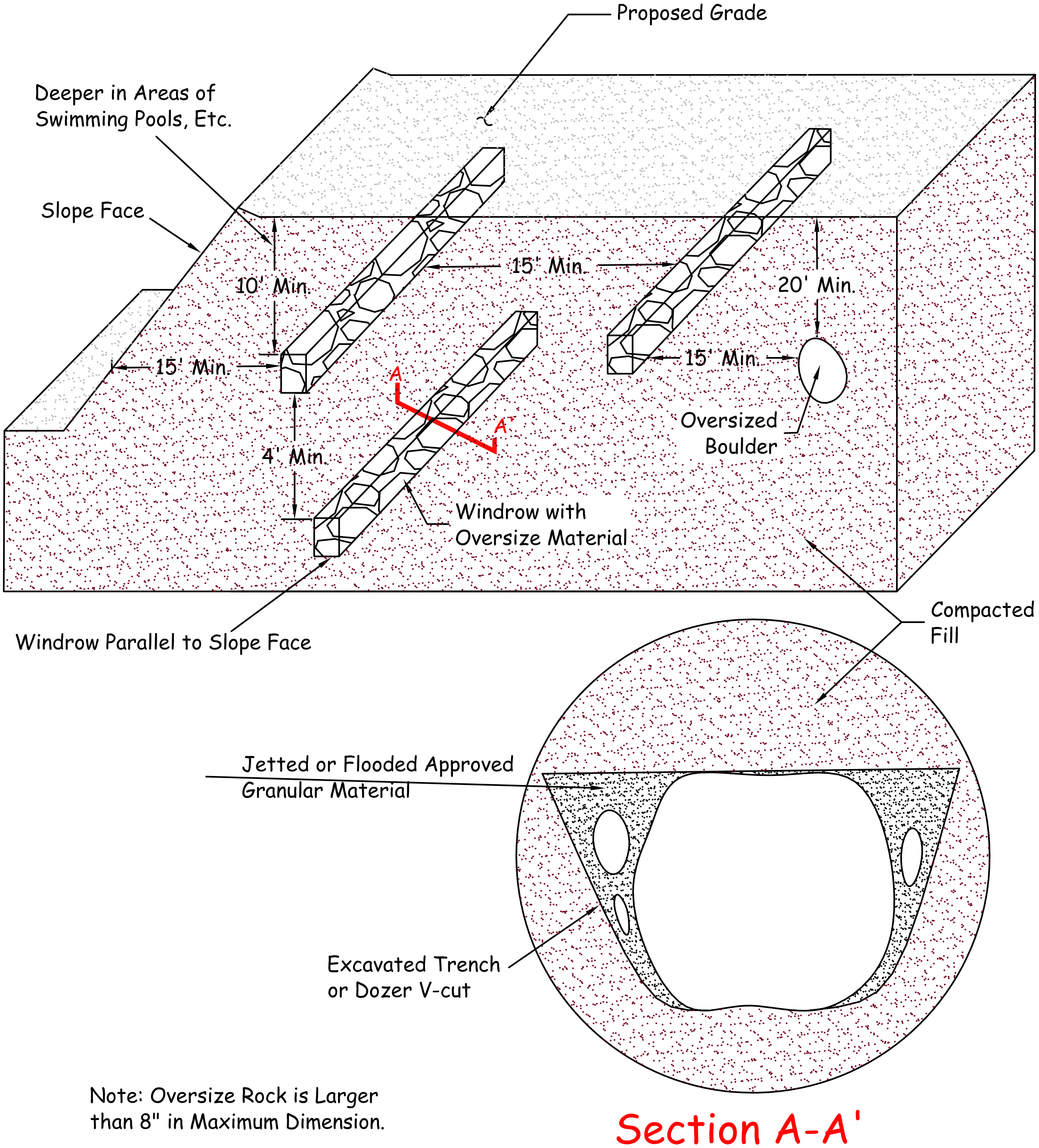
# TYPICAL SETTLEMENT PLATE AND RISER




Note: Oversize Rock is Larger than 8" in Maximum Dimension.

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# OVERSIZE ROCK DISPOSAL DETAIL

# Attachment E: 2 Year Storm Hydrology


PROACTIVE
ENGINEERING CONSULTANTS
WWW.PROACTIVEENGINEERING.NET (949) 716-7460
27042 Towne Centre Drive, Suite 110, Foothill Ranch, CA 92610

OUR LADY OF PEACE - HYDROLOGY MAP EXISTING CHURCH

11/10/2025

```
****************************************************************************
             RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
            (Reference: 1986 ORANGE COUNTY HYDROLOGY CRITERION)
         (c) Copyright 1983-2015 Advanced Engineering Software (aes)
             Ver. 22.0  Release Date: 07/01/2015  License ID 1673

                           Analysis prepared by:






 ************************** DESCRIPTION OF STUDY **************************
* OLP CHURCH                                                               *
* EXISTING                                                                 *
* 2 YEAR                                                                   *
 **************************************************************************

  FILE NAME: CHRCHEX2.DAT
  TIME/DATE OF STUDY: 13:46 11/08/2025
============================================================================
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
============================================================================
                     --*TIME-OF-CONCENTRATION MODEL*--

  USER SPECIFIED STORM EVENT(YEAR) =    2.00
  SPECIFIED MINIMUM PIPE SIZE(INCH) =  18.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.95
  *DATA BANK RAINFALL USED*
  *ANTECEDENT MOISTURE CONDITION (AMC) II ASSUMED FOR RATIONAL METHOD*

  *USER-DEFINED STREET-SECTIONS FOR COUPLED PIPEFLOW AND STREETFLOW MODEL*
     HALF-  CROWN TO   STREET-CROSSFALL:   CURB  GUTTER-GEOMETRIES:  MANNING
     WIDTH  CROSSFALL  IN-  / OUT-/PARK-  HEIGHT  WIDTH  LIP   HIKE  FACTOR
NO.   (FT)     (FT)    SIDE / SIDE/ WAY    (FT)    (FT)  (FT)  (FT)    (n)
===  =====  =========  =================  ======  ===== ====== ===== =======
  1   30.0     20.0    0.018/0.018/0.020   0.67    2.00 0.0312 0.167 0.0150

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.00 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint =  6.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*
  *USER-SPECIFIED MINIMUM TOPOGRAPHIC SLOPE ADJUSTMENT NOT SELECTED

****************************************************************************
  FLOW PROCESS FROM NODE    100.00 TO NODE    110.00 IS CODE =  21
```

```
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
  >>USE TIME-OF-CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA<<
============================================================================
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   260.00
  ELEVATION DATA: UPSTREAM(FEET) =    164.50  DOWNSTREAM(FEET) =    160.30

  Tc = K*[(LENGTH** 3.00)/(ELEVATION CHANGE)]**0.20
  SUBAREA ANALYSIS USED MINIMUM Tc(MIN.) =   11.079
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.434
  SUBAREA Tc AND LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS   Tc
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN  (MIN.)
  NATURAL POOR COVER
  "BARREN"                   C        0.38      0.25     1.000    91   11.08
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  1.000
  SUBAREA RUNOFF(CFS) =      0.40
  TOTAL AREA(ACRES) =      0.38   PEAK FLOW RATE(CFS) =      0.40

****************************************************************************
  FLOW PROCESS FROM NODE    110.00 TO NODE    111.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    160.30  DOWNSTREAM(FEET) =    158.30
  CHANNEL LENGTH THRU SUBAREA(FEET) =   115.00   CHANNEL SLOPE =  0.0174
  CHANNEL BASE(FEET) =   70.00   "Z" FACTOR =   5.000
  MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   1.00
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.260
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  COMMERCIAL                 C        0.34      0.25     0.100    69
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.100
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.59
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   0.68
  AVERAGE FLOW DEPTH(FEET) =   0.01   TRAVEL TIME(MIN.) =   2.81
  Tc(MIN.) =   13.88
  SUBAREA AREA(ACRES) =     0.34       SUBAREA RUNOFF(CFS) =    0.38
  EFFECTIVE AREA(ACRES) =      0.72     AREA-AVERAGED Fm(INCH/HR) =   0.14
  AREA-AVERAGED Fp(INCH/HR) =   0.25  AREA-AVERAGED Ap =   0.57
  TOTAL AREA(ACRES) =        0.7         PEAK FLOW RATE(CFS) =       0.72

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.01   FLOW VELOCITY(FEET/SEC.) =   0.83
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    111.00 =     375.00 FEET.
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    111.00 TO NODE    111.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
  TIME OF CONCENTRATION(MIN.) =   13.88
  RAINFALL INTENSITY(INCH/HR) =   1.26
  AREA-AVERAGED Fm(INCH/HR) =  0.14
  AREA-AVERAGED Fp(INCH/HR) =  0.25
  AREA-AVERAGED Ap =  0.57
  EFFECTIVE STREAM AREA(ACRES) =       0.72
  TOTAL STREAM AREA(ACRES) =       0.72
  PEAK FLOW RATE(CFS) AT CONFLUENCE =       0.72

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    121.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
  >>USE TIME-OF-CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA<<
============================================================================
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   285.00
  ELEVATION DATA: UPSTREAM(FEET) =    164.20  DOWNSTREAM(FEET) =    158.70

  Tc = K*[(LENGTH** 3.00)/(ELEVATION CHANGE)]**0.20
  SUBAREA ANALYSIS USED MINIMUM Tc(MIN.) =   11.092
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.433
  SUBAREA Tc AND LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS   Tc
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN  (MIN.)
  NATURAL POOR COVER
  "BARREN"                   C        0.61      0.25     1.000    91   11.09
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  1.000
  SUBAREA RUNOFF(CFS) =      0.65
  TOTAL AREA(ACRES) =      0.61   PEAK FLOW RATE(CFS) =      0.65

****************************************************************************
  FLOW PROCESS FROM NODE    121.00 TO NODE    111.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =   11.09
  RAINFALL INTENSITY(INCH/HR) =   1.43
  AREA-AVERAGED Fm(INCH/HR) =  0.25
  AREA-AVERAGED Fp(INCH/HR) =  0.25
```

```
  AREA-AVERAGED Ap =  1.00
  EFFECTIVE STREAM AREA(ACRES) =       0.61
  TOTAL STREAM AREA(ACRES) =       0.61
  PEAK FLOW RATE(CFS) AT CONFLUENCE =       0.65

  ** CONFLUENCE DATA **
   STREAM       Q      Tc   Intensity   Fp(Fm)     Ap     Ae     HEADWATER
   NUMBER     (CFS)  (MIN.) (INCH/HR) (INCH/HR)         (ACRES)    NODE
      1        0.72   13.88    1.260  0.25( 0.14) 0.57       0.7     100.00
      2        0.65   11.09    1.433  0.25( 0.25) 1.00       0.6     120.00

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  2 STREAMS.

  ** PEAK FLOW RATE TABLE **
   STREAM       Q      Tc   Intensity   Fp(Fm)     Ap     Ae     HEADWATER
   NUMBER     (CFS)  (MIN.) (INCH/HR) (INCH/HR)         (ACRES)    NODE
      1        1.32   11.09    1.433  0.25( 0.20) 0.79       1.2     120.00
      2        1.28   13.88    1.260  0.25( 0.19) 0.77       1.3     100.00

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =       1.32    Tc(MIN.) =    11.09
  EFFECTIVE AREA(ACRES) =       1.19   AREA-AVERAGED Fm(INCH/HR) =  0.20
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.79
  TOTAL AREA(ACRES) =       1.3
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    111.00 =     375.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    111.00 TO NODE    112.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   158.30  DOWNSTREAM(FEET) =   156.20
  FLOW LENGTH(FEET) =   140.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   3.9 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   4.61
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       1.32
  PIPE TRAVEL TIME(MIN.) =   0.51    Tc(MIN.) =   11.60
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    112.00 =     515.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    112.00 TO NODE    112.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
```

```
  TIME OF CONCENTRATION(MIN.) =   11.60
  RAINFALL INTENSITY(INCH/HR) =   1.40
  AREA-AVERAGED Fm(INCH/HR) =  0.20
  AREA-AVERAGED Fp(INCH/HR) =  0.25
  AREA-AVERAGED Ap =  0.79
  EFFECTIVE STREAM AREA(ACRES) =       1.19
  TOTAL STREAM AREA(ACRES) =       1.33
  PEAK FLOW RATE(CFS) AT CONFLUENCE =       1.32

****************************************************************************
  FLOW PROCESS FROM NODE    130.00 TO NODE    131.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
  >>USE TIME-OF-CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA<<
============================================================================
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   230.00
  ELEVATION DATA: UPSTREAM(FEET) =    162.20  DOWNSTREAM(FEET) =    157.70

  Tc = K*[(LENGTH** 3.00)/(ELEVATION CHANGE)]**0.20
  SUBAREA ANALYSIS USED MINIMUM Tc(MIN.) =    5.879
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  2.063
  SUBAREA Tc AND LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS   Tc
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN  (MIN.)
  COMMERCIAL                 C        0.29      0.25     0.100    69    5.88
  NATURAL POOR COVER
  "BARREN"                   C        0.53      0.25     1.000    91   10.15
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.682
  SUBAREA RUNOFF(CFS) =      1.40
  TOTAL AREA(ACRES) =      0.82   PEAK FLOW RATE(CFS) =      1.40

****************************************************************************
  FLOW PROCESS FROM NODE    131.00 TO NODE    112.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   157.70  DOWNSTREAM(FEET) =   156.20
  FLOW LENGTH(FEET) =   150.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   4.5 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   4.05
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       1.40
  PIPE TRAVEL TIME(MIN.) =   0.62    Tc(MIN.) =    6.50
  LONGEST FLOWPATH FROM NODE    130.00 TO NODE    112.00 =     380.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    112.00 TO NODE    112.00 IS CODE =   1
```

```
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =    6.50
  RAINFALL INTENSITY(INCH/HR) =   1.95
  AREA-AVERAGED Fm(INCH/HR) =  0.17
  AREA-AVERAGED Fp(INCH/HR) =  0.25
  AREA-AVERAGED Ap =  0.68
  EFFECTIVE STREAM AREA(ACRES) =       0.82
  TOTAL STREAM AREA(ACRES) =       0.82
  PEAK FLOW RATE(CFS) AT CONFLUENCE =       1.40

  ** CONFLUENCE DATA **
   STREAM       Q      Tc   Intensity   Fp(Fm)     Ap     Ae     HEADWATER
   NUMBER     (CFS)  (MIN.) (INCH/HR) (INCH/HR)         (ACRES)    NODE
      1        1.32   11.60    1.397  0.25( 0.20) 0.79       1.2     120.00
      1        1.28   14.40    1.234  0.25( 0.19) 0.77       1.3     100.00
      2        1.40    6.50    1.948  0.25( 0.17) 0.68       0.8     130.00

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  2 STREAMS.

  ** PEAK FLOW RATE TABLE **
   STREAM       Q      Tc   Intensity   Fp(Fm)     Ap     Ae     HEADWATER
   NUMBER     (CFS)  (MIN.) (INCH/HR) (INCH/HR)         (ACRES)    NODE
      1        2.47    6.50    1.948  0.25( 0.18) 0.73       1.5     130.00
      2        2.28   11.60    1.397  0.25( 0.19) 0.75       2.0     120.00
      3        2.11   14.40    1.234  0.25( 0.18) 0.74       2.2     100.00

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =       2.47    Tc(MIN.) =     6.50
  EFFECTIVE AREA(ACRES) =       1.48   AREA-AVERAGED Fm(INCH/HR) =  0.18
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.73
  TOTAL AREA(ACRES) =       2.2
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    112.00 =     515.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    112.00 TO NODE    112.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  MAINLINE Tc(MIN.) =    6.50
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.948
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  COMMERCIAL                 C        0.22      0.25     0.100    69
```

```
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.100
  SUBAREA AREA(ACRES) =    0.22      SUBAREA RUNOFF(CFS) =    0.38
  EFFECTIVE AREA(ACRES) =      1.70   AREA-AVERAGED Fm(INCH/HR) =  0.16
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.65
  TOTAL AREA(ACRES) =        2.4       PEAK FLOW RATE(CFS) =       2.74
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        2.4  TC(MIN.) =      6.50
  EFFECTIVE AREA(ACRES) =      1.70  AREA-AVERAGED Fm(INCH/HR)=  0.16
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap = 0.650
  PEAK FLOW RATE(CFS)   =       2.74

  ** PEAK FLOW RATE TABLE **
   STREAM       Q       Tc   Intensity   Fp(Fm)     Ap     Ae     HEADWATER
   NUMBER     (CFS)  (MIN.) (INCH/HR) (INCH/HR)         (ACRES)    NODE
      1        2.74    6.50    1.948  0.25( 0.16) 0.65       1.7     130.00
      2        2.45   11.60    1.397  0.25( 0.17) 0.68       2.2     120.00
      3        2.27   14.40    1.234  0.25( 0.17) 0.68       2.4     100.00
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```


PROACTIVE ENGINEERING CONSULTANTS
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27042 Towne Centre Drive, Suite 110, Foothill Ranch, CA 92610

OUR LADY OF PEACE - PROP CHURCH

11/10/2025

```
****************************************************************************
             RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
            (Reference: 1986 ORANGE COUNTY HYDROLOGY CRITERION)
         (c) Copyright 1983-2015 Advanced Engineering Software (aes)
             Ver. 22.0  Release Date: 07/01/2015  License ID 1673

                           Analysis prepared by:






 ************************** DESCRIPTION OF STUDY **************************
* OLP CHURCH                                                               *
* PROPOSED                                                                 *
* 2 YEAR                                                                   *
 **************************************************************************

  FILE NAME: CHRCHPR2.DAT
  TIME/DATE OF STUDY: 13:53 11/08/2025
============================================================================
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
============================================================================
                    --*TIME-OF-CONCENTRATION MODEL*--

  USER SPECIFIED STORM EVENT(YEAR) =    2.00
  SPECIFIED MINIMUM PIPE SIZE(INCH) =  18.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.95
  *DATA BANK RAINFALL USED*
  *ANTECEDENT MOISTURE CONDITION (AMC) II ASSUMED FOR RATIONAL METHOD*

  *USER-DEFINED STREET-SECTIONS FOR COUPLED PIPEFLOW AND STREETFLOW MODEL*
     HALF-  CROWN TO   STREET-CROSSFALL:   CURB  GUTTER-GEOMETRIES:  MANNING
     WIDTH  CROSSFALL  IN-  / OUT-/PARK-  HEIGHT  WIDTH  LIP   HIKE  FACTOR
NO.   (FT)     (FT)    SIDE / SIDE/ WAY    (FT)    (FT)  (FT)  (FT)    (n)
===  =====  =========  =================  ======  ===== ====== ===== =======
  1   30.0     20.0    0.018/0.018/0.020   0.67    2.00 0.0312 0.167 0.0150

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.00 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint =  6.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*
  *USER-SPECIFIED MINIMUM TOPOGRAPHIC SLOPE ADJUSTMENT NOT SELECTED

****************************************************************************
  FLOW PROCESS FROM NODE    110.00 TO NODE    111.00 IS CODE =  21
```

```
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
  >>USE TIME-OF-CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA<<
============================================================================
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   215.00
  ELEVATION DATA: UPSTREAM(FEET) =    164.50  DOWNSTREAM(FEET) =    160.50

  Tc = K*[(LENGTH** 3.00)/(ELEVATION CHANGE)]**0.20
  SUBAREA ANALYSIS USED MINIMUM Tc(MIN.) =    5.780
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  2.083
  SUBAREA Tc AND LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS   Tc
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN  (MIN.)
  COMMERCIAL                 C        0.15      0.25     0.100    69    5.78
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.100
  SUBAREA RUNOFF(CFS) =      0.28
  TOTAL AREA(ACRES) =      0.15   PEAK FLOW RATE(CFS) =      0.28

****************************************************************************
  FLOW PROCESS FROM NODE    111.00 TO NODE    112.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   160.50  DOWNSTREAM(FEET) =   159.10
  FLOW LENGTH(FEET) =   255.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   2.4 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   2.02
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       0.28
  PIPE TRAVEL TIME(MIN.) =   2.11    Tc(MIN.) =    7.89
  LONGEST FLOWPATH FROM NODE    110.00 TO NODE    112.00 =     470.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    112.00 TO NODE    112.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  MAINLINE Tc(MIN.) =    7.89
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.743
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  COMMERCIAL                 C        1.51      0.25     0.100    69
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.100
  SUBAREA AREA(ACRES) =    1.51      SUBAREA RUNOFF(CFS) =    2.33
  EFFECTIVE AREA(ACRES) =      1.66   AREA-AVERAGED Fm(INCH/HR) =  0.03
```

```
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.10
  TOTAL AREA(ACRES) =        1.7       PEAK FLOW RATE(CFS) =       2.57

****************************************************************************
  FLOW PROCESS FROM NODE    112.00 TO NODE    113.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   159.10  DOWNSTREAM(FEET) =   158.00
  FLOW LENGTH(FEET) =   225.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   7.4 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   3.72
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       2.57
  PIPE TRAVEL TIME(MIN.) =   1.01    Tc(MIN.) =    8.90
  LONGEST FLOWPATH FROM NODE    110.00 TO NODE    113.00 =     695.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    113.00 TO NODE    113.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  MAINLINE Tc(MIN.) =    8.90
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.626
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  COMMERCIAL                 C        0.25      0.25     0.100    69
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.100
  SUBAREA AREA(ACRES) =    0.25      SUBAREA RUNOFF(CFS) =    0.36
  EFFECTIVE AREA(ACRES) =      1.91   AREA-AVERAGED Fm(INCH/HR) =  0.03
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.10
  TOTAL AREA(ACRES) =        1.9       PEAK FLOW RATE(CFS) =       2.75

****************************************************************************
  FLOW PROCESS FROM NODE    113.00 TO NODE    113.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  MAINLINE Tc(MIN.) =    8.90
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.626
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  COMMERCIAL                 C        1.17      0.25     0.100    69
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.100
```

```
  SUBAREA AREA(ACRES) =    1.17      SUBAREA RUNOFF(CFS) =    1.69
  EFFECTIVE AREA(ACRES) =      3.08   AREA-AVERAGED Fm(INCH/HR) =  0.03
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.10
  TOTAL AREA(ACRES) =        3.1       PEAK FLOW RATE(CFS) =      4.44
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        3.1  TC(MIN.) =      8.90
  EFFECTIVE AREA(ACRES) =      3.08  AREA-AVERAGED Fm(INCH/HR)=  0.03
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap = 0.100
  PEAK FLOW RATE(CFS)   =       4.44
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```

# Water Quality Management Plan (WQMP)

## Project Name:

Our Lady of Peace - Residential

**Prepared for:**

**Roman Catholic Bishop of Orange**

**13280 Chapman Ave**

**Garden Grove, CA 92840**

**Prepared by:**


**Proactive Engineering Consultants**

**Engineer Amy Jaramillo Registration No. 94608**

**27051 Towne Centre Dr, Suite 270**

**Foothill Ranch, CA 92610**

**949-259-5708**


**Prepared on:**

**November 2025**

# Project Owner's Certification

| | | | |
|---|---|---|---|
| Permit/Application No. | 00955110-PCPU | Grading Permit No. | N/A |
| Tract/Parcel Map No. | N/A | Building Permit No. | N/A |
| CUP, SUP, and/or APN (Specify Lot Numbers if Portions of Tract) | | | N/A |

This Water Quality Management Plan (WQMP) has been prepared for Roman Catholic Bishop of Orange by Proactive Engineering. The WQMP is intended to comply with the requirements of the local NPDES Stormwater Program requiring the preparation of the plan.

The undersigned, while it owns the subject property, is responsible for the implementation of the provisions of this plan and will ensure that this plan is amended as appropriate to reflect up-to-date conditions on the site consistent with the current Orange County Drainage Area Management Plan (DAMP) and the intent of the non-point source NPDES Permit for Waste Discharge Requirements for the County of Orange, Orange County Flood Control District and the incorporated Cities of Orange County within the Santa Ana Region. Once the undersigned transfers its interest in the property, its successors-in-interest shall bear the aforementioned responsibility to implement and amend the WQMP. An appropriate number of approved and signed copies of this document shall be available on the subject site in perpetuity.

| **Owner:** | | | |
|---|---|---|---|
| Title | Deacon Javier H. Rodriguez | | |
| Company | Roman Catholic Bishop of Orange | | |
| Address | 13280 Chapman Ave Garden Grove, CA 92840 | | |
| Email | jrodriguez@rcbo.org | | |
| Telephone # | 714-282-3032 | | |
| Signature | | Date | |

# Contents

**Page No.**



## Attachments

# Section I Discretionary Permit(s) and Water Quality Conditions

Provide discretionary permit and water quality information. *Refer to Section 2.1 in the Technical Guidance Document (TGD) available from the Orange County Stormwater Program (ocwatersheds.com).*

| Project Infomation | | | |
|---|---|---|---|
| Permit/Application No. | 00955110-PCPU | Tract/Parcel Map No. | N/A |
| Additional Information/ Comments: | N/A | | |
| **Water Quality Conditions** | | | |
| Water Quality Conditions (list verbatim) | N/A | | |
| **Watershed-Based Plan Conditions** | | | |
| Provide applicable conditions from watershed - based plans including WIHMPs and TMDLS. | N/A | | |

# Section II Project Description

## II.1 Project Description

| Description of Proposed Project | | | | |
|---|---|---|---|---|
| Development Category (Verbatim from WQMP): | New development projects that create 10,000 square feet or more of impervious surface. This category includes commercial, industrial, residential housing subdivisions, mixed-use, and public projects on private or public property that falls under the planning and building authority or the Permittees. | | | |
| Project Area ($ft^2$): 133,294 | Number of Dwelling Units: 68 | | SIC Code: 6513 | |
| Narrative Project Description: | The proposed project replaces an existing church parking lot with 68 apartments, designed as 3 story homes in 3-4 unit buildings. | | | |
| Project Area | Pervious | | Impervious | |
| | Area (acres or sq ft) | Percentage | Area (acres or sq ft) | Percentage |
| Pre-Project Conditions | 1.81 acres | 50% | 1.82 acres | 50% |
| Post-Project Conditions | 0.48 acres | 16% | 2.44 acres | 84% |
| Drainage Patterns/Connections | On-site drainage will be picked up at the end of each alley with small catch basins/grate inlets. From there a storm drain pipe will run along the edge of the property (nearest to Roosevelt) and connect to the existing catch basin at the corner of Roosevelt and Remington.<br><br>From there it flows in a mainline RCP storm drain in Roosevelt that ranges from 24" to 48". This pipeline eventually outlets into Central Irvine Channel, then Peters Canyon Channel, into San Diego Creek and then into Newport Bay. | | | |

## II.2 Potential Stormwater Pollutants

| Pollutants of Concern | | | |
|---|---|---|---|
| Pollutant | Circle One: E=Expected to be of concern<br>N=Not Expected to be of concern | | Additional Information and Comments |
| Suspended-Solid/ Sediment | E | N | |
| Nutrients | E | N | |
| Heavy Metals | E | N | |
| Pathogens (Bacteria/Virus) | E | N | |
| Pesticides | E | N | |
| Oil and Grease | E | N | |
| Toxic Organic Compounds | E | N | |
| Trash and Debris | E | N | |

## II.3 Hydrologic Conditions of Concern

☐ No – Show map

☒ Yes – Describe applicable hydrologic conditions of concern below.

Streams downstream of the project are potentially susceptible to hydromodification, this includes parts of Peters Canyon Channel, San Diego Creek Channel and Newport Bay.

The 2 year 24 hour post development runoff volume is more than 5% greater than the existing condition 2 year 24 hour runoff volume.

## II.4 Post Development Drainage Characteristics

On-site drainage will be picked up at the end of each alley with small catch basins/grate inlets. From there a storm drain pipe will run along the edge of the property (nearest to Roosevelt) and connect to the existing catch basin at the corner of Roosevelt and Remington. The downstream flow path will remain unchanged.

## II.5 Property Ownership/Management

The property will be owned by the Roman Catholic Bishop of Orange.

# Section III Site Description

## III.1 Physical Setting

| | |
|---|---|
| Planning Area/ Community Name | Our Lady of Peace |
| Location/Address | 14010 Remington |
| | Irvine, CA 92620 |
| Land Use | Residential |
| Zoning | Medium-High Density Residential |
| Acreage | 2.92 |
| Predominant Soil Type | C |

## *III.2 Site Characteristics*

| | |
|---|---|
| *Precipitation Zone* | The Design Capture Storm Depth is 0.80" based on OCTGD Rainfall Zone Figure XVI-1. |
| *Topography* | The site is relatively flat with a general grade of 1-2% from northwest to southeast. |
| *Drainage Patterns/Connections* | The site generally drains from the west to the east. Today it sheet flows over much of a barren field. There are a series of on-site drainage inlets that connect to a small on-site storm drain system and eventually connect to the existing catch basin on Remington (at the corner of Remington and Roosevelt).<br>From there it flows in a mainline RCP storm drain in Roosevelt that ranges from 24" to 48". This pipeline eventually outlets into Central Irvine Channel, then Peters Canyon Channel, into San Diego Creek and then into Newport Bay.<br>Part of the project is a Parcel Map to subdivide the existing parcel into 2 separate pieces. The existing and proposed drainage patterns can be |

| | |
|---|---|
| | found in Attachment E hydrology maps. The majority of what will be the residential property flows to the same location, the catch basin at the corner of Roosevelt and Remington, in both the existing and proposed condition. There is a portion in the east in the existing condition that flows to the residential portion. For this reason the existing and proposed drainage areas are not exactly the same. |
| *Soil Type, Geology, and Infiltration Properties* | The soil is primarily clayey sand, silty sand and sandy clay. The site is underlain by Young Alluvial Fan Deposits (Qyf). |
| *Hydrogeologic (Groundwater) Conditions* | Groundwater was found at approximately 54' deep. Historic high groundwater is estimated at greater than 40'. |
| *Geotechnical Conditions (relevant to infiltration)* | Infiltration testing produced rates of 0.01 and 0.02 inches/hour without a factor of safety. Therefore, infiltration is not feasible. (See report in Attachment D) |
| *Off-Site Drainage* | N/A |
| *Utility and Infrastructure Information* | Sewer & water are served by IRWD. Public storm drain in the adjacent roadways is owned and maintained by the City of Irvine. |

# III.3 Watershed Description

| | |
|---|---|
| Receiving Waters | Peters Canyon Channel<br>San Diego Creek<br>Newport Bay |
| 303(d) Listed Impairments | **San Diego Creek Reach 1** (Benthic Community Effects, DDT, Indicator Bacteria, Malathion, Nutrients, Sedimentation/Siltation, Selenium, Toxaphene, Toxicity)<br>**Newport Bay Upper** (Chlordane, Copper, DDT, Indicator Bacteria, Malathion, Nutrients, PCBs, Sedimentation/Siltation, Toxicity<br>**Newport Bay Lower** (Chlordane, Copper, DDT, Indicator Bacteria, Nutrients, PBCs, Toxicity) |
| Applicable TMDLs | All waterbodies listed above are requirement status:<br>Category 5A - a TMDL is required, but not yet completed<br>Category 5B - being addressed by USEPA approved TMDL |
| Pollutants of Concern for the Project | Suspended-Solid/ Sediment<br>Nutrients<br>Pesticides<br>Oil and Grease<br>Toxic Organic Compounds<br>Trash and Debris |
| Environmentally Sensitive and Special Biological Significant Areas | N/A |

# Section IV Best Management Practices (BMPs)

## IV. 1 Project Performance Criteria

| | | | |
|---|---|---|---|
| (NOC Permit Area only) Is there an approved WIHMP or equivalent for the project area that includes more stringent LID feasibility criteria or if there are opportunities identified for implementing LID on regional or sub-regional basis? | | YES ☐ | NO ☒ |
| If yes, describe WIHMP feasibility criteria or regional/sub-regional LID opportunities. | N/A | | |

| Project Performance Criteria (continued) | |
|---|---|
| If HCOC exists, list applicable hydromodification control performance criteria (Section 7.II-2.4.2.2 in MWQMP) | • Post-development runoff volume for the two-year frequency storm does not exceed that of the predevelopment condition by more than five percent<br>• Time of concentration of post-development runoff for the two-year storm event is not less than that for the predevelopment condition by more than five percent |
| List applicable LID performance criteria (Section 7.II-2.4.3 from MWQMP) | • Priority Projects must infiltrate, harvest and use, evapotranspire, or biotreat/biofilter, the 85th percentile, 24-hour storm event (Design Capture Volume).<br>• A properly designed biotreatment system may only be considered if infiltration, harvest and use, and evapotranspiration (ET) cannot be feasibly implemented for the full design capture volume. In this case, infiltration, harvest and use, and ET practices must be implemented to the greatest extent feasible and biotreatment may be provided for the remaining design capture volume. |
| List applicable treatment control BMP performance criteria (Section 7.II-3.2.2 from MWQMP) | • If it is not feasible to meet LID performance criteria through retention and/or biotreatment provided on-site or at a sub-regional/regional scale, then treatment control BMPs shall be provided on-site or offsite prior to discharge to waters of the US. Sizing of treatment control BMP(s) shall be based on either the unmet volume after claiming applicable water quality credits, if appropriate (See Section 7.II-3.1 Water Quality Credits) and as calculated in TGD Appendix VI. If treatment control BMPs can treat all of the remaining unmet volume and have a medium to high effectiveness for reducing the primary POCs, the project is considered to be in compliance; a waiver application and participation in an alternative program is not required.<br>• If the cost of providing treatment control BMPs greatly outweighs the pollution control benefits they would provide, a waiver of treatment control and LID requirements can be requested and alternative compliance approaches must be used to fulfill the remaining unmet volume. |

| | |
|---|---|
| Calculate LID design storm capture volume for Project. | <u>LID Design Storm Capture Volume (DCV)</u><br><br>Defined as:<br>V = C × d × A × 43,560 sf/ac × 1/12 in/ft<br><br>Where:<br>V = runoff volume during the design storm event, cu-ft<br>C = runoff coefficient = (0.75 × imp + 0.15)<br>imp = impervious fraction of drainage area (ranges from 0 to 1)<br>d = storm depth (inches)<br>A = tributary area (acres)<br><br>The Design Capture Volume for the project is as follows:<br>85th percentile, 24-hr storm depth = d = 0.80 inches<br>Drainage Area = A = 2.92 acres<br>Imperviousness = imp = 0.84<br><br>V =2.92 ac × 0.80 inches × ((0.75 × 0.84) + 0.15) × 43,560 sf/ac × 1/12 in/ft<br>V = 6,614 cu-ft<br>*Note: Volume calculated is for reference only, LID BMP solution will be a flow based proprietary biotreatment solution. See Attachment B for calculations.* |

# IV.2. SITE DESIGN AND DRAINAGE PLAN

Where feasible, impervious area dispersion was utilized to allow for evapotranspiration and incidental infiltration of runoff, prior to discharging to the storm drain system.

Infiltration was considered as a BMP for the development but due to low measured rates the project is infeasible for infiltration per the Geotechnical Report in Attachment D

Rainwater harvesting and re-use was considered as a BMP. The total landscape area does not require enough demand to justify the use of harvesting BMPs. See Worksheet J in Attachment B.

Therefore, bio-treatment was selected as the proposed BMP to meet LID performance criteria. Runoff from the project will be collected and treated by modular wetland systems. Bio treatment unit details are included in Attachment C.

The entire site was treated as one drainage management area. One unit at the downstream end will treat the required flows and the peak flows will bypass the water quality treatment system. The WQMP site showing the BMP location and overall drainage pattern can be found in Attachment A.

DMA 1:

A=2.9 acres
Imp=0.84
C=0.78
Qdesign=0.52cfs

# IV.3 LID BMP SELECTION AND PROJECT CONFORMANCE ANALYSIS

## IV.3.1 Hydrologic Source Controls

| Name | Included? |
|---|---|
| Localized on-lot infiltration | ☐ |
| Impervious area dispersion (e.g. roof top disconnection) | ☐ |
| Street trees (canopy interception) | ☒ |
| Residential rain barrels (not actively managed) | ☐ |
| Green roofs/Brown roofs | ☐ |
| Blue roofs | ☐ |
| Impervious area reduction (e.g. permeable pavers, site design) | ☒ |
| Other: | ☐ |

**HSC-3 Street Trees**

Trees will be planted within the parking lot and central plaza area. These trees will intercept and provide some volume reduction benefits for the project.

**Impervious Area Reduction**

The central church plaza area will incorporate as much landscape and pervious area as possible to provide aesthetic and water quality benefits.

**NOTE:** DCV reduction credit has not been determined for these areas as their benefits are incidental and the areas not specifically designed to retain runoff.

## IV.3.2 Infiltration BMPs

| Name | Included? |
|---|---|
| Bioretention without underdrains | ☐ |
| Rain gardens | ☐ |
| Porous landscaping | ☐ |
| Infiltration planters | ☐ |
| Retention swales | ☐ |
| Infiltration trenches | ☐ |
| Infiltration basins | ☐ |
| Drywells | ☐ |
| Subsurface infiltration galleries | ☐ |
| French drains | ☐ |
| Permeable asphalt | ☐ |
| Permeable concrete | ☐ |
| Permeable concrete pavers | ☐ |
| Other: | ☐ |
| Other: | ☐ |

Per the Geotech report in Attachment D infiltration testing produced rates of 0.01 and 0.02 inches/hour without a factor of safety. Therefore, infiltration is not feasible.

## IV.3.3 Evapotranspiration, Rainwater Harvesting BMPs

| Name | Included? |
|---|---|
| All HSCs; *See Section IV.3.1* | ☐ |
| Surface-based infiltration BMPs | ☐ |
| Biotreatment BMPs | ☐ |
| Above-ground cisterns and basins | ☐ |
| Underground detention | ☐ |
| Other: | ☐ |
| Other: | ☐ |
| Other: | ☐ |

Due to the small landscape area onsite harvest and re-use is not practical or economical. Refer to Worksheet J in Attachment B.

## IV.3.4 Biotreatment BMPs

| Name | Included? |
|---|---|
| Bioretention with underdrains | ☐ |
| Stormwater planter boxes with underdrains | ☐ |
| Rain gardens with underdrains | ☐ |
| Constructed wetlands | ☐ |
| Vegetated swales | ☐ |
| Vegetated filter strips | ☐ |
| Proprietary vegetated biotreatment systems | ☒ |
| Wet extended detention basin | ☐ |
| Dry extended detention basins | ☐ |
| Other: | ☐ |
| Other: | ☐ |

The LID DCV criteria will be met by using a flow-based biotreatment BMP solution. One unit will treat the whole site's required flows. Peak flows will bypass the water quality system. See Attachment B for calculations and Attachment C for system details.

## IV.3.5 Hydromodification Control BMPs

| BMP Name | BMP Description |
|---|---|
| Oversized storm drain pipes | In order to mitigate the peak flow differential from the proposed condition down to the existing condition storm drain pipelines will be oversized. The additional pipe volume will serve as a detention system and an orifice/weir structure will be installed to outlet flows at a rate similar to the existing condition. |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |

## IV.3.8 Non-structural Source Control BMPs

Fill out non-structural source control check box forms or provide a brief narrative explaining if non-structural source controls were not used.

| Non-Structural Source Control BMPs | | | | |
|---|---|---|---|---|
| **Identifier** | **Name** | **Check One** | | **If not applicable, state brief reason** |
| | | **Included** | **Not Applicable** | |
| N1 | Education for Property Owners, Tenants and Occupants | ☒ | ☐ | |
| N2 | Activity Restrictions | ☒ | ☐ | |
| N3 | Common Area Landscape Management | ☒ | ☐ | |
| N4 | BMP Maintenance | ☒ | ☐ | |
| N5 | Title 22 CCR Compliance (How development will comply) | ☒ | ☒ | Not required for residential |
| N6 | Local Industrial Permit Compliance | ☐ | ☒ | Not an industrial project |
| N7 | Spill Contingency Plan | ☐ | ☒ | No proposed hazardous materials onsite |
| N8 | Underground Storage Tank Compliance | ☐ | ☒ | No underground storage |
| N9 | Hazardous Materials Disclosure Compliance | ☐ | ☒ | No hazardous material onsite |
| N10 | Uniform Fire Code Implementation | ☐ | ☒ | Proposed facility does not propose to store toxic or highly toxic compressed gas. |
| N11 | Common Area Litter Control | ☒ | ☐ | |
| N12 | Employee Training | ☒ | ☐ | |
| N13 | Housekeeping of Loading Docks | ☐ | ☒ | No loading docks |
| N14 | Common Area Catch Basin Inspection | ☒ | ☐ | |
| N15 | Street Sweeping Private Streets and Parking Lots | ☒ | ☐ | |
| N16 | Retail Gasoline Outlets | ☐ | ☒ | No retail gas |

### IV.3.9 Structural Source Control BMPs

Fill out structural source control check box forms or provide a brief narrative explaining if Structural source controls were not used.

| Structural Source Control BMPs | | | | |
|---|---|---|---|---|
| **Identifier** | **Name** | **Check One** | | **If not applicable, state brief reason** |
| | | **Included** | **Not Applicable** | |
| S1 | Provide storm drain system stenciling and signage | ☒ | ☐ | |
| S2 | Design and construct outdoor material storage areas to reduce pollution introduction | ☐ | ☒ | No storage areas proposed |
| S3 | Design and construct trash and waste storage areas to reduce pollution introduction | ☒ | ☐ | |
| S4 | Use efficient irrigation systems & landscape design, water conservation, smart controllers, and source control | ☒ | ☐ | |
| S5 | Protect slopes and channels and provide energy dissipation | ☐ | ☒ | No slopes/channels |
| | Incorporate requirements applicable to individual priority project categories (from SDRWQCB NPDES Permit) | ☐ | ☒ | Santa Ana Region |
| S6 | Dock areas | ☐ | ☒ | No docks |
| S7 | Maintenance bays | ☐ | ☒ | No bays |
| S8 | Vehicle wash areas | ☐ | ☒ | No wash areas |
| S9 | Outdoor processing areas | ☐ | ☒ | No outdoor processing |
| S10 | Equipment wash areas | ☐ | ☒ | No wash areas |
| S11 | Fueling areas | ☐ | ☒ | No fueling areas |
| S12 | Hillside landscaping | ☐ | ☒ | No hillsides |
| S13 | Wash water control for food preparation areas | ☐ | ☒ | No food prep |
| S14 | Community car wash racks | ☐ | ☒ | No car washes |

# IV.4 ALTERNATIVE COMPLIANCE PLAN (IF APPLICABLE)

## IV.4.1 Water Quality Credits

Determine if water quality credits are applicable for the project. *Refer to Section 3.1 of the Model WQMP for description of credits and Appendix VI of the TGD for calculation methods for applying water quality credits.*

| Description of Proposed Project | | |
|---|---|---|
| Project Types that Qualify for Water Quality Credits (Select all that apply): | | |
| ☐Redevelopment projects that reduce the overall impervious footprint of the project site. | ☐Brownfield redevelopment, meaning redevelopment, expansion, or reuse of real property which may be complicated by the presence or potential presence of hazardous substances, pollutants or contaminants, and which have the potential to contribute to adverse ground or surface WQ if not redeveloped. | ☐ Higher density development projects which include two distinct categories (credits can only be taken for one category): those with more than seven units per acre of development (lower credit allowance); vertical density developments, for example, those with a Floor to Area Ratio (FAR) of 2 or those having more than 18 units per acre (greater credit allowance). |
| ☐ Mixed use development, such as a combination of residential, commercial, industrial, office, institutional, or other land uses which incorporate design principles that can demonstrate environmental benefits that would not be realized through single use projects (e.g. reduced vehicle trip traffic with the potential to reduce sources of water or air pollution). | ☐ Transit-oriented developments, such as a mixed use residential or commercial area designed to maximize access to public transportation; similar to above criterion, but where the development center is within one half mile of a mass transit center (e.g. bus, rail, light rail or commuter train station). Such projects would not be able to take credit for both categories, but may have greater credit assigned | ☐ Redevelopment projects in an established historic district, historic preservation area, or similar significant city area including core City Center areas (to be defined through mapping). |

| | | | | |
|---|---|---|---|---|
| ☐Developments with dedication of undeveloped portions to parks, preservation areas and other pervious uses. | ☐ Developments in a city center area. | ☐ Developments in historic districts or historic preservation areas. | ☐ Live-work developments, a variety of developments designed to support residential and vocational needs together – similar to criteria to mixed use development; would not be able to take credit for both categories. | ☐In-fill projects, the conversion of empty lots and other underused spaces into more beneficially used spaces, such as residential or commercial areas. |

| | |
|---|---|
| Calculation of Water Quality Credits (if applicable) | N/A |

## IV.4.2 Alternative Compliance Plan Information

Describe an alternative compliance plan (if applicable). Include alternative compliance obligations (i.e., gallons, pounds) and describe proposed alternative compliance measures. *Refer to Section 7.II 3.0 in the WQMP.*

N/A

# Section V Inspection/Maintenance Responsibility for BMPs

Fill out information in table below. Prepare and attach an Operation and Maintenance Plan. Identify the mechanism through which BMPs will be maintained. Inspection and maintenance records must be kept for a minimum of five years for inspection by the regulatory agencies. *Refer to Section 7.II 4.0 in the Model WQMP.*

| BMP INSPECTION/MAINTENANCE | | | |
|---|---|---|---|
| **BMP** | **Responsible Party(s)** | **Inspection/Maintenance Activities Required** | **Minimum Frequency of Activities** |
| **HYDROLOGIC SOURCE CONTROL BMPs** | | | |
| HSC-3 Street Trees | Owner | Conduct general inspection and maintenance monthly per routine landscaping maintenance activities. Trim trees as needed. Conduct bi-annual tree health evaluations. | After significant storm events and monthly with landscaping maintenance |
| **BIOTREATMENT BMPs** | | | |
| Proprietary Biotreatment | Owner | Comply with "Operations & Maintenance" by manufacturer. Attached with Final WQMP.<br><br>Inspect unit for accumulated debris and sediment and plant health; remove trash; trim vegetation.<br><br>Scarify media to promote infiltration. Add additional mulch | Before and after major storm events. |
| | | Replace 3" to 6" of media layer. | 10 years |
| **NON-STRUCTURAL SOURCE CONTROL BMPs** | | | |
| N1 Education for Property Owners, Tenants and Occupants | Owner | Educational materials will be provided to the owner. Materials shall include those shown in Section VII of this WQMP. | At project completion |
| N2 Activity Restrictions | Owner | Owner will follow activity restrictions to protect surface water quality. | Ongoing |

| | | | |
|---|---|---|---|
| N3 Common Area Landscape Management | Owner | Maintenance shall be consistent with City requirements; any fertilizer and/or pesticide usages shall be consistent with City and manufacturer guidelines for use of fertilizers and pesticides. Maintenance includes mowing, weeding, and debris removal. Trimming, replanting and replacement of mulch shall be performed on an as-needed basis. Trimmings, clippings, and other waste shall be properly disposed of off-site in accordance with local regulations. Materials temporarily stockpiled during maintenance activities shall be placed away from water courses and drain inlets. | Monthly and as needed |
| N4 BMP Maintenance | Owner | Maintenance of BMPs implemented at the project site shall be performed at the frequency prescribed in this WQMP. Records of inspections and BMP maintenance shall be maintained by the City and documented with the WQMP. | Ongoing |
| N11 Common Area Litter control | Owner | Litter patrol, violations investigation, reporting and other litter control activities shall be performed by the owner as needed and in conjunction with maintenance activities for common areas. | Ongoing |
| N12 Employee Training | Owner | Owner till train all current and new employees on maintenance procedures for all site BMPs | Ongoing |
| N14 Common Area Catch Basin Inspection | Owner | Catch basin inlets, area drains, curb-and-gutter systems and other drainage systems shall be inspected prior to October 1st of each year and after large storm events. If necessary, drains shall be cleaned prior to any succeeding rain events. 80% of private facilities shall be inspected and cleaned annually, with 100% of facilities inspected and maintained within a 2-year period. | Annually |
| N15 Street Sweeping | Owner | Parking lots and drive aisles will be swept and kept clear of debris. | Monthly |

| **STRUCTURAL SOURCE CONTROL BMPs** | | | |
|---|---|---|---|
| S1 Provide storm drain system stencilling and signage | Owner | As a part of the final civil engineering drawings, it will be required by the owner to stencil on all of the project's catch basins, where applicable in paved areas, the words, "No Dumping - Drains to Ocean." Storm drain stencils shall be inspected for legibility, at minimum, once prior to the storm season, no later than October 1st each year. Those determined to be illegible will be re-stencilled as soon as possible. | Annually |
| S3 – Trash & Waste Storage Areas | Owner | Trash areas shall be inspected to insure no leaks or spills from the trash containers. Drop inlet will be inspected and filters cleaned out. | Monthly/Quarterly (before/after major rain events) |
| S4 Use efficient irrigation systems & landscape design, water conservation, smart controllers, and source control | Owner | In conjunction with routine maintenance activities, verify that landscape design continues to function properly by adjusting properly to eliminate overspray to hardscape areas, and to verify that irrigation timing and cycle lengths are adjusted in accordance with water demands, given time of year, weather, day or night time temperatures based on system specifications and local climate patterns. | Monthly |

# Section VI Site Plan and Drainage Plan

## VI.1 SITE PLAN AND DRAINAGE PLAN

See Attachment A.

Include a site plan and drainage plan sheet set containing the following minimum information:

- Project location
- Site boundary
- Land uses and land covers, as applicable
- Suitability/feasibility constraints
- Structural BMP locations
- Drainage delineations and flow information
- Drainage connections
- BMP details

# Section VII Educational Materials

Refer to the Orange County Stormwater Program (ocwatersheds.com) for a library of materials available. For the copy submitted to the Permittee, only attach the educational materials specifically applicable to the project. Other materials specific to the project may be included as well and must be attached.

| Education Materials | | | |
|---|---|---|---|
| **Residential Material (http://www.ocwatersheds.com)** | **Check If Applicable** | **Business Material (http://www.ocwatersheds.com)** | **Check If Applicable** |
| The Ocean Begins at Your Front Door | ☒ | Tips for the Automotive Industry | ☐ |
| Tips for Car Wash Fund-raisers | ☐ | Tips for Using Concrete and Mortar | ☐ |
| Tips for the Home Mechanic | ☐ | Tips for the Food Service Industry | ☐ |
| Homeowners Guide for Sustainable Water Use | ☒ | Proper Maintenance Practices for Your Business | ☐ |
| Household Tips | ☐ | **Other Material** | **Check If Attached** |
| Proper Disposal of Household Hazardous Waste | ☒ | | |
| Recycle at Your Local Used Oil Collection Center (North County) | ☐ | | ☐ |
| Recycle at Your Local Used Oil Collection Center (Central County) | ☒ | | ☐ |
| Recycle at Your Local Used Oil Collection Center (South County) | ☐ | | ☐ |
| Tips for Maintaining a Septic Tank System | ☐ | | ☐ |
| Responsible Pest Control | ☐ | | ☐ |
| Sewer Spill | ☐ | | ☐ |
| Tips for the Home Improvement Projects | ☒ | | ☐ |
| Tips for Horse Care | ☐ | | ☐ |
| Tips for Landscaping and Gardening | ☐ | | ☐ |
| Tips for Pet Care | ☐ | | ☐ |
| Tips for Pool Maintenance | ☐ | | ☐ |
| Tips for Residential Pool, Landscape and Hardscape Drains | ☐ | | ☐ |
| Tips for Projects Using Paint | ☒ | | ☐ |

# Attachment A: Site Plan

## LEGEND

- DRAINAGE BOUNDARY
- STORM DRAIN
- EX STORM DRAIN
- DMA-1 2.9 AREA
- PROPRIETARY BIOFILTRATION UNIT


30 15 0 30 60
SCALE: 1"=30'


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**OUR LADY OF PEACE - RESIDENTIAL-WQMP SITE PLAN**

11/10/2025

Plotted: Nov 10, 2025 – 8:33pm by: A_Jaramillo Drawing Name: N:\20.151.000\DLV\REPORTS\PWQMP-RES\OLP RES-WQMP SITE PLAN.dwg

# Attachment B: Worksheets and Calculations

### Worksheet D: Capture Efficiency Method for Flow-Based BMPs

| | | | | |
|---|---|---|---|---|
| ***Step 1: Determine the design capture storm depth used for calculating volume*** | | | | |
| 1 | Enter the time of concentration, $T_c$ (min) (See **Appendix** IV.2) | $T_c$= | 8.28 | |
| 2 | Using Figure III.4, determine the design intensity at which the estimated time of concentration ($T_c$) achieves 80% capture efficiency, $I_1$ | $I_1$= | 0.23 | in/hr |
| 3 | Enter the effect depth of provided HSCs upstream, $d_{HSC}$ (inches) (Worksheet A) | $d_{HSC}$= | 0 | inches |
| 4 | Enter capture efficiency corresponding to $d_{HSC}$, $Y_2$ (Worksheet A) | $Y_2$= | 0 | % |
| 5 | Using Figure III.4, determine the design intensity at which the time of concentration ($T_c$) achieves the upstream capture efficiency($Y_2$), $I_2$ | $I_2$= | 0 | |
| 6 | Determine the design intensity that must be provided by BMP, $I_{design}= I_1 - I_2$ | $I_{design}$= | 0.23 | |
| ***Step 2: Calculate the design flowrate*** | | | | |
| 1 | Enter Project area tributary to BMP (s), *A* (acres) | A= | 2.9 | acres |
| 2 | Enter Project Imperviousness, *imp* (unitless) | imp= | 0.84 | |
| 3 | Calculate runoff coefficient, $C = (0.75 \times imp) + 0.15$ | C= | 0.78 | |
| 4 | Calculate design flowrate, $Q_{design} = (C \times i_{design} \times A)$ | $Q_{design}$= | 0.52 | cfs |
| ***Supporting Calculations*** 0.78 x 0.23in/hr x 2.9ac x 43,560 sqft/ac x 1hr/60min x 1min/60sec x 1ft/12in = 0.52 cfs | | | | |

Describe system:

| MODEL # | DIMENSIONS | WETLANDMEDIA SURFACE AREA (sq. ft.) | TREATMENT FLOW RATE (cfs) |
|---|---|---|---|
| MWS-L-8-20 | 9' x 21' | 252 | 0.577 |

Provide time of concentration assumptions:

```
============================================================================
 END OF STUDY SUMMARY:
 TOTAL AREA(ACRES)      =        2.7  TC(MIN.) =     8.28
 EFFECTIVE AREA(ACRES) =      2.71  AREA-AVERAGED Fm(INCH/HR)=  0.05
 AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap = 0.200
 PEAK FLOW RATE(CFS)    =       4.05
============================================================================
============================================================================
 END OF RATIONAL METHOD ANALYSIS
```

**Figure III.4. Capture Efficiency Nomograph for Off-line Flow-based Systems in Orange County**

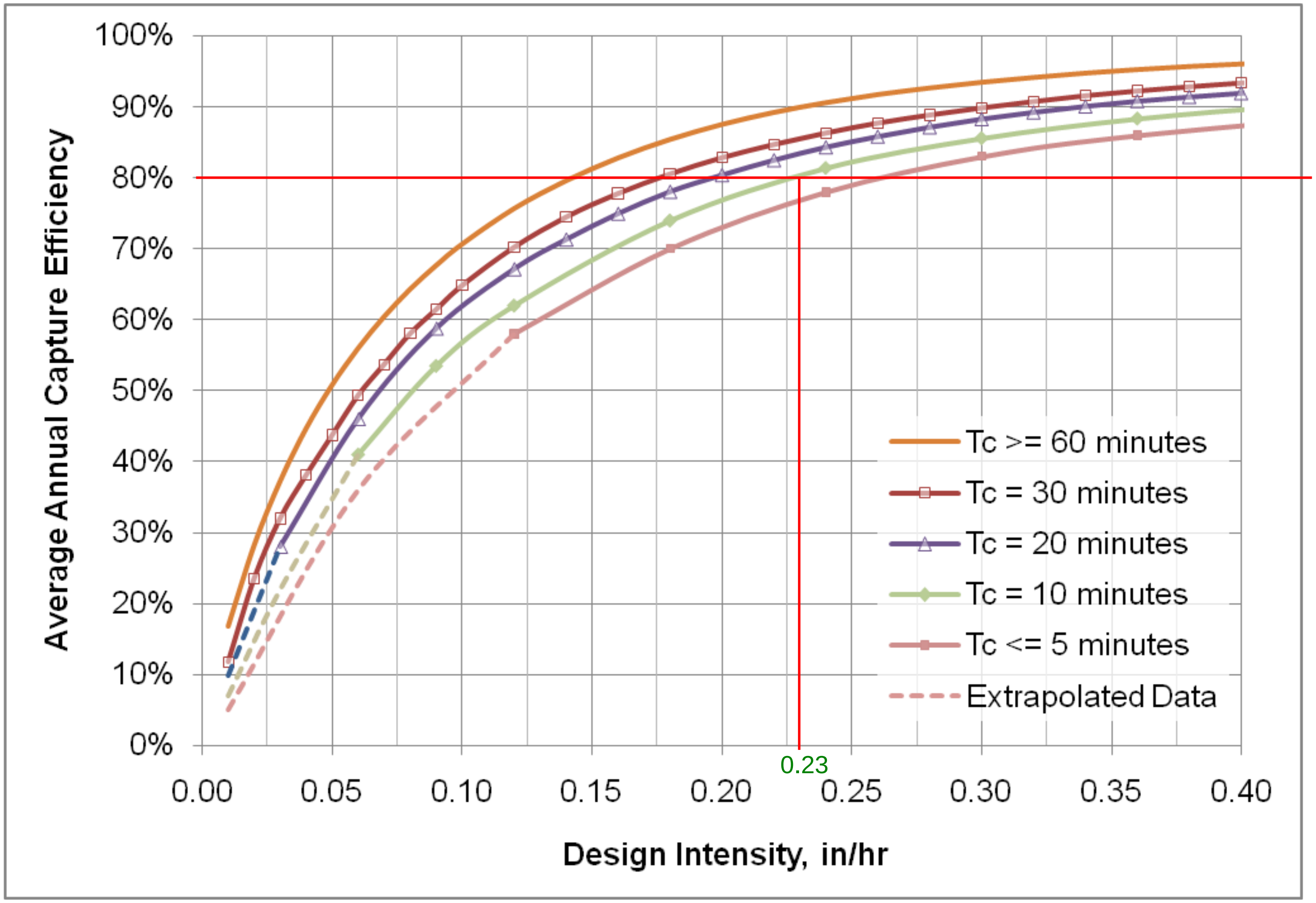


**Table X.8: Minimum Irrigated Area for Potential Partial Capture Feasibility**

| General Landscape Type | Conservation Design: $K_L$ = 0.35 | | | Active Turf Areas: $K_L$ = 0.7 | | |
|---|---|---|---|---|---|---|
| *Closest ET Station* | *Irvine* | *Santa Ana* | *Laguna* | *Irvine* | *Santa Ana* | *Laguna* |
| **Design Capture Storm Depth, inches** | **Minimum Required Irrigated Area per Tributary Impervious Acre for Potential Partial Capture, ac/ac** | | | | | |
| 0.60 | 0.66 | 0.68 | 0.72 | 0.33 | 0.34 | 0.36 |
| 0.65 | 0.72 | 0.73 | 0.78 | 0.36 | 0.37 | 0.39 |
| 0.70 | 0.77 | 0.79 | 0.84 | 0.39 | 0.39 | 0.42 |
| 0.75 | 0.83 | 0.84 | 0.90 | 0.41 | 0.42 | 0.45 |
| 0.80 | 0.88 | 0.90 | 0.96 | 0.44 | 0.45 | 0.48 |
| 0.85 | 0.93 | 0.95 | 1.02 | 0.47 | 0.48 | 0.51 |
| 0.90 | 0.99 | 1.01 | 1.08 | 0.49 | 0.51 | 0.54 |
| 0.95 | 1.04 | 1.07 | 1.14 | 0.52 | 0.53 | 0.57 |
| 1.00 | 1.10 | 1.12 | 1.20 | 0.55 | 0.56 | 0.60 |

**Worksheet J: Summary of Harvested Water Demand and Feasibility**

| | | | | |
|---|---|---|---|---|
| 1 | What demands for harvested water exist in the tributary area (check all that apply): | | | |
| 2 | Toilet and urinal flushing | | ☑ | |
| 3 | Landscape irrigation | | ☑ | |
| 4 | Other:________________________________ | | ☐ | |
| 5 | What is the design capture storm depth? (**Figure III.1**) | d | 0.8 | inches |
| 6 | What is the project size? | A | 2.92 | ac |
| 7 | What is the acreage of impervious area? | IA | 2.23 | ac |
| | For projects with multiple types of demand ( toilet flushing, indoor demand, and/or other demand) | | | |
| 8 | What is the minimum use required for partial capture? (**Table X.6**) | 650 | | gpd |
| 9 | What is the project estimated wet season total daily use? | Very low considering small landscape areas | | gpd |
| 10 | Is partial capture potentially feasible? (Line 9 > Line 8?) | No | | |
| | For projects with only toilet flushing demand | | | |
| 11 | What is the minimum TUTIA for partial capture? (**Table X.7**) | | | |
| 12 | What is the project estimated TUTIA? | | | |

**Worksheet J: Summary of Harvested Water Demand and Feasibility**

| | | | |
|---|---|---|---|
| 13 | Is partial capture potentially feasible? (Line 12 > Line 11?) | | |
| | For projects with only irrigation demand | | |
| 14 | What is the minimum irrigation area required based on conservation landscape design? ( **Table X.8**) | | ac |
| 15 | What is the proposed project irrigated area? (multiply conservation landscaping by 1; multiply active turf by 2) | | ac |
| 16 | Is partial capture potentially feasible? (Line 15 > Line 14?) | | |
| Provide supporting assumptions and citations for controlling demand calculation: | | | |

**Table X.6: Harvested Water Demand Thresholds for Minimum Partial Capture**

| Design Capture Storm Depth[1], inches | Wet Season Demand Required for Minimum Partial Capture[2], gpd per impervious acre |
|---|---|
| **0.60** | 490 |
| **0.65** | 530 |
| **0.70** | 570 |
| **0.75** | 610 |
| **0.80** | 650 |
| **0.85** | 690 |
| **0.90** | 730 |
| **0.95** | 770 |
| **1.00** | 810 |

1 - Based on isopluvial map (See **XVI.1**)

2 –Minimum Partial Capture is a performance standard whereby system performance exceeds 40 percent capture (See **Appendix** XIII) , such that the system must be considered for use even if it cannot achieve the full DCV.

# Attachment C: BMP Fact Sheets & Details

# BIO-5/BIO-7: PROPRIETARY BIOTREATMENT

**Category: Biotreatment with Partial Infiltration** (when accompanied by supplemental retention)

**Biotreatment with No Infiltration** (when used without supplemental retention)

Proprietary biotreatment BMPs are proprietary devices that are manufactured to treat stormwater. **Acceptance criteria for proprietary biotreatment BMPs are defined in Appendix J**. Proprietary BMPs that do not meet these acceptance criteria are not permitted. In addition, proprietary biotreatment BMPs must meet the definition of biofiltration in order to be used as LID biotreatment BMPs. There are two configurations of proprietary biotreatment, as explained in the following subsections.

**BIO-5: Proprietary Biotreatment with Enhanced Retention Configuration**

As standalone systems, proprietary biotreatment BMPs typically provide negligible volume reduction. To be used as a “biotreatment BMP with partial infiltration,” these BMPs must be accompanied by a retention compartment. This could consist of several options:

- Permeable pavement upstream of the proprietary BMPl
- Shallow infiltration gallery or chambers downstream of the BMP, connected to underdrains.
- Proprietary biotreatment downstream of a cistern for harvest and use.
- Use of adequate hydrologic source controls in the watershed to meet volume reduction targets (see Sizing section of this Fact Sheet).
- Other configurations that are determined to be appropriate to maximize the feasible volume reduction for the DMA.

Guidance for retention compartments is provided in other fact sheets, such as INF-5 (Permeable Pavement) and INF-6 (Underground Infiltration).

**BIO-7: Standard Configuration without Supplemental Retention**

For conditions that do not require partial infiltration, volume retention is not a performance goal. Acceptable proprietary biotreatment BMPs may be used as standalone systems. Guidance related to complementary retention can be disregarded.

## Pollutant Removal Considerations

BMPs that meet the acceptance criteria in **Appendix J** are considered to provide adequate treatment for pollutants of concern. According to these critera, there are different levels of treatment certification needed for different pollutants of concern.

## Recommended Design Criteria and Considerations

| Design Criteria | Intent/Rationale |
|---|---|
| ☐ Sediment sources should be controlled prior to operation of the system. | Proprietary systems are susceptible to clogging similar to other BMPs. Systems should not be used in areas that will continue to receive elevated sediment loading following construction, such as from open space area. |
| ☐ When accompanied by infiltration compartments, the ponding should not be higher than the underdrain elevation of the proprietary BMP. | This is intended to ensure that the complementary retention compartment does not reduce the hydraulic capacity of the proprietary biotreatment BMP. |
| ☐ When accompanied by infiltration compartments, these infiltration BMPs must adhere to siting guidance found in the respective fact sheet for the BMP | Specific siting considerations apply to infiltration BMPs. |
| ☐ Proprietary biotreatment systems typically do not require separate pretreatment | These BMPs typically include integrated mechanisms for pretreatment. |
| ☐ Proprietary BMPs must be designed in a manner consistent with manufacturer recommendations and consistent with the design configuration that was tested as part of the BMP certification | Proprietary devices have device-specific design, installation, and maintenance details which must be followed for proper treatment results. |
| ☐ In right of way areas, plant selection should not impair traffic sightlines or vehicle access. | Vegetation must not be prohibitive for typical vehicular movement and parking access needs. |
| ☐ Manufacturer guidance on vegetation selection and establishment should be followed | Manufacturers have experience with plant survival in specific climates for the BMP-specific conditions. |

## Calculations and Sizing Method

Proprietary Biotreatment BMPs are flow-based BMPs. See **Appendix E** for acceptable sizing methods.

Supplemental retention elements (for BIO-5 configuration) should be sized for one of the following targets, where possible:

- Approximately 40 percent long term volume reduction.
- Retention storage provided for approximately one-third of the DCV.
- Infiltration footprint (collective of all infiltrating elements of the project design) meeting target defined in **Section E.4.2**.

## Construction Guidance

| | Construction Guidance | Intent/Rationale |
|---|---|---|
| ☐ | Plans should include a construction sequence for the BMP. Revisions proposed by the contractor should be reviewed by the engineer. The construction sequence should address erosion control, utilities, BMP installation, inspections, testing and certifications, vegetation, stabilization, and post-construction monitoring. | Construction sequencing is critical to avoid issues/damage and allow appropriate inspections, testing, and certifications to be performed. |
| ☐ | Provide for inspection of buried infrastructure (e.g., underdrain, filter course) before it is buried. | It is impractical to inspect buried elements once they are covered. |
| ☐ | Fully stabilize sources of sediment within the tributary area (i.e., no exposed soil) prior to placing the finished BMP into service. | Sediment loading can seriously impair the capacity of the BMP. |
| ☐ | Allow plants and mulch to stabilize for as long as practicable (preferably several months) prior to placing the finished BMP into service. | Stabilization of the system allows plants to mature before stressing the system with stormwater loading. |

## O&M Activities and Frequencies

| Activity | Frequency |
|---|---|
| **GENERAL INSPECTIONS** | |
| Remove trash and debris | Four times per year during wet season, including inspection just before the wet season and within 24 hours after at least two storm events ≥ 0.5 inches. |
| Identify excess erosion or scour | |
| Identify sediment accumulation that requires maintenance | |
| Inspect during storm event, when possible, to estimate treatment capacity and determine if premature bypass is occurring | |
| Evaluate plant health and need for corrective action | |
| Identify any needed corrective maintenance that will require site-specific planning or design | |
| **OPERATION AND MAINTENANCE** | |
| • O&M of proprietary BMPs must follow established manufacturer guidelines<br>• O&M of accompanying retention BMPs should follow the guidelines established in the associated fact sheet for that BMP. | |

Bio Clean

A Forterra Company

# Modular Wetlands® System Linear

## A Stormwater Biofiltration Solution

# OVERVIEW

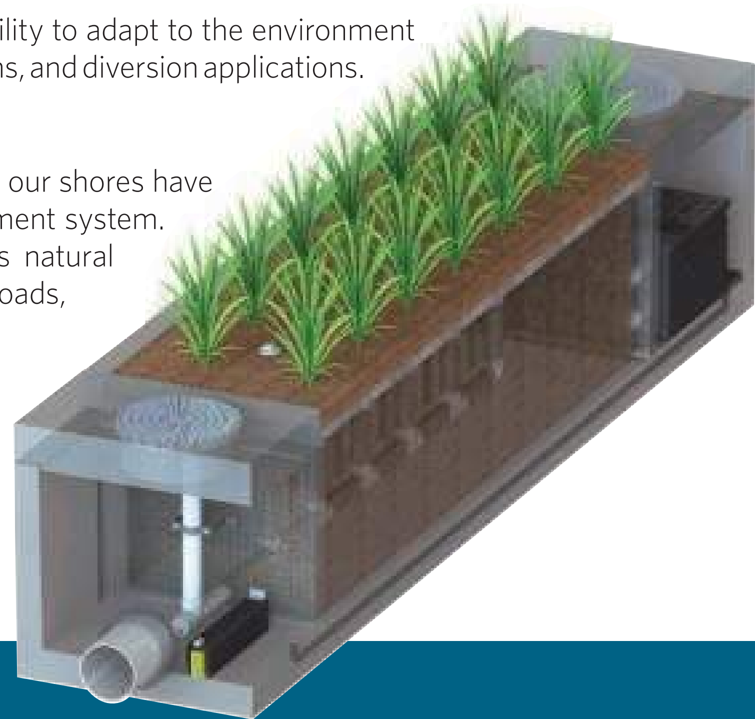
The Bio Clean Modular Wetlands® System Linear represents a pioneering breakthrough in stormwater technology as the only biofiltration system to utilize patented horizontal flow, allowing for a smaller footprint, higher treatment capacity, and a wide range of versatility. While most biofilters use little or no pretreatment, the Modular Wetlands® incorporates an advanced pretreatment chamber that includes separation and pre-filter cartridges. In this chamber, sediment and hydrocarbons are removed from runoff before entering the biofiltration chamber, reducing maintenance costs and improving performance.

Horizontal flow also gives the system the unique ability to adapt to the environment through a variety of configurations, bypass orientations, and diversion applications.

### The Urban Impact

For hundreds of years, natural wetlands surrounding our shores have played an integral role as nature's stormwater treatment system. But as cities grow and develop, our environment's natural filtration systems are blanketed with impervious roads, rooftops, and parking lots.

Bio Clean understands this loss and has spent years re-establishing nature's presence in urban areas, and rejuvenating waterways with the Modular Wetlands® System Linear.



## PERFORMANCE

The Modular Wetlands® continues to outperform other treatment methods with superior pollutant removal for TSS, heavy metals, nutrients, hydrocarbons, and bacteria. Since 2007 the Modular Wetlands® has been field tested on numerous sites across the country and is proven to effectively remove pollutants through a combination of physical, chemical, and biological filtration processes. In fact, the Modular Wetlands® harnesses some of the same biological processes found in natural wetlands in order to collect, transform, and remove even the most harmful pollutants.

| | | | | |
|---|---|---|---|---|
| 66% REMOVAL OF DISSOLVED ZINC | 69% REMOVAL OF TOTAL ZINC | 38% REMOVAL OF DISSOLVED COPPER | 64% REMOVAL OF TOTAL PHOSPHORUS | |
| 45% REMOVAL OF NITROGEN | 50% REMOVAL OF TOTAL COPPER | 95% REMOVAL OF MOTOR OIL | 67% REMOVAL OF ORTHO PHOSPHORUS | 85% REMOVAL OF TSS |

# APPROVALS

The Modular Wetlands® System Linear has successfully met years of challenging technical reviews and testing from some of the most prestigious and demanding agencies in the nation and perhaps the world. Here is a list of some of the most high-profile approvals, certifications, and verifications from around the country.

State of Washington TAPE GULD Basic Enhanced/Metals Phosphorus

**Washington State Department of Ecology TAPE Approved**
The MWS Linear is approved for General Use Level Designation (GULD) for Basic, Enhanced, and Phosphorus treatment at 1 gpm/ft$^2$ loading rate. The highest performing BMP on the market for all main pollutant categories.

CA

**California Water Resources Control Board, Full Capture Certification**
The Modular Wetlands® System is the first biofiltration system to receive certification as a full capture trash treatment control device.

VA

**Virginia Department of Environmental Quality, Assignment**
The Virginia Department of Environmental Quality assigned the MWS Linear the highest phosphorus removal rating for manufactured treatment devices to meet the new Virginia Stormwater Management Program (VSMP) regulation technical criteria.

MD

**Maryland Department of the Environment, Approved ESD**
Granted Environmental Site Design (ESD) status for new construction, redevelopment, and retrofitting when designed in accordance with the design manual.

MA

**MASTEP Evaluation**
The University of Massachusetts at Amherst – Water Resources Research Center issued a technical evaluation report noting removal rates up to 84% TSS, 70% total phosphorus, 68.5% total zinc, and more.

RI

**Rhode Island Department of Environmental Management, Approved BMP**
Approved as an authorized BMP and noted to achieve the following minimum removal efficiencies: 85% TSS, 60% pathogens, 30% total phosphorus, and 30% total nitrogen.

## ADVANTAGES

- HORIZONTAL FLOW BIOFILTRATION
- GREATER FILTER SURFACE AREA
- PRETREATMENT CHAMBER
- PATENTED PERIMETER VOID AREA
- FLOW CONTROL
- NO DEPRESSED PLANTER AREA
- AUTO DRAINDOWN MEANS NO MOSQUITO VECTOR

# OPERATION

The Modular Wetlands® System Linear is the most efficient and versatile biofiltration system on the market, and it is the only system with horizontal flow which:

- Improves performance
- Reduces footprint
- Minimizes maintenance

Figure 1 & Figure 2 illustrate the invaluable benefits of horizontal flow and the multiple treatment stages.

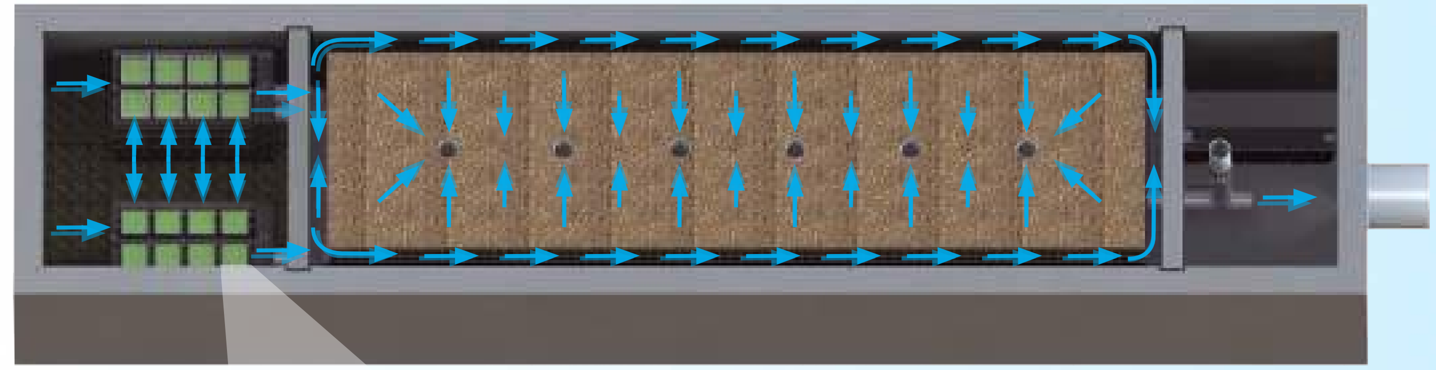
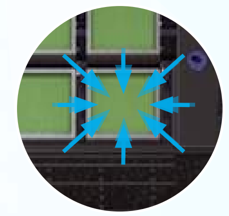


Figure 2, Top View



2x to 3x more surface area than traditional downward flow bioretention systems.

## 1 PRETREATMENT

### SEPARATION

- Trash, sediment, and debris are separated before entering the pre-filter cartridges
- Designed for easy maintenance access

### PRE-FILTER CARTRIDGES

- Over 25 sq. ft. of surface area per cartridge
- Utilizes BioMediaGREEN™ filter material
- Removes over 80% of TSS and 90% of hydrocarbons
- Prevents pollutants that cause clogging from migrating to the biofiltration chamber

## 2 BIOFILTRATION

### HORIZONTAL FLOW

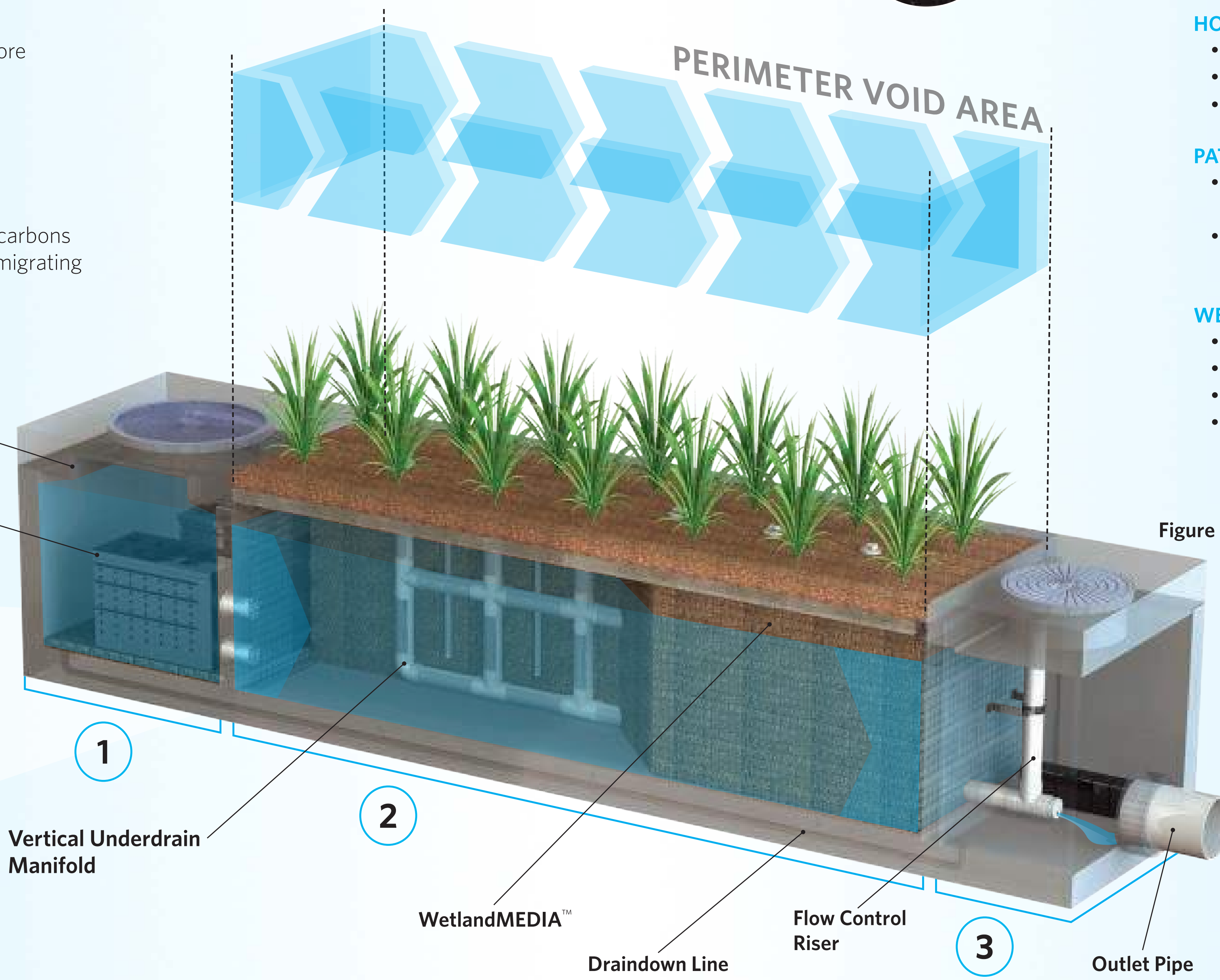
- Less clogging than downward flow biofilters
- Water flow is subsurface
- Improves biological filtration

### PATENTED PERIMETER VOID AREA

- Vertically extends void area between the walls and the WetlandMEDIA™ on all four sides
- Maximizes surface area of the media for higher treatment capacity

### WETLANDMEDIA

- Contains no organics and removes phosphorus
- Greater surface area and 48% void space
- Maximum evapotranspiration
- High ion exchange capacity and lightweight



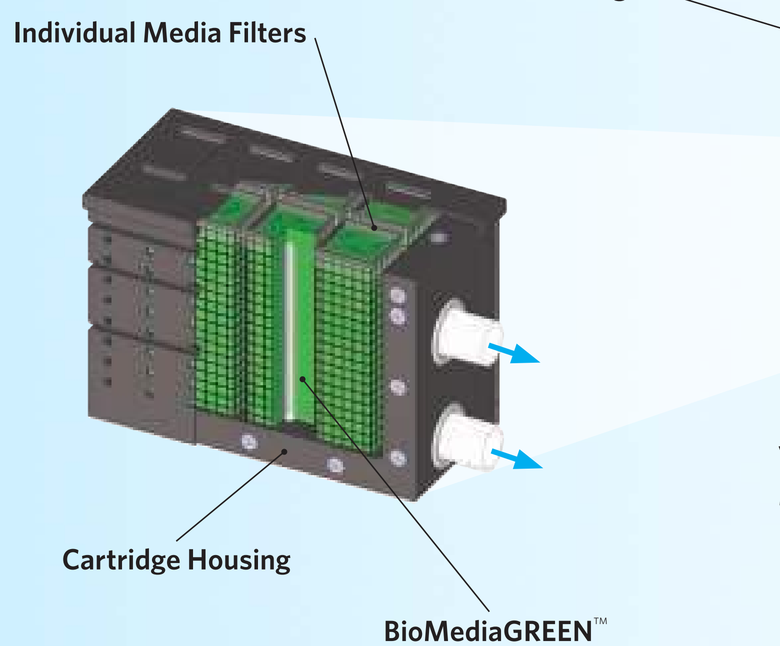




Figure 1

## 3 DISCHARGE

### FLOW CONTROL

- Orifice plate controls flow of water through WetlandMEDIA™ to a level lower than the media's capacity
- Extends the life of the media and improves performance

### DRAINDOWN FILTER

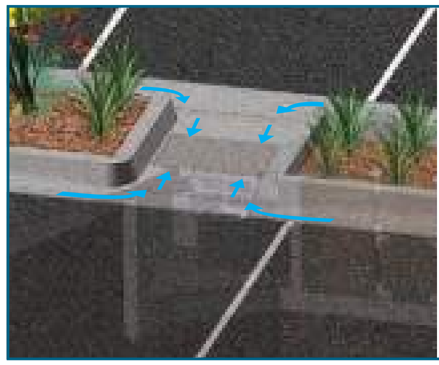
- The draindown is an optional feature that completely drains the pretreatment chamber
- Water that drains from the pretreatment chamber between storm events will be treated

# CONFIGURATIONS

The Modular Wetlands® System Linear is the preferred biofiltration system of civil engineers across the country due to its versatile design. This highly versatile system has available "pipe-in" options on most models, along with built-in curb or grated inlets for simple integration into your storm drain design.

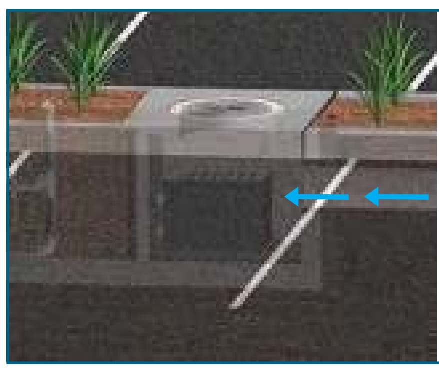
## CURB TYPE

The Curb Type configuration accepts sheet flow through a curb opening and is commonly used along roadways and parking lots. It can be used in sump or flow-by conditions. Length of curb opening varies based on model and size.



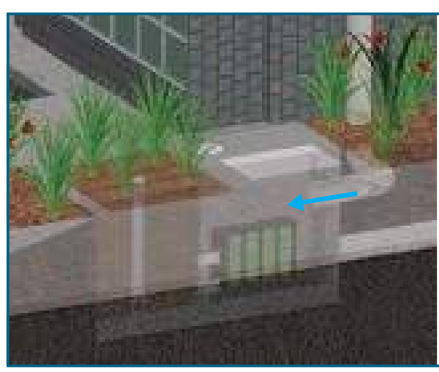
## GRATE TYPE

The Grate Type configuration offers the same features and benefits as the Curb Type but with a grated/drop inlet above the systems pretreatment chamber. It has the added benefit of allowing pedestrian access over the inlet. ADA-compliant grates are available to assure easy and safe access. The Grate Type can also be used in scenarios where runoff needs to be intercepted on both sides of landscape islands.



## VAULT TYPE

The system's patented horizontal flow biofilter is able to accept inflow pipes directly into the pretreatment chamber, meaning the Modular Wetlands® can be used in end-of-the-line installations. This greatly improves feasibility over typical decentralized designs that are required with other biofiltration/bioretention systems. Another benefit of the "pipe-in" design is the ability to install the system downstream of underground detention systems to meet water quality volume requirements.



## DOWNSPOUT TYPE

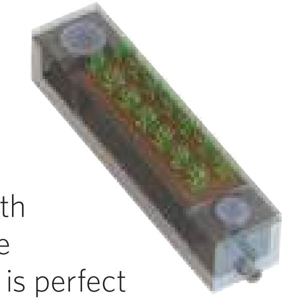
The Downspout Type is a variation of the Vault Type and is designed to accept a vertical downspout pipe from rooftop and podium areas. Some models have the option of utilizing an internal bypass, simplifying the overall design. The system can be installed as a raised planter, and the exterior can be stuccoed or covered with other finishes to match the look of adjacent buildings.

# ORIENTATIONS

### SIDE-BY-SIDE

The Side-By-Side orientation places the pretreatment and discharge chamber adjacent to one another with the biofiltration chamber running parallel on either side. This minimizes the system length, providing a highly compact footprint. It has been proven useful in situations such as streets with directly adjacent sidewalks, as half of the system can be placed under that sidewalk. This orientation also offers internal bypass options as discussed below.

### END-TO-END



The End-To-End orientation places the pretreatment and discharge chambers on opposite ends of the biofiltration chamber, therefore minimizing the width of the system to 5 ft. (outside dimension). This orientation is perfect for linear projects and street retrofits where existing utilities and sidewalks limit the amount of space available for installation. One limitation of this orientation is that bypass must be external.

# BYPASS

### INTERNAL BYPASS WEIR (SIDE-BY-SIDE ONLY)

The Side-By-Side orientation places the pretreatment and discharge chambers adjacent to one another allowing for integration of internal bypass. The wall between these chambers can act as a bypass weir when flows exceed the system's treatment capacity, thus allowing bypass from the pretreatment chamber directly to the discharge chamber.

### EXTERNAL DIVERSION WEIR STRUCTURE

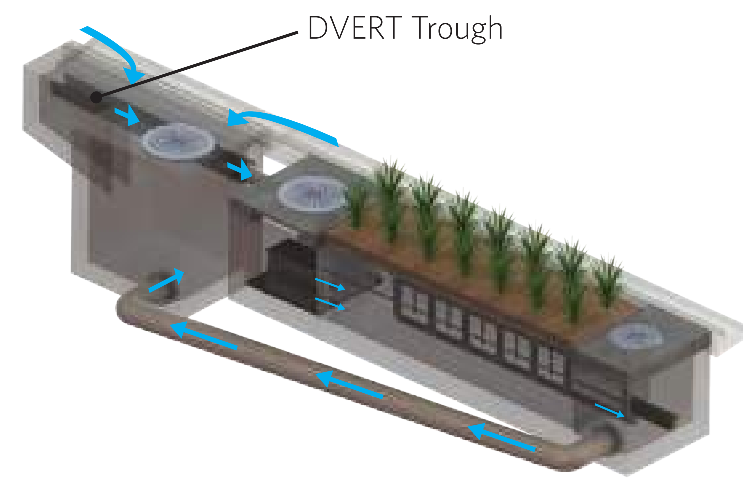
This traditional offline diversion method can be used with the Modular Wetlands® in scenarios where runoff is being piped to the system. These simple and effective structures are generally configured with two outflow pipes. The first is a smaller pipe on the upstream side of the diversion weir - to divert low flows over to the Modular Wetlands® for treatment. The second is the main pipe that receives water once the system has exceeded treatment capacity and water flows over the weir.

### FLOW-BY-DESIGN

This method is one in which the system is placed just upstream of a standard curb or grate inlet to intercept the first flush. Higher flows simply pass by the Modular Wetlands® and into the standard inlet downstream.

### DVERT LOW FLOW DIVERSION

This simple yet innovative diversion trough can be installed in existing or new curb and grate inlets to divert the first flush to the Modular Wetlands® via pipe. It works similar to a rain gutter and is installed just below the opening into the inlet. It captures the low flows and channels them over




to a connecting pipe exiting out the wall of the inlet and leading to the MWS Linear. The DVERT is perfect for retrofit and green street applications that allow the Modular Wetlands® to be installed anywhere space is available.

# SPECIFICATIONS

## FLOW-BASED DESIGNS

The Modular Wetlands® System Linear can be used in stand-alone applications to meet treatment flow requirements. Since the Modular Wetlands® is the only biofiltration system that can accept inflow pipes several feet below the surface, it can be used not only in decentralized design applications but also as a large central end-of-the-line application for maximum feasibility.

| MODEL # | DIMENSIONS | WETLANDMEDIA SURFACE AREA (sq. ft.) | TREATMENT FLOW RATE (cfs) |
|---|---|---|---|
| MWS-L-4-4 | 4' x 4' | 23 | 0.052 |
| MWS-L-4-6 | 4' x 6' | 32 | 0.073 |
| MWS-L-4-8 | 4' x 8' | 50 | 0.115 |
| MWS-L-4-13 | 4' x 13' | 63 | 0.144 |
| MWS-L-4-15 | 4' x 15' | 76 | 0.175 |
| MWS-L-4-17 | 4' x 17' | 90 | 0.206 |
| MWS-L-4-19 | 4' x 19' | 103 | 0.237 |
| MWS-L-4-21 | 4' x 21' | 117 | 0.268 |
| MWS-L-6-8 | 7' x 9' | 64 | 0.147 |
| MWS-L-8-8 | 8' x 8' | 100 | 0.230 |
| MWS-L-8-12 | 8' x 12' | 151 | 0.346 |
| MWS-L-8-16 | 8' x 16' | 201 | 0.462 |
| MWS-L-8-20 | 9' x 21' | 252 | 0.577 |
| MWS-L-8-24 | 9' x 25' | 302 | 0.693 |
| MWS-L-10-20 | 10' x 20' | 302 | 0.693 |

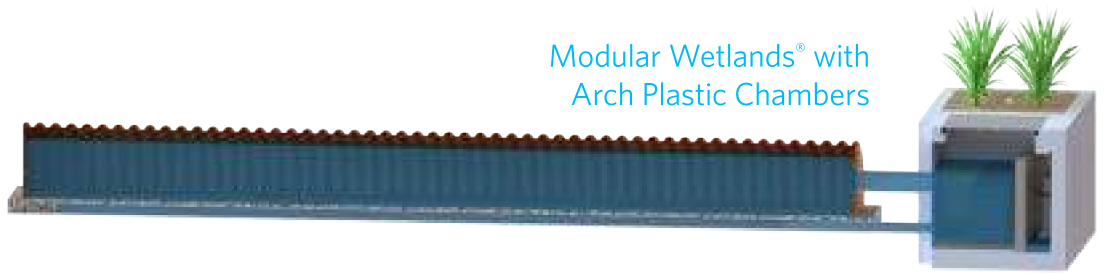
# VOLUME-BASED DESIGNS

## HORIZONTAL FLOW BIOFILTRATION ADVANTAGE

Modular Wetlands® with Box Culvert Prestorage

The Modular Wetlands® System Linear offers a unique advantage in the world of biofiltration due to its exclusive horizontal flow design: Volume-Based Design. No other biofilter has the ability to be placed downstream of detention ponds, extended dry detention basins, underground storage systems and permeable paver reservoirs. The systems horizontal flow configuration and built-in orifice control allows it to be installed with just 6" of fall between inlet and outlet pipe for a simple connection to projects with shallow downstream tie-in points. In the example above, the Modular Wetlands® is installed downstream of underground box culvert storage. Designed for the water quality volume, the Modular Wetlands® will treat and discharge the required volume within local draindown time requirements.

Modular Wetlands® with Arch Plastic Chambers



### DESIGN SUPPORT

Bio Clean engineers are trained to provide you with superior support for all volume sizing configurations throughout the country. Our vast knowledge of state and local regulations allow us to quickly and efficiently size a system to maximize feasibility. Volume control and hydromodification regulations are expanding the need to decrease the cost and size of your biofiltration system. Bio Clean will help you realize these cost savings with the Modular Wetlands®, the only biofilter than can be used downstream of storage BMPs.

## ADVANTAGES

- LOWER COST THAN FLOW-BASED DESIGN
- MEETS LID REQUIREMENTS
- BUILT-IN ORIFICE CONTROL STRUCTURE
- WORKS WITH DEEP INSTALLATIONS

# APPLICATIONS

The Modular Wetlands® System Linear has been successfully used on numerous new construction and retrofit projects. The system's superior versatility makes it beneficial for a wide range of stormwater and waste water applications - treating rooftops, streetscapes, parking lots, and industrial sites.

**INDUSTRIAL**
Many states enforce strict regulations for discharges from industrial sites. The Modular Wetlands® has helped various sites meet difficult EPA-mandated effluent limits for dissolved metals and other pollutants.

**RESIDENTIAL**
Low to high density developments can benefit from the versatile design of the Modular Wetlands®. The system can be used in both decentralized LID design and cost-effective end-of-the-line configurations.

**STREETS**
Street applications can be challenging due to limited space. The Modular Wetlands® is very adaptable, and it offers the smallest footprint to work around the constraints of existing utilities on retrofit projects.

**PARKING LOTS**
Parking lots are designed to maximize space and the Modular Wetlands'® 4 ft. standard planter width allows for easy integration into parking lot islands and other landscape medians.

**COMMERCIAL**
Compared to bioretention systems, the Modular Wetlands® can treat far more area in less space, meeting treatment and volume control requirements.

**MIXED USE**
The Modular Wetlands® can be installed as a raised planter to treat runoff from rooftops or patios, making it perfect for sustainable "live-work" spaces.

**More applications include:**
• Agriculture • Reuse • Low Impact Development • Waste Water

# PLANT SELECTION

Abundant plants, trees, and grasses bring value and an aesthetic benefit to any urban setting, but those in the Modular Wetlands® System Linear do even more - they increase pollutant removal. What's not seen, but very important, is that below grade, the stormwater runoff/flow is being subjected to nature's secret weapon: a dynamic physical, chemical, and biological process working to break down and remove non-point source pollutants. The flow rate is controlled in the Modular Wetlands®, giving the plants more contact time so that pollutants are more successfully decomposed, volatilized, and incorporated into the biomass of the Modular Wetlands'® micro/macro flora and fauna.

A wide range of plants are suitable for use in the Modular Wetlands®, but selections vary by location and climate. View suitable plants by visiting biocleanenvironmental.com/plants.

# INSTALLATION

The Modular Wetlands® is simple, easy to install, and has a space-efficient design that offers lower excavation and installation costs compared to traditional tree-box type systems. The structure of the system resembles precast catch basin or utility vaults and is installed in a similar fashion.

The system is delivered fully assembled for quick installation. Generally, the structure can be unloaded and set in place in 15 minutes. Our experienced team of field technicians is available to supervise installations and provide technical support.

# MAINTENANCE

Reduce your maintenance costs, man hours, and materials with the Modular Wetlands®. Unlike other biofiltration systems that provide no pretreatment, the Modular Wetlands® is a self-contained treatment train which incorporates simple and effective pretreatment.

Maintenance requirements for the biofilter itself are almost completely eliminated, as the pretreatment chamber removes and isolates trash, sediments, and hydrocarbons. What's left is the simple maintenance of an easily accessible pretreatment chamber that can be cleaned by hand or with a standard vac truck. Only periodic replacement of low-cost media in the pre-filter cartridges is required for long-term operation, and there is absolutely no need to replace expensive biofiltration media.

Bio Clean
A Forterra Company

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122018R1A

# Attachment D: Geotechnical Report

LGC Geotechnical, Inc.

May 28, 2024 | Project No. 23174-01

***Roman Catholic Bishop of Orange***
13280 Chapman Avenue
Garden Grove, CA 92840

***Subject:*** ***Preliminary Geotechnical Subsurface Evaluation, Proposed Residential Development, 14010 Remington, Irvine, California***

In accordance with your request, LGC Geotechnical, Inc. has performed a geotechnical subsurface evaluation for the proposed residential development located at 14010 Remington in the City of Irvine, California. This report summarizes the results of our background review, subsurface exploration, and geotechnical analyses of the data collected, and presents our findings, conclusions, and preliminary recommendations for the proposed residential project.

If you should have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Respectfully,

***LGC Geotechnical, Inc.***

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Distribution: (1) Addressee (electronic copy, wet signed copies can be provided upon request)

***TABLE OF CONTENTS***

## *LIST OF TABLES, ILLUSTRATIONS, & APPENDICES*

### *Tables*



### *Figures*



### *Appendices*

## *1.0 INTRODUCTION*

### *1.1 Purpose and Scope of Services*

This report presents the results of our geotechnical subsurface evaluation for the proposed residential development located at 14010 Remington in the City of Irvine, California (see Site Location Map, Figure 1). The purpose of our work was to collect subsurface data in order to confirm that the site can be developed from a geotechnical perspective. Our scope of services included:

- Review of pertinent readily available geotechnical information and geologic maps (Appendix A).
- Subsurface evaluation including excavation, sampling, and logging of six small-diameter hollow stem borings.
- Infiltration testing at two of the small-diameter hollow-stem borings.
- Laboratory testing of representative samples obtained during our subsurface evaluation (Appendix C).
- Geotechnical analysis and evaluation of the data obtained, including:
    - Suitability of the site for the proposed development from a geotechnical standpoint;
    - Description of the site geology, and subsurface soil and groundwater conditions;
    - Evaluation of the seismic conditions at the site, including seismic design criteria based on the 2022 California Building Code (CBC); and
    - Recommendations for remedial grading operations and site preparation.
- Preparation of this report presenting our findings, conclusions and recommendations with respect to the proposed site development.

### *1.2 Background/Project Description*

The approximately 2.9-acre site is bound to the north by an existing church, to the west by Remington, to the south by Roosevelt and east by a residential development. Review of historical aerial photographs suggests the site was undeveloped farmland prior to 1980. By 1985, Roosevelt was constructed but the site remained undeveloped until 2000. The church and associated parking lot was completed by 2002. The site has essentially remained unchanged since then.

The proposed development will consist of 68 multi-family residential units with associated improvements. We expect the proposed residential development will be at-grade with relatively light building loads (column and wall loads assumed to be a maximum of approximately 30 kips and 3 kips per lineal foot, respectively). At this time we do not have any design cut/fill information but we assume design cuts and fills will be less than 5 feet.

The recommendations provided herein are based upon the estimated structural loading, potential grade changes, and expected layout information above. We understand that the project plans are currently being developed at this time; LGC Geotechnical should be provided with updated project plans (including grading, foundation, retaining walls, etc.) and any changes to the assumed structural loads when they become available, in order to either confirm or modify the recommendations provided herein.

Note that an evaluation of the adjacent proposed church site was conducted simultaneously to the proposed residential site. The limits of these sites can be found in Figure 2 - Boring Location Map. The laboratory test results from the church evaluation were also included within this report.

### 1.3 ***Subsurface Evaluation***

LGC Geotechnical performed a subsurface geotechnical evaluation of the site consisting of the excavation of 6 hollow-stem auger borings.

Six hollow-stem borings (HS-1 through HS-4, I-1, and I-2) were drilled to depths ranging from approximately 10 to 50 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were excavated using a track-mounted drill rig equipped with 8-inch-diameter hollow-stem augers. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler generally obtained at 2.5 to 5-foot vertical increments. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch-tall brass rings. The SPT sampler and MCD sampler were driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. Bulk samples were also collected and logged at select depths for laboratory testing. At the completion of drilling, the borings were backfilled with the native soil cuttings and tamped. Some settlement of the backfill soils may occur over time.

The approximate locations of borings are shown on the Geotechnical Exploration Location Map (Figure 2). Boring logs are presented in Appendix B.

### 1.4 ***Field Percolation Testing***

Two falling head field percolation tests (I-1 and I-2) were performed in the approximate locations indicated on our Geotechnical Map (Figure 2). Estimation of infiltration rates for the site was accomplished in general accordance with the guidelines set forth by the County of Orange County (2013). A 3-inch diameter perforated PVC pipe with filter sock was placed in the borehole, and the annulus was backfilled with gravel, including placement of approximately 2 inches of gravel at the bottom of the borehole. The infiltration wells were pre-soaked the day prior to testing. During the pre-test, if the water level drops more than 6 inches in 25 minutes for two consecutive readings, the test procedure for coarse-grained soils was followed. If the water level did not drop 6 inches in one or both pre-test readings, the procedure for fine-grained soils was followed. The procedure for coarse-grained soils requires performing the test for one hour and taking one reading every 10 minutes from a fixed reference point. The procedure for fine-grained soils requires performing the test for six hours and taking one reading every 30 minutes from a fixed reference point.

The pre-tests indicated the procedure for the fine-grained soils should be followed. The calculated infiltration is normalized relative to the three-dimensional flow that occurs within the field test to a one-dimensional flow out of the bottom of the boring only (i.e., "Porchet

Method"). The measured infiltration rates (for feasibility purposes only) are provided in Table 1 below. Please note that infiltration is discussed in Section 4.8 and field data is provided in Appendix D.

Please note that the values provided in Table 1 do not include reduction factors associated with the test procedure, site variability, and long-term siltation plugging that are used to calculate the design infiltration rate.

***TABLE 1***

***Summary of Field Infiltration Testing***

| Infiltration Test No. | Approx. Depth Below Existing Grade (ft) | Measured Infiltration Rate* (in./hr.) |
|---|---|---|
| I-1 | 10 | 0.03 |
| I-2 | 10 | 0.01 |

*Measured Infiltration Rates Do Not Include Factor of Safety.

## *1.5 Laboratory Testing*

Laboratory testing was performed on representative soil samples obtained from our subsurface evaluation. Laboratory testing included in-situ moisture and density tests, expansion index, laboratory compaction, collapse and corrosion testing.

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 80 pounds per cubic foot (pcf) to 106 pcf, with an average of approximately 94 pcf. Field moisture contents ranged from approximately 1 percent to 24 percent, with an average of approximately 13 percent.
- Two samples were tested for fines content indicated a fines content (passing No. 200 sieve) ranging from 48 to 68 percent. According to the Unified Soils Classification System (USCS), the tested samples are classified as both "coarse and fine-grained" soil.
- Two Expansion Index (EI) tests were performed. The result indicated an EI value ranging from 112 to 135, corresponding to "High" to "Very High" expansion potential.
- Two laboratory compaction tests of near surface samples indicated a maximum dry density ranging from 115.0 to 117.5 with an optimum moisture content ranging from 12.5 to 14.0 percent.
- Six swell/collapse tests were performed. The plots are provided in Appendix C.
- Corrosion testing indicated soluble sulfate content of 95 parts per million (ppm), chloride content of 60 ppm, and pH value of 7.90, and minimum resistivity of 720 ohm-cm.

A summary of the results is presented in Appendix C. The moisture and dry density test results are presented on the boring logs in Appendix B.

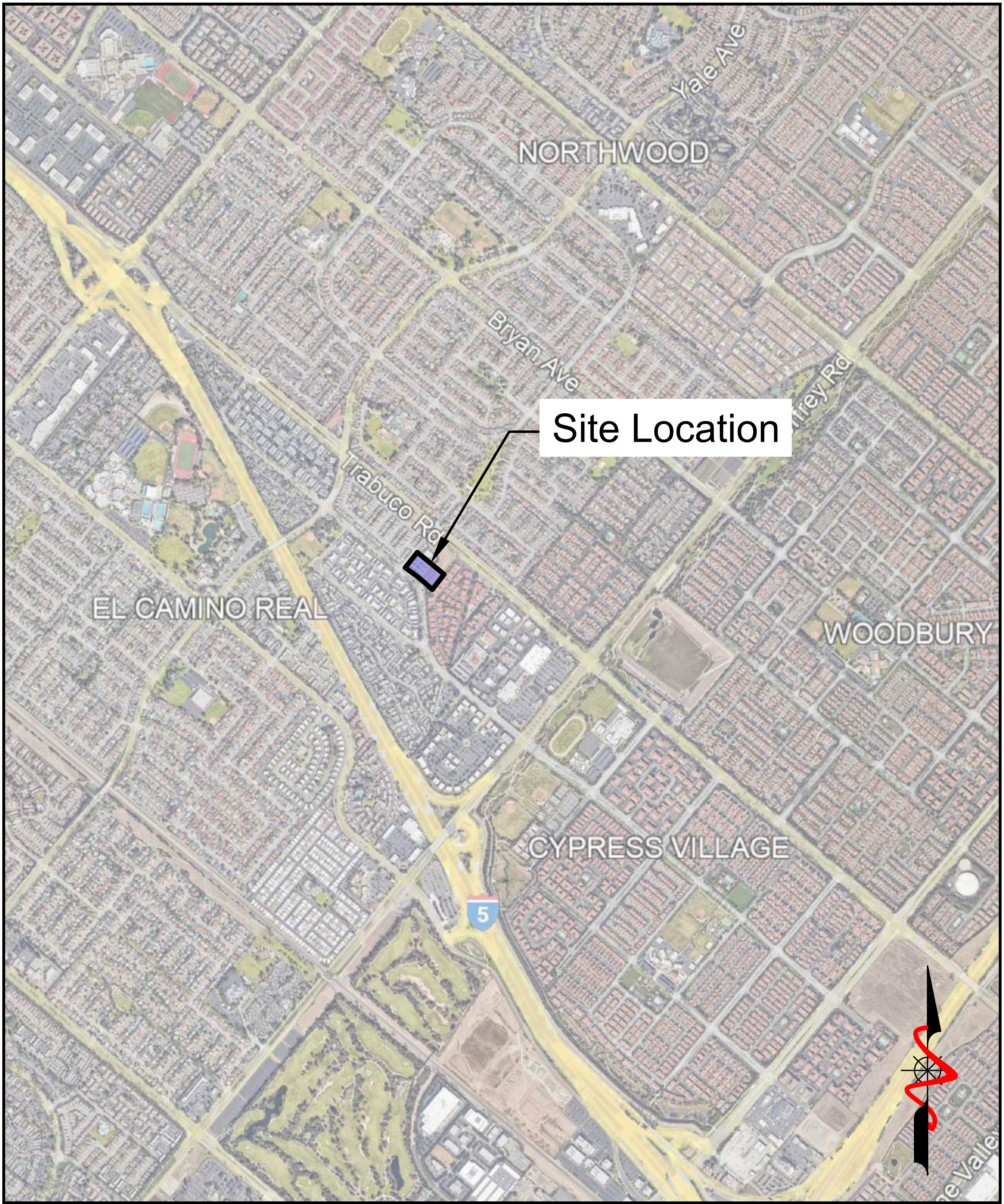


**FIGURE 1**
**Site Location Map**

LGC Geotechnical, Inc.

| | |
|---|---|
| PROJECT NAME | Our Lady of Peace, Irvine |
| PROJECT NO. | 23174-01 |
| ENG. / GEOL. | DJB |
| SCALE | Not to Scale |
| DATE | May 2024 |

## *2.0 GEOTECHNICAL CONDITIONS*

### *2.1 Regional Geology*

The subject site is located along the southeastern margin of the Los Angeles Basin, a large structural depression within the Peninsular Ranges geomorphic province of California. The southeastern portion of Los Angeles Basin includes the Tustin Plain, a shallow alluvial portion of the coastal plain. The site is located in the eastern portion of the Tustin Plain in the Irvine subbasin, generally composed of approximately 200 to 700 feet of unconsolidated to semi-consolidated Pliocene, Quaternary, and Holocene-age alluvial sediments. Soils within this zone consist predominantly of interbedded discontinuous lenses of clay, silt, sand and gravel.

Regional geologic mapping and local topographic expressions do not indicate the presence of large-scale landslides within or adjacent to the project area.

### *2.2 Site Geology and Generalized Subsurface Conditions*

Based on regional mapping (Morton, 2006) and our observations of the site, the subject site is underlain by Young Alluvial Fan Deposits (Qyf). In portions of the site, the young alluvial deposits are overlain by a thin veneer of older artificial fill that was placed during the development of the existing development. The depth of this older artificial fill was found to range from approximately 1 to 2 feet below existing grade during our evaluation but may be found at a deeper depth during construction. As indicated in our field explorations, site soils generally consisted of medium dense to dense clayey sands/silty sands and stiff to hard sandy clays.

It should be noted that borings are only representative of the location and time where/when they are performed, and varying subsurface conditions may exist outside of the performed location. In addition, subsurface conditions can change over time. The soil descriptions provided should not be construed to mean that the subsurface profile is uniform, and that soil is homogenous within the project area. Descriptions of the subsurface conditions are presented on the boring and geotechnical trench logs located in Appendix B.

### *2.3 Groundwater*

Groundwater was not encountered to the maximum explored depth of 51.5 feet below existing grade. Historic high groundwater is estimated to be greater than 40 feet below existing ground surface (CGS, 2001).

In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local seepage or during rainy seasons. Groundwater conditions below the site may be variable, depending on numerous factors including seasonal rainfall, local irrigation, and groundwater pumping, among others.

## *2.4 Faulting and Seismic Hazards*

Prompted by damaging earthquakes in California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. Their purpose was to prevent the construction of urban developments across the trace of active faults, resulting in the Alquist-Priolo Earthquake Fault Zoning Act. Earthquake Fault Zones have been delineated along the traces of active faults within California. Where developments for human occupation are proposed within these zones, the State requires detailed fault evaluations be performed so that engineering geologists can mitigate the hazards associated with active faulting by identifying the location of active faults and allowing for a setback from zones of previous ground rupture.

The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation. The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching, shallow ground rupture, soil liquefaction and dynamic settlement. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. Some of the major active nearby faults that could produce these secondary effects include the San Joaquin Hills, Whittier, Elsinore, Newport-Inglewood, among others (CGS, 2018). A discussion of these secondary effects is provided in the following sections.

### *2.4.1 Liquefaction and Dynamic Settlement*

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that loose, saturated, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils, and cohesive soils exhibit low to negligible liquefaction potential. In general, cohesive soils are not considered susceptible to liquefaction. Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Furthermore, dynamic settlement of dry sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CDMG, 2001), the subject site is not located within a liquefaction hazard zone. Based on our evaluation, site soils are generally not susceptible to liquefaction due to the fine-grained nature of the on-site soils and low probability of groundwater in the upper 50 feet. Therefore, liquefaction potential is considered low.

### 2.4.2 *Lateral Spreading*

Lateral spreading is a type of liquefaction induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move downslope towards a free face (such as a river channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Due to the site being relatively flat, low liquefaction potential and the lack of an adjacent "free face" to drive lateral spreading, the potential for lateral spreading is considered very low.

## 2.5 *Seismic Design Criteria*

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2022 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the CBC. Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16. Representative site coordinates of latitude 33.699559 degrees north and longitude -117.768908 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations ($S_{MS}$ and $S_{M1}$) and adjusted design spectral response acceleration parameters ($S_{DS}$ and $S_{D1}$) for Site Class D are provided in Table 2 on the following page. Since site soils are Site Class D, additional adjustments are required to code acceleration response spectrums as outlined below and provided in ASCE 7-16. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

***TABLE 2***

***Seismic Design Parameters***

| Selected Parameters from 2022 CBC, Section 1613 - Earthquake Loads | Seismic Design Values | Notes/Exceptions |
|---|---|---|
| Distance to applicable faults classifies the site as a “Near-Fault” site. | | Section 11.4.1 of ASCE 7 |
| Site Class | D* | Chapter 20 of ASCE 7 |
| Ss (Risk-Targeted Spectral Acceleration for Short Periods) | 1.257g | From SEAOC, 2024 |
| $S_1$ (Risk-Targeted Spectral Accelerations for 1-Second Periods) | 0.449g | From SEAOC, 2024 |
| $F_a$ (per Table 1613.2.3(1)) | 1.0 | For Simplified Design Procedure of Section 12.14 of ASCE 7, $F_a$ shall be taken as 1.4 (Section 12.14.8.1) |
| $F_v$ (per Table 1613.2.3(2)) | 1.851 | Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7 |
| $S_{MS}$ for Site Class D [Note: $S_{MS} = F_aS_S$] | 1.257g | - |
| $S_{M1}$ for Site Class D [Note: $S_{M1} = F_vS_1$] | 0.831g | Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7 |
| $S_{DS}$ for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$] | 0.838g | - |
| $S_{D1}$ for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$] | 0.554g | Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7 |
| $C_{RS}$ (Mapped Risk Coefficient at 0.2 sec) | 0.940 | ASCE 7 Chapter 22 |
| $C_{R1}$ (Mapped Risk Coefficient at 1 sec) | 0.932 | ASCE 7 Chapter 22 |
| *Since site soils are Site Class D and S1 is greater than or equal to 0.2, the seismic response coefficient Cs is determined by Eq. 12.8-2 for values of T ≤ 1.5Ts and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for TL ≥ T > Ts, or Eq. 12.8-4 for T > TL. Refer to ASCE 7-16. Site Class F modified to Site Class D, seismic parameters only applicable for structure period ≤ 0.5 second, refer to discussion above. | | |

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.55 at a distance of approximately 14.72 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period (Design Earthquake) indicates that an earthquake magnitude of 6.56 at a distance of approximately 21.38 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2022 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean ($MCE_G$) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The $PGA_M$ for the site is equal to 0.576g (SEAOC, 2024). The design PGA is equal to 0.384g (2/3 of $PGA_M$).

## 2.6 ***Oversized Material***

Oversized materials (material larger than 8 inches in maximum dimension) are not anticipated to be encountered during site grading based on our subsurface evaluation. If encountered, recommendations are provided for appropriate handling of oversized materials in Appendix E.

## 2.7 ***Expansion Potential***

Based on the results of laboratory testing from our evaluation and from nearby sites, finished grade soils are anticipated to have a "High to Very High" expansion potential. Final expansion potential of site soils should be determined at the completion of grading.

## 2.8 ***Soils Susceptible to Hydro-Collapse***

Soils that are typically susceptible to hydro-collapse (or collapsible soils) are predominately sand and silt held in a loose honeycomb structure. This relatively loose honeycomb structure is held together by small amounts clay or calcium carbonate acting as a temporary (soluble) cementing agent. If the soil remains dry the soil maintains its structure, however the addition of water to the soil will greatly weaken the honeycomb structure and the soil can subsequently experience immediate collapses. This collapse results in rapid soil settlement and potential damage to any improvements which are located without the zone of influence of the collapsible soils.

Laboratory testing for hydro-collapse is typically performed per American Society for Testing and Materials (ASTM) Test Method D5333 or ASTM D 2435. The two most common categorizations of hydro-collapse potential are ASTM D5333 and the Naval Facilities Engineering Command (NFEC, 1986) are shown in Table 3A and 3B below. As indicated in Table 3B soils with collapse potential above 5 percent are considered "trouble".

***TABLE 3A***

***Classification of Soil Collapsibility per ASTM D 5333***

| Collapse Index (Ie) | Collapse (%) | Collapse Potential |
|---|---|---|
| 0 | 0 | None |
| 0.001 - 0.02 | 0.1 - 2 | Slight |
| 0.021 - 0.60 | 2.1 - 6 | Moderate |
| 0.60 - 0.10 | 6 - 10 | Moderately Severe |
| > 0.10 | > 10 | Severe |

***TABLE 3B***

***Classification of Soil Collapsibility per NFEC, 1986***

| Collapse Potential (%) | Severity of Problem |
|---|---|
| 0-1 | No Problem |
| 1-5 | Moderate Trouble |
| 5-10 | Trouble |
| 10-20 | Severe Trouble |
| 20 | Very Severe Trouble |

A summary of the onsite soils with the most significant collapse potential are provided in Table 4 on the following page. The measured collapse potential is up to 7.5 percent, with three samples indicating collapse potential of 5 percent or greater.

***TABLE 4***

***Summary of Hydro-Collapse Laboratory Test Results***

| **Boring** | **Laboratory Measured Collapse (%)** |
|---|---|
| HS-2 @ 10 feet | 7.5 |
| HS-2 @ 15 feet | 1.7 |
| HS-3 @ 10 feet | 6.2 |
| HS-5 @ 10 feet | 5.1 |

## *3.0 CONCLUSIONS*

Based on the results of our subsurface geotechnical evaluation, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided that the recommendations contained in the following sections are incorporated during site development. A summary of our geotechnical conclusions are as follows:

- In general, borings excavated at the subject site indicate primarily native soils consisting of clayey sand, silty sand and sandy clay to the maximum explored depth of approximately 51.5 feet below existing grade. Native alluvial materials were found locally overlain by a relatively thin veneer of old artificial fill associated with previous development of the site. The near-surface loose and compressible soils are not suitable for the planned improvements in their present condition (refer to Section 4.1).
- Groundwater was not encountered to the maximum explored depth of 51.5 feet below existing grade. Historic high groundwater is estimated to be greater than 40 feet below existing ground surface (CGS, 2001).
- The subject study area is not located within a mapped State of California Earthquake Fault Zone, and based upon our review of published geologic mapping, no known active or potentially active faults are known to exist within or in the immediate vicinity of the site. Therefore, the potential for ground rupture as a result of faulting is considered very low.
- The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- The site is not located in a mapped liquefaction hazard zone. Based on our evaluation, site soils are generally not susceptible to liquefaction due to the fine-grained nature of the on-site soils and low probability of groundwater in the upper 50 feet. Therefore, liquefaction potential is considered low.
- Based on the results of preliminary laboratory testing, site soils are anticipated to have “High to Very High” expansion potential. Final design expansion potential must be determined at the completion of grading.
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order. We anticipate that soils generated from the excavations will be generally suitable for re-use as compacted fill, provided they are relatively free of rocks larger than 8 inches in dimension, construction debris, and significant organic material.
- Oversized materials (greater than 8 inches in maximum dimension) are not likely to be encountered during site grading. If encountered, recommendations are provided for appropriate handling of oversized materials in Appendix E.
- The onsite soils are not suitable for backfill of site retaining walls. Therefore, import of sandy soils meeting project recommendations will be required.
- Field testing resulted in a measured infiltration rate ranging from 0.01 to 0.03 inches per hour. The measured infiltration rates do not include a factor of safety. Discussion regarding infiltration is provided in Section 4.8.
- Some of the site soils are susceptible to hydro-collapse. To mitigate this condition, the soils most susceptible to collapse should be temporarily removed and recompacted as fill during the grading operations.
- Pre-soaking of the subgrade for building slabs (and flatwork) will be required due to site expansive soils. The duration and process varies greatly based on the chosen method and is also

dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 2 to 3 weeks).

## 4.0 RECOMMENDATIONS

The following recommendations are to be considered preliminary and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the owner.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2022 CBC requirements. With regard to the possible occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an "acceptable level." The "acceptable level" of risk is defined by the California Code of Regulations as "that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project" [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvement may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions but cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

The geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual as-graded conditions.

### 4.1 Site Earthwork

Rough grading shall include remedial earthwork grading and placement of engineered compacted fill to design grades. Geotechnical recommendations for precise grading and construction of the proposed new improvements will be provided, as necessary.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the 2022 CBC/City of Irvine requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede those included in Appendix E. The following recommendations may be revised within future grading plan review reports or based on the actual conditions encountered during site grading.

#### 4.1.1 Site Preparation

Prior to grading, areas to be developed should undergo complete demolition, removal of all vegetation, and clearing of pavements, existing utilities, foundation and slab elements from the site. Vegetation, debris from the previous land use and excessive organic material should be removed and properly disposed of offsite. Holes resulting from removals of buried obstructions, which extend below proposed remedial and/or finish grades, should be replaced with suitable compacted fill material. If the demolition contractor removes subsurface utilities below the proposed remedial grading depth we recommend the excavations either be left open until grading operations begin or properly compacted to the depth of remedial grading.

If cesspools or septic systems are encountered, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. Any encountered wells should be properly abandoned in accordance with regulatory requirements.

### *4.1.2 Removal Depths and Limits*

In order to provide a relatively uniform bearing condition for the planned improvements, we recommend the near-surface potentially compressible soils and soils highly susceptible to collapse be temporarily removed and recompacted as engineered fill. If any undocumented artificial fill is encountered within the influence of the proposed building pads, the soils should be removed to competent native materials.

In order to promote more uniform soil conditions, soils shall be temporarily removed and recompacted to a minimum depth of approximately 12 feet below existing grade or 3 feet below the bottom of proposed foundations, whichever is deeper. Additionally, existing undocumented fill and unsuitable topsoil encountered within the building footprints should be temporarily removed and recompacted as compacted fill. Where space is available, the envelope for removal and recompaction should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade or 5 feet beyond the edges of the proposed building improvements, whichever is larger.

Local conditions may be encountered during excavation that could require additional over-excavation beyond the above noted minimum in order to obtain an acceptable subgrade. The actual depths and lateral extents of grading will be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. Removal areas and areas to be over-excavated should be accurately staked in the field by the Project Surveyor.

### *4.1.3 Temporary Excavations*

CORRECT?

Temporary excavations should be performed in accordance with project plans, specifications, and applicable Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. Based on our field investigation, the majority of site soils are anticipated to be OSHA Type "B" soils (refer to the attached boring logs) with the potential for shallow groundwater. Sandy soils are present and should be considered susceptible to caving. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Where proposed improvements will be adjacent to property lines, the potential for impacting existing offsite improvements may be reduced by performing "ABC" slot cuts while performing earthwork removal and recompaction. "ABC" slot cuts are defined as excavations perpendicular to sensitive property boundaries that are divided into multiple "slots" of equal width. If slots are labeled A, B, C, A, B, C, etc., then all "A" slots can be excavated at the same time but must be backfilled before all "B" slots can be excavated, etc. Any given slot should be backfilled immediately with properly compacted fill to finish grade prior to excavation of the adjacent two slots. Please note some sands susceptible to caving are present at the site. Recommendations for slot cut dimensions should be evaluated during grading. Protection of the existing offsite improvements during grading is the responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a minimum distance equivalent to a 1:1(horizontal to vertical) projection from the bottom of the excavation or 5 feet, whichever is greater. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

It should be noted that any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation. If requested, temporary shoring parameters will be provided.

#### *4.1.4 __Removal Bottoms and Subgrade Preparation__*

In general, removal bottoms, over-excavation bottoms and areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition (generally within optimum and 2 percent above optimum moisture content), and re-compacted per project recommendations.

Removal bottoms, over-excavation bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement.

#### *4.1.5 __Material for Fill__*

From a geotechnical perspective, the onsite soils are generally considered suitable for use as general compacted fill, provided they are screened of significant organic materials, construction debris and any oversized material (8 inches in greatest dimension).

From a geotechnical viewpoint, import soils for general fill (i.e., non-retaining wall backfill) should consist of clean, granular soils of Medium expansion potential (expansion index 90 or less based on ASTM D4829). Import for retaining wall backfill should meet the criteria outlined in the paragraph below. Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of three working days prior to any planned importation.

Retaining wall backfill should consist of imported sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per ASTM Test Method D1140 (or ASTM D6913/D422) and a "Very Low" expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches in maximum dimension.

Aggregate base should conform to the requirements of Section 200-2 of the most recent version of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials and/or City of Irvine requirements.

The placement of demolition materials in compacted fill is acceptable from a geotechnical viewpoint provided the demolition material is broken up into pieces not larger than typically used for aggregate base (approximately 2 to 4 inches in maximum dimension) and well blended into fill soils with essentially not resulting voids. Demolition material placed in fills must be free of construction debris (wood, organics, etc.) and reinforcing steel. If you elect to incorporate asphalt concrete fragments into the fill materials, approval from an environmental viewpoint and/or local agency may be required and is not the purview of the geotechnical consultant. From our previous experience, we recommend that asphalt concrete fragments be limited to fill areas within planned street areas (i.e., not within building pad areas) or exported.

#### *4.1.6 Placement and Compaction of Fills*

Material to be placed as fill should be brought to near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM D1557). Moisture conditioning of site soils will be required in order to achieve adequate compaction. Drying and/or mixing the very moist soils will be required prior to reusing the materials in compacted fills. Soils are also present that will require additional moisture in order to achieve the required compaction.

The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in compacted thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing by LGC Geotechnical. Oversized material as previously defined should be removed from site fills.

During backfill of excavations, the fill should be properly benched into firm and competent soils of temporary backcut slopes as it is placed in lifts.

Aggregate base material should be compacted to a minimum of 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction per ASTM D1557 at near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content).

### 4.1.7 ***Trench and Retaining Wall Backfill and Compaction***

The onsite materials may generally be suitable as trench backfill provided the soils are screened of organic matter and rocks greater than 6 inches in diameter. Trench backfill should be compacted in uniform lifts (generally not exceeding 12 inches in compacted thickness) by mechanical means to at least 90 percent relative compaction (per ASTM Test Method D1557). A representative from LGC Geotechnical should observe and test the backfill to verify compliance with the project recommendations.

Some manufacturers have restrictions for the type of soil in contact with pipes. Manufacturer specifications for soils surrounding the pipe should be reviewed and approved by the pipe designer for use prior to construction. The use of open-graded rock as backfill must include means of reducing migration of fines of adjacent materials into the void spaces. This can be done by wrapping the open-graded rock in a filter fabric or introducing an excavatable flowable fill to the rock.

Compactive efforts must be made for all backfill soil types except for flowable fill. There must be room between the trench side walls and the springline of the pipe for mechanical compaction equipment to get adequate compaction of the soil under the haunches of the pipe. If mechanical compaction may result in damage to pipes, clean sand having a Sand Equivalent (SE) greater than or equal to 20 (per California Test Method [CTM] 217) can be used to shade the pipes. Sand backfill should be densified by jetting or flooding and then tamped to ensure adequate compaction. Sand grains should be from a natural source with rounded shape. Manufactured sand from crushed rock or recycled material is not suitable for jetting/flooding as the grains are typically angular in shape and do not densify well enough with these methods.

Retaining wall backfill should consist of approved imported sandy soils as outlined in preceding Section 4.1.5. The onsite soils are not suitable for backfill of site retaining walls based on their fines content and expansion potential. Therefore, import of sandy soils meeting project recommendations will be required. The limits of select sandy backfill should extend a minimum ½ the height of the retaining wall or the width of the heel (if applicable), whichever is greater, refer to Figure 3. Retaining wall backfill soils should be compacted in relatively uniform thin lifts to at least 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted.

A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project recommendations.

#### *4.1.8 Shrinkage and Subsidence*

Allowance in the earthwork volumes budget should be made for an estimated 5 to 15 percent reduction in volume of near-surface (upper approximate 5 feet) soils. It should be stressed that these values are only estimates and that an actual shrinkage factor would be extremely difficult to predetermine. The effective change in volume of onsite soils will depend primarily on the type of compaction equipment, method of compaction used onsite by the contractor, and accuracy of the topographic survey.

Due to the combined variability in topographic surveys, inability to precisely model the removals and variability of on-site near-surface conditions, it is our opinion that the site will not balance at the end of grading. If importing/exporting a large volume of soils is not considered feasible or economical, we recommend a balance area be designated onsite that can fluctuate up or down based on the actual volume of soil. We recommend a "balance" area that can accommodate on the order of 5 percent (plus or minus) of the total grading volume be considered.

### *4.2 Preliminary Foundation Recommendations*

Provided that the remedial grading recommendations provided herein are implemented, the site may be considered suitable for the support of the residential structures using a post-tensioned foundation system designed to resist the impacts of expansive soils. Site soils are anticipated to be of High to Very High expansion potential (EI of greater than 90 per ASTM D4829). However, this must be verified based on as-graded conditions. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of project plans (i.e., foundation, grading and site layout plans) as well as completion of earthwork. Recommended soil bearing and estimated static settlement are provided in Section 4.3.

#### *4.2.1 Provisional Post-Tensioned Foundation Design Parameters*

The geotechnical parameters provided herein may be used for post-tensioned slab foundations with a deepened perimeter footing or a post-tensioned mat slab. These parameters have been determined in general accordance with the Post-Tensioning Institute (PTI) Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, referenced in Chapter 18 of the 2022 CBC. In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural designer/architect. Other types of stiff slabs may be used in place of the CBC post-tensioned slab design provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the CBC/PTI method.

Our design parameters are based on our experience with similar projects, test results onsite, and the anticipated nature of the soil (with respect to expansion potential). Please note that implementation of our recommendations will not eliminate foundation movement (and related distress) should the moisture content of the subgrade soils fluctuate. It is the intent of these recommendations to help maintain the integrity of the proposed structures and reduce (not eliminate) movement, based upon the anticipated site soil conditions. Should future owners and/or property maintenance personnel not properly maintain the areas surrounding the foundation, for example by overwatering, then we anticipate for highly expansive soils the maximum differential movement of the perimeter of the foundation to the center of the foundation to be on the order of a couple of inches. Soils of lower expansion potential are anticipated to show less movement.

***TABLE 5***

***Preliminary Post-Tensioned Foundation Design Parameters***

| **Parameter** | **PT Slab with Perimeter Footing** | **PT Mat with Thicken ed Edge** |
|---|---|---|
| Expansion Potential | Very High | Very High |
| Thornthwaite Moisture Index | -20 | -20 |
| Constant Soil Suction | PF 3.9 | PF 3.9 |
| Center Lift<br>Edge moisture variation distance, $e_m$<br>Center lift, $y_m$ | <br>6.1 feet<br>1.0 inch | <br>6.1 feet<br>1.2 inch |
| Edge Lift<br>Edge moisture variation distance, $e_m$<br>Edge lift, $y_m$ | <br>3.4 feet<br>2.5 inches | <br>3.4 feet<br>2.5 inches |
| Modulus of Subgrade Reaction, k (assuming presoaking as indicated below) | 100 pci | 100 pci |
| Minimum perimeter footing/thickened edge embedment below finish grade | 30 inches | 6 inches |
| Minimum Slab Thickness | 6 inches[2] | 10 inches[2] |
| Presoak | 140% of Opt.<br>30 inches | 140% of Opt.<br>30 inches |

1. Expansion index is for preliminary design purposes. Further evaluation is needed at the completion of grading.
2. Recommendations for foundation reinforcement and slab thickness are ultimately the purview of the foundation engineer/structural engineer based upon geotechnical criteria and structural engineering considerations.
3. Recommendations for vapor retarders below slabs are also the purview of the foundation engineer/structural engineer and should be provided in accordance with applicable code requirements.

#### *4.2.2 Shallow Foundation Maintenance*

The geotechnical parameters provided herein assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage and adequately maintained so that ponding, which causes significant moisture changes below the foundation, does not occur. Our recommendations do not account for excessive irrigation and/or incorrect landscape design. Plants should only be provided with sufficient irrigation for life and not overwatered to saturate subgrade soils. Sunken planters placed adjacent to the foundation should either be designed with an efficient drainage system or liners to prevent moisture infiltration below the foundation. Some lifting of the perimeter foundation beam should be expected even with properly constructed planters.

In addition to the factors mentioned above, future owners/property management personnel should be made aware of the potential negative influences of trees and/or other large vegetation. Roots that extend near the vicinity of foundations can cause distress to foundations. Future owners (and the owner's landscape architect) should not plant trees/large shrubs closer to the foundations than a distance equal to half the mature height of the tree or 20 feet, whichever is more conservative unless specifically provided with root barriers to prevent root growth below the building foundation.

It is the owner's responsibility to perform periodic maintenance during hot and dry periods to ensure that adequate watering has been provided to keep soil from separating or pulling back from the foundation. Future owners and property management personnel should be informed and educated regarding the importance of maintaining a constant level of soil-moisture. The owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing potentially expansive soils to become too dry. Expansive soils can undergo shrinkage during drying, and swelling during the rainy winter season, or when irrigation is resumed. This can result in distress to building structures and hardscape improvements. The builder should provide these recommendations to future owners and property management personnel.

#### *4.2.3 Slab Underlayment Guidelines*

The following is for informational purposes only since slab underlayment (e.g., moisture retarder, sand or gravel layers for concrete curing and/or capillary break) is unrelated to the geotechnical performance of the foundation and thereby not the purview of the geotechnical consultant. Post-construction moisture migration should be expected below the foundation. The foundation engineer/architect should determine whether the use of a capillary break (sand or gravel layer), in conjunction with the vapor retarder, is necessary or required by code. Sand layer thickness and location (above and/or below vapor retarder) should also be determined by the foundation engineer/architect.

### *4.3 Soil Bearing and Lateral Resistance*

Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment and 150 psf for each additional foot of foundation width to a maximum value of 2,500 psf. A mat foundation a minimum of 6 inches below lowest adjacent grade may be designed for an allowable soil bearing pressure of 1,200 psf. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated are for total dead loads and frequently applied live loads and may be increased by ⅓ for short duration loading (i.e., wind or seismic loads).

In utilizing the above-mentioned allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be 1-inch or less. Differential settlement may be taken as half of the total settlement (i.e., ½-inch over a horizontal span of 40 feet).

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.25 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 200 psf per foot of depth (or pcf) to a maximum of 2,000 psf may be used for the sides of footings poured against properly compacted fill. Allowable passive pressure may be increased to 265 pcf (maximum of 2,650 psf) for short duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively. The structural designer should incorporate appropriate factors of safety and/or load factors in their design.

### *4.4 Lateral Earth Pressures and Retaining Wall Design Considerations*

The following may be used for design of site retaining walls. Lateral earth pressures are provided as equivalent fluid unit weights, in psf per foot of depth (or pcf). These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

The following lateral earth pressures are presented in Table 6 below, for approved imported free draining, clean granular (sandy) soils with a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D-421/422) and a "Very Low" expansion potential (EI of 20 or less per ASTM D4829). The site soils are not suitable for retaining wall backfill due to their fines content and expansion index. Import of soils meeting the criteria outlined above will need to be used for retaining wall backfill soil. The wall designer should clearly indicate on the retaining wall plans the required imported sandy soil backfill criteria. These preliminary findings should be confirmed during grading.

***TABLE 6***

***Lateral Earth Pressures – Approved Imported Sandy Soils***

| **Conditions** | **Equivalent Fluid Unit Weight (pcf)** | **Equivalent Fluid Unit Weight (pcf)** |
|---|---|---|
| | **Level Backfill** | **2:1 Sloped Backfill** |
| | **Approved Sandy Soils** | **Approved Sandy Soils** |
| Active | 35 | 55 |
| At-Rest | 55 | 70 |

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. This would include 90-degree corners of retaining walls. Such walls should be designed for "at-rest." The equivalent fluid pressure values assume free-draining conditions and a drainage system will be installed and maintained to prevent the build-up of hydrostatic pressures. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, saturation of near-surface (upper approximate 1-foot) soils in front of the retaining walls, the perforated subdrain pipe should be located as low as possible behind the retaining wall. The outlet pipe should be sloped to drain to a suitable outlet. In general, we do not recommend retaining wall outlet pipes be connected to area drains. If subdrains are connected to area drains, special care should be taken to maintain these drains. Typical conventional retaining wall drainage is shown on Figure 3. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Waterproofing and outlet systems are not the purview of the geotechnical consultant.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed retaining wall footing will surcharge the proposed retaining wall. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist a uniform lateral pressure of 85 pounds per square foot (psf) due to normal street vehicle traffic, if applicable. Uniform lateral surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.45 and 0.3 may be used for at-rest and active conditions, respectively. The retaining wall designer should contact the geotechnical consultant for any required geotechnical input in estimating surcharge loads.

If retaining walls greater than 6 feet in height are proposed, the retaining wall designer should contact the geotechnical engineer for specific seismic lateral earth pressure increments based on the configuration of the planned retaining wall structures.

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in

Section 4.3. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

### 4.5 *Corrosivity to Concrete and Metal*

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils to buried concrete and metal facilities. We therefore present the results of our testing with regard to corrosion for the use of the client and other consultants, as they determine necessary.

Corrosion testing of near-surface bulk samples indicated a soluble sulfate content value of 95 ppm (less than 0.01 percent), a chloride content of 60 ppm, pH of 7.90, and a minimum resistivity of 720 ohm-centimeters. Based on Caltrans Corrosion Guidelines (2021), soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 2,000 ppm (0.2 percent) or greater. Based on the test results, soils are not considered corrosive using Caltrans criteria. Note that based on minimum resistivity the soils are considered severely corrosive to metallic improvements. If improvements that may be susceptible to corrosion are proposed, it is recommended that further evaluation by a corrosion engineer be performed.

Based on our laboratory test results of representative site soil samples and our experience, onsite soils should be considered as having a severity categorization of “moderate” and are designated class “S1” per ACI 318, Table 19.3.1.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, Table 19.3.2.1 using the “S1” sulfate classification.

Laboratory testing may need to be performed at the completion of grading by the project corrosion engineer to further evaluate the as-graded soil corrosivity characteristics. Accordingly, revision of the corrosion potential may be needed, should future test results differ substantially from the conditions reported herein. The client and/or other members of the development team should consider this during the design and planning phase of the project and formulate an appropriate course of action.

### 4.6 *Preliminary Asphalt Concrete Pavement Sections*

For the purposes of these preliminary recommendations, we have selected a preliminary design R-value of 5 and calculated pavement sections for Traffic Indices of 5.5, 6.0 and 6.5. R-value testing of the street subgrade will need to be performed to confirm our preliminary testing results/assumptions once the streets have been graded to finish subgrade elevations and the final Traffic Index is determined by the Civil Engineer or City Engineer. We are not responsible for selecting a design Traffic Index. It is our understanding that the City of Irvine requires a minimum pavement section of 4.2 inches of asphalt over 6 inches of base.

***TABLE 7***

***Preliminary Pavement Sections***

| Assumed Traffic Index | 5.5 | 6.0 | 6.5 |
|---|---|---|---|
| R -Value Subgrade | 5 | 5 | 5 |
| AC Thickness | 4.2 inches | 4.2 inches | 5.0 inches |
| AB Thickness | 10.0 inches | 12.5 inches | 13.0 inches |

Due to anticipated construction traffic prior to the completion of the project, we recommend that the total thickness (base course and capping course) of AC be placed at essentially the same time. Construction traffic loading on only the base course of the AC will increase the potential for pavement distress. It should be noted that construction traffic such as concrete trucks will likely exceed traffic loading after completion of construction.

Increasing the thickness of asphalt or adding additional base material will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Earthwork recommendations regarding aggregate base and subgrade are provided in the previous Section "Site Earthwork" and the related sub-sections of this report.

## *4.7 Nonstructural Concrete Flatwork*

Nonstructural concrete flatwork (such as walkways, private drives, patio slabs, etc.) has a potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete may be designed in accordance with the minimum guidelines outlined in Table 8 on the following page. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

***TABLE 8***

***Nonstructural Concrete Flatwork Guidelines***

| | Community Sidewalks (≤6 feet wide) | Private Drives | Patios/Entryways / Walkways (adjacent to homes or flatwork >6 feet wide) | City Sidewalk Curb and Gutters |
|---|---|---|---|---|
| Minimum Thickness (in.) | 4 (nominal) | 5 (full) | 5 (full) | City/Agency Standard |
| Presaturation | 140% of optimum m/c to 12 inches | 140% of optimum m/c to 12 inches | 140% of optimum m/c to 24 inches | City/Agency Standard |
| Reinforcement | — | No. 3 at 24 inches on centers | No. 3 at 24 inches on centers | City/Agency Standard |
| Thickened Edge (in.) | — | 8 x 8 | — | City/Agency Standard |
| Crack Control Joints | Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness | Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness | Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness | City/Agency Standard |
| Maximum Joint Spacing | 5 feet | 10 feet or quarter cut whichever is closer | 6 feet | City/Agency Standard |

To reduce the potential for driveways to separate from the garage slab, the builder may elect to install dowels to tie these two elements together. Similarly, future homeowners should consider the use of dowels to connect flatwork to the foundation.

## *4.8 Subsurface Water Infiltration*

Recent regulatory changes have occurred that mandate that storm water be infiltrated below grade into subsurface soils rather than collected in a conventional storm drain system. Typically, a combination of methods are implemented to reduce surface water runoff and increase infiltration including; permeable pavements/pavers for roadways and walkways, directing surface water runoff to grass-lined swales, retention areas, and/or drywells, etc.

It should be noted that collecting and concentrating surface water for the purpose of intentionally infiltrating below grade, conflicts with the geotechnical engineering objective of directing surface water away from slopes, structures and other improvements. The geotechnical stability and integrity of a site is reliant upon appropriately handling surface water. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement. The results of our field infiltration testing indicate the observed 1-D infiltration rate for I-1 and I-2 (not including required factors of safety for design) was 0.01 to 0.03 inches per hour. The design infiltration rate is thereby equal to the Observed Infiltration Rate provided in Table 1 (inches per hour) divided by the design factor of safety. The design factor of safety must be a minimum of 2.0 but may be increased at the discretion of the design engineer (County of Orange, 2013). Per the County of Orange infiltration guidelines (2013), infiltration of stormwater is not required when the factored infiltration rate (measured infiltration rate with safety factors applied) is less than 0.3 inches per hour in the vicinity of the BMPs.

Based on the results of field infiltration testing indicating essentially zero infiltration and soils with high fines content (silts and clays), we strongly recommend against the intentional infiltration of stormwater into the subsurface soils.

### *4.9 Control of Surface Water and Drainage Control*

From a geotechnical perspective, we recommend that compacted finished grade soils adjacent to proposed structures be sloped away from the proposed structures and towards an approved drainage device or unobstructed swale. Drainage swales, wherever feasible, should not be constructed within 5 feet of buildings. Where lot and building geometry necessitates that drainage swales be routed closer than 5 feet to structural foundations, we recommend the use of area drains together with drainage swales. Drainage swales used in conjunction with area drains should be designed by the project civil engineer so that a properly constructed and maintained system will prevent ponding within 5 feet of the foundation. Code compliance of grades is not the purview of the geotechnical consultant.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Overwatering must be avoided.

### *4.10 Geotechnical Plan Review*

Project plans (grading, foundation, retaining wall, etc.) should be reviewed by this office prior to construction to verify that our geotechnical recommendations have been incorporated. Additional or modified geotechnical recommendations may be required based on the proposed layout.

### *4.11 Geotechnical Observation and Testing*

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC Geotechnical. Geotechnical observation and testing is required per Section 1705 of the 2022 California Building Code (CBC).

Geotechnical observation and/or testing should be performed by LGC Geotechnical at the following stages:

- During grading (removal bottoms, fill placement, etc.);
- During retaining wall backfill and compaction;
- During utility trench backfill and compaction;
- After presoaking building pad and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- Preparation of pavement subgrade and placement of aggregate base;
- After building and wall footing excavation and prior to placement of steel reinforcement and/or concrete; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

## *5.0 LIMITATIONS*

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is based on data obtained from limited observations of the site, which have been extrapolated to characterize the site. While the scope of services performed is considered suitable to adequately characterize the site geotechnical conditions relative to the proposed development, no practical evaluation can completely eliminate uncertainty regarding the anticipated geotechnical conditions in connection with a subject site. Variations may exist and conditions not observed or described in this report may be encountered during grading and construction.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants (at a minimum the civil engineer, structural engineer, landscape architect) and incorporated into their plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

The findings of this report are valid as of the present date. However, changes in the conditions of a site can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. The findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site. This report is intended exclusively for use by the client, any use of or reliance on this report by a third party shall be at such party's sole risk.

In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and modification.

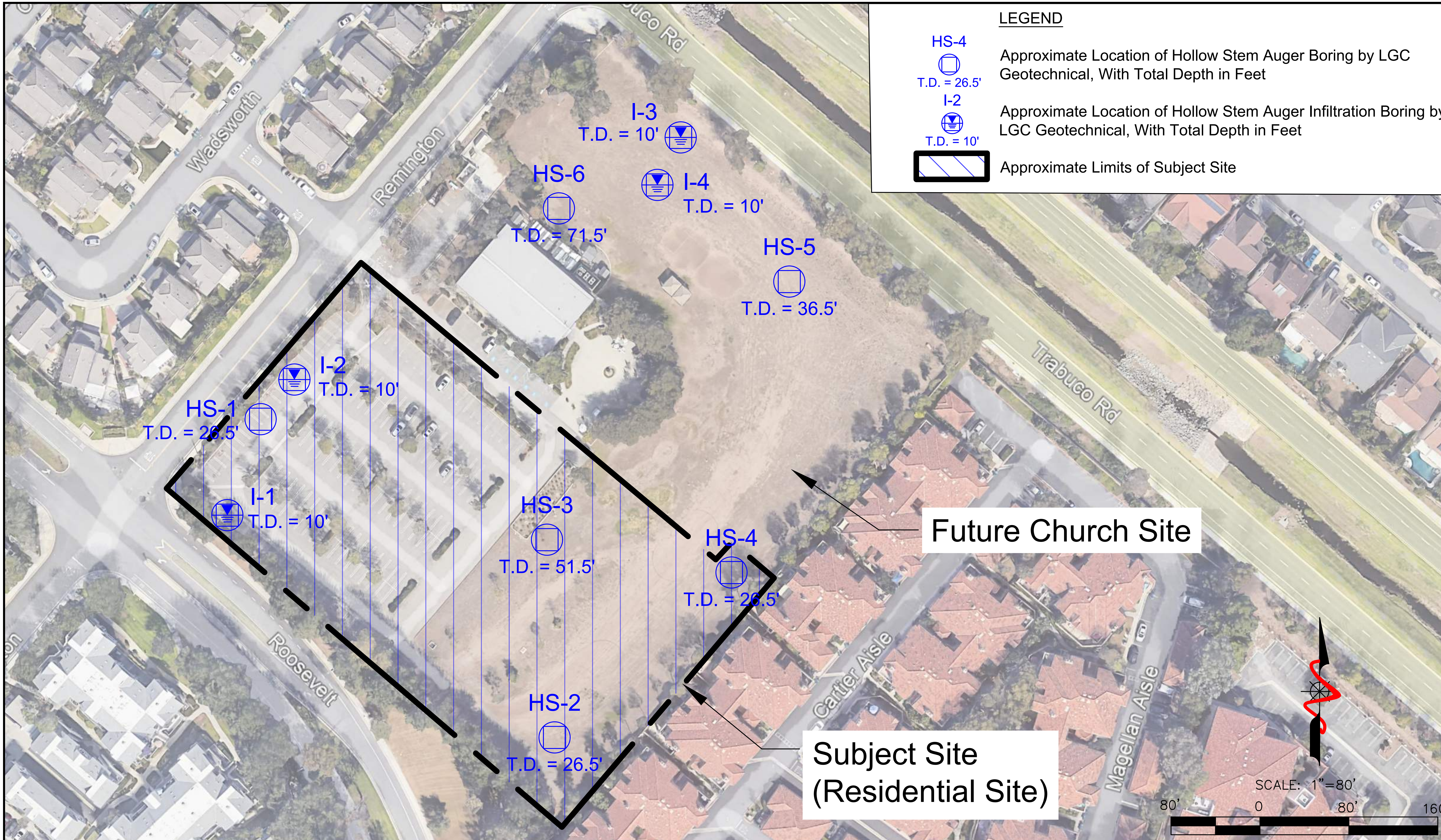


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***FIGURE 2***
***Boring Location Map***

| | |
|---|---|
| PROJECT NAME | Our Lady of Peace, Irvine |
| PROJECT NO. | 23174-01 |
| ENG. / GEOL. | DJB |
| SCALE | 1" = 80' |
| DATE | May 2024 |

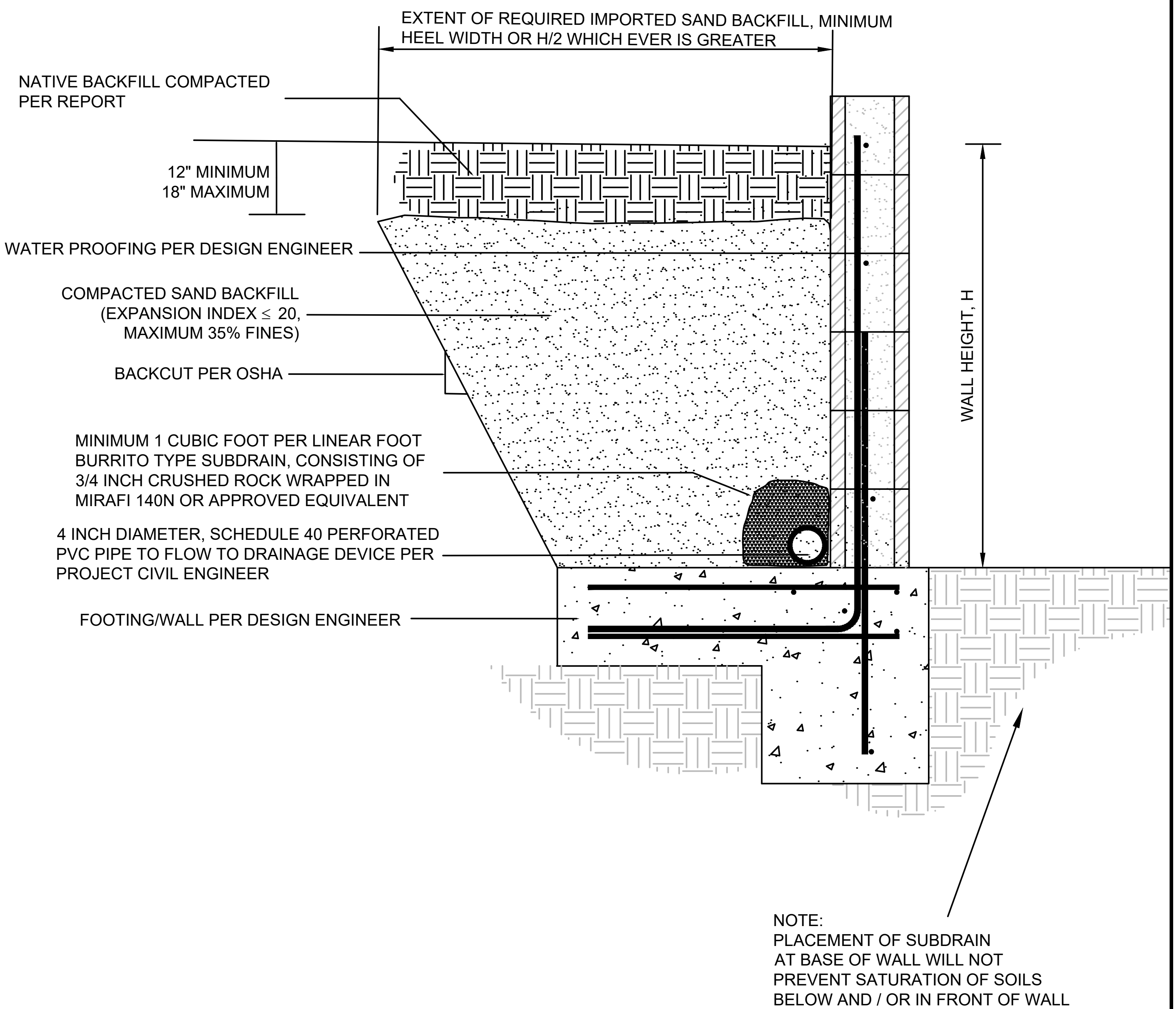


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**FIGURE 3**
**Retaining Wall Backfill Detail**

| | |
|---|---|
| PROJECT NAME | Our Lady of Peace, Irvine |
| PROJECT NO. | 23174-01 |
| ENG. / GEOL. | DJB |
| SCALE | Not to Scale |
| DATE | May 2024 |

# *Appendix A*
# *References*

***APPENDIX A***

***<u>References</u>***


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# *Appendix B*
# *Boring Logs*

# Geotechnical Boring Log Borehole HS-1

| | |
|---|---|
| **Date:** 2/22/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 8" |
| **Elevation of Top of Hole:** ~154' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 1 of 1 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | @ 0' - 3" of Asphalt over 7" of Base | |
| | | | | | | | | **Young Axial Channel Deposits (Qya):** | |
| | | B-1 | R-1 | 4 8 17 | 103.5 | 19.3 | CL | @ 2.5' - Sandy CLAY: brown, very moist, very stiff | |
| 150 | 5 | | R-2 | 10 13 14 | 95.8 | 15.7 | SC | @ 5' - Clayey SAND: brown, very moist, medium dense | |
| | | | R-3 | 16 24 25 | 91.5 | 16.1 | | @ 7.5' - Clayey SAND: brown, very moist, dense | |
| 145 | 10 | | R-4 | 9 10 10 | 90.1 | 23.0 | | @ 10' - Clayey SAND: brown, very moist, medium dense | |
| 140 | 15 | | SPT-1 | 5 11 14 | | 10.3 | | @ 15' - Clayey SAND: brown, moist, dense | |
| 135 | 20 | | R-5 | 10 14 14 | 101.9 | 13.6 | | @ 20' - Clayey SAND: reddish brown, very moist, medium dense | |
| 130 | 25 | | SPT-2 | 4 6 8 | | 18.0 | | @ 25' - Clayey SAND: brown, very moist, medium dense | |
| 125 | 30 | | | | | | | Total Depth = 26.5'<br>No Groundwater Encountered<br>Backfilled with Cuttings and Capped with Cold Patch on 2/22/24 | |

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

**SAMPLE TYPES:**
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

**TEST TYPES:**
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 10/20/2022

# Geotechnical Boring Log Borehole HS-2

**Date:** 2/22/24
**Drilling Company:** 2R Drilling
**Project Name:** Our Lady of Peace, Irvine
**Type of Rig:** Limited Access Rig
**Project Number:** 23174-01
**Drop:** 30" **Hole Diameter:** 8"
**Elevation of Top of Hole:** ~158' MSL
**Drive Weight:** 140 pounds
**Hole Location:** See Geotechnical Map



Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | @0' - Grass | |
| | | | | | | | | **Young Axial Channel Deposits (Qya):** | |
| 155 | | B-1 | R-1 | 4 10 24 | 99.8 | 19.0 | CL | @ 2.5' - Sandy CLAY: brown, very moist, very stiff | |
| | 5 | | R-2 | 16 20 20 | 92.7 | 11.8 | SC | @ 5' - Clayey SAND: brown, moist, dense | |
| 150 | | | R-3 | 16 20 23 | 94.7 | 10.2 | | @ 7.5' - Clayey SAND: brown, moist, dense | |
| | 10 | | R-4 | 11 11 15 | 89.7 | 9.1 | SM | @ 10' - Silty SAND: light brown, moist, medium dense | |
| 145 | | | | | | | | | |
| | 15 | | R-5 | 7 10 12 | 103.4 | 1.1 | SP-SM | @ 15' - SAND with Silt to Silty SAND: light reddish brown, dry, medium dense | CO |
| 140 | | | | | | | | | |
| | 20 | | SPT-1 | 5 9 14 | | 16.1 | CL | @ 20' - Sandy CLAY: brown, moist, very stiff | |
| 135 | | | | | | | | | |
| | 25 | | R-6 | 6 7 9 | 90.6 | 21.8 | SC | @ 25' - Clayey SAND: brown, very moist, medium dense | |
| 130 | | | | | | | | Total Depth = 26.5'<br>No Groundwater Encountered<br>Backfilled with Cuttings on 2/22/24 | |
| | 30 | | | | | | | | |

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

**SAMPLE TYPES:**
B BULK SAMPLE
R RING SAMPLE (CA Modified Sampler)
G GRAB SAMPLE
SPT STANDARD PENETRATION TEST SAMPLE
GROUNDWATER TABLE

**TEST TYPES:**
DS DIRECT SHEAR
MD MAXIMUM DENSITY
SA SIEVE ANALYSIS
S&H SIEVE AND HYDROMETER
EI EXPANSION INDEX
CN CONSOLIDATION
CR CORROSION
AL ATTERBERG LIMITS
CO COLLAPSE/SWELL
RV R-VALUE
-#200 % PASSING # 200 SIEVE

# Geotechnical Boring Log Borehole HS-3

| | |
|---|---|
| **Date:** 2/22/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 8" |
| **Elevation of Top of Hole:** ~157' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 1 of 2 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | @0' - Grass | MD EI CR |
| 155 | | B-1 | | | | | | **Young Axial Channel Deposits (Qya):** | |
| | | | R-1 | 4 10 17 | 96.0 | 14.7 | SC | @ 2.5' - Clayey SAND: brown, very moist, medium dense | |
| | 5 | | R-2 | 17 19 19 | 82.1 | 11.9 | | @ 5' - Clayey SAND: brown, moist, dense | |
| 150 | | | R-3 | 14 18 22 | 97.2 | 10.0 | | @ 7.5' - Clayey SAND: brown, moist, dense | |
| | 10 | | R-4 | 19 20 25 | 92.2 | 9.0 | | @ 10' - Clayey SAND: light brown, moist, dense | |
| 145 | | | | | | | | | |
| | 15 | | SPT-1 | 4 5 4 | | 11.2 | | @ 15' - Clayey SAND: brown, moist, medium dense | |
| 140 | | | | | | | | | |
| | 20 | | R-5 | 7 11 11 | 95.2 | 15.4 | | @ 20' - Clayey SAND: light brown, very moist, medium dense | |
| 135 | | | | | | | | | |
| | 25 | | SPT-2 | 3 4 5 | | 9.5 | | @ 25' - Clayey SAND: brown, moist, medium dense | |
| 130 | | | | | | | | | |
| | 30 | | | | | | | | |

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**SAMPLE TYPES:**
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

**TEST TYPES:**
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 10/20/2022

# Geotechnical Boring Log Borehole HS-3

| | |
|---|---|
| **Date:** 2/22/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 8" |
| **Elevation of Top of Hole:** ~157' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 2 of 2 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 30 | | R-6 | 8 12 14 | 95.9 | 7.8 | SC | @ 30' - Clayey SAND: pale brown, moist, medium dense | |
| 125 | | | | | | | | | |
| | 35 | | SPT-3 | 4 4 5 | | 12.3 | SM | @ 35' - Silty SAND: brown, moist, medium dense | |
| 120 | | | | | | | | | |
| | 40 | | R-7 | 9 16 20 | 105.6 | 10.1 | | @ 40' - Silty SAND: brown, moist, medium dense | |
| 115 | | | | | | | | | |
| | 45 | | SPT-4 | 6 7 7 | | 7.3 | | @ 45' - Silty SAND: brown, moist, medium dense | |
| 110 | | | | | | | | | |
| | 50 | | R-8 | 9 9 24 | 99.6 | 23.6 | CL | @ 50' - Sandy CLAY: brown, very moist, very stiff | |
| 105 | | | | | | | | Total Depth = 51.5'<br>No Groundwater Encountered<br>Backfilled with Cuttings on 2/22/24 | |
| | 55 | | | | | | | | |
| 100 | | | | | | | | | |
| | 60 | | | | | | | | |

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**SAMPLE TYPES:**
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

**TEST TYPES:**
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

# Geotechnical Boring Log Borehole HS-4

| | |
|---|---|
| **Date:** 2/23/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 8" |
| **Elevation of Top of Hole:** ~160' MSL (google) | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 1 of 1 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | @0' - Grass<br>**Young Axial Channel Deposits (Qya):** | |
| | | B-1 | R-1 | 3 6 12 | 98.0 | 13.7 | CL | @ 2.5' - Sandy CLAY: brown, moist, stiff | |
| 155 | 5 | | R-2 | 10 10 13 | 79.7 | 11.5 | SC | @ 5' - Clayey SAND: brown, moist, medium dense | |
| | | | R-3 | 17 16 14 | 92.1 | 12.2 | | @ 7.5' - Clayey SAND: brown, moist, medium dense | |
| 150 | 10 | | R-4 | 15 16 22 | 84.4 | 12.8 | CL | @ 10' - Sandy CLAY: brown, moist, hard | |
| 145 | 15 | | R-5 | 6 7 11 | 86.4 | 14.2 | ML | @ 15' - Sandy SILT: light brown, moist, stiff | |
| 140 | 20 | | SPT-1 | 4 7 8 | | 10.2 | SC | @ 20' - Clayey SAND: brown, moist, medium dense | |
| 135 | 25 | | R-6 | 4 8 9 | 88.9 | 8.4 | SM | @ 25' - Silty SAND: reddish brown, moist, medium dense | |
| 130 | 30 | | | | | | | Total Depth = 26.5'<br>No Groundwater Encountered<br>Backfilled with Cuttings on 2/23/24 | |

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

**SAMPLE TYPES:**
- B BULK SAMPLE
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- G GRAB SAMPLE
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- GROUNDWATER TABLE

**TEST TYPES:**
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- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 10/20/2022

# Geotechnical Boring Log Borehole I-1

| | |
|---|---|
| **Date:** 2/22/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 8" |
| **Elevation of Top of Hole:** ~152' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 1 of 1 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | @0' - 5" of Asphalt over 5.5" of Base | |
| 150 | | | | | | | | **Young Axial Channel Deposits (Qya):** | |
| | | | R-1 | 10 12 17 | 89.9 | 15.9 | SC | @ 2.5' - Clayey SAND: brown, very moist, medium dense | |
| | 5 | | R-2 | 13 15 17 | 103.1 | 15.7 | CL | @ 5' - Sandy CLAY: dark brown, moist, very stiff | |
| 145 | | | | | | | | | |
| | | | R-3 | 10 15 18 | 102.0 | 10.6 | SC | @ 8.5' - Clayey SAND: brown, moist, medium dense. | -#200 |
| | 10 | | | | | | | Total Depth = 10'<br>No Groundwater Encountered<br>3" Perforated Pipe with Filter Sock and Gravel Installed<br>Pipe Removed, Backfilled with Cuttings and Capped with Cold Patch on 2/26/2024 | |
| 140 | | | | | | | | | |
| | 15 | | | | | | | | |
| 135 | | | | | | | | | |
| | 20 | | | | | | | | |
| 130 | | | | | | | | | |
| | 25 | | | | | | | | |
| 125 | | | | | | | | | |
| | 30 | | | | | | | | |

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**SAMPLE TYPES:**
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

**TEST TYPES:**
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 10/20/2022

# Geotechnical Boring Log Borehole I-2

| | |
|---|---|
| **Date:** 2/22/24 | **Drilling Company:** 2R Drilling |
| **Project Name:** Our Lady of Peace, Irvine | **Type of Rig:** Limited Access Rig |
| **Project Number:** 23174-01 | **Drop:** 30" **Hole Diameter:** 8" |
| **Elevation of Top of Hole:** ~155' MSL | **Drive Weight:** 140 pounds |
| **Hole Location:** See Geotechnical Map | Page 1 of 1 |

Logged By RNP
Sampled By RNP
Checked By BPP

| Elevation (ft) | Depth (ft) | Graphic Log | Sample Number | Blow Count | Dry Density (pcf) | Moisture (%) | USCS Symbol | DESCRIPTION | Type of Test |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | @0' - 4" of Asphalt over 8.5" of Base | |
| | | | | | | | | <u>**Young Axial Channel Deposits (Qya):**</u> | |
| | | | R-1 | 4 8 13 | 107.3 | 20.4 | CL | @ 2.5' - Sandy CLAY: dark brown, very moist, very stiff | |
| 150 | 5 | | R-2 | 6 10 13 | 94.1 | 16.5 | SC | @ 5' - Clayey SAND: brown, very moist, medium dense | |
| | | | R-3 | 8 14 18 | 96.2 | 15.3 | | @ 8.5' - Clayey SAND: brown, very moist, medium dense | |
| 145 | 10 | | | | | | | Total Depth = 10'<br>No Groundwater Encountered<br>3" Perforated Pipe with Filter Sock and Gravel installed<br>Pipe Removed, Backfilled with Cuttings and Capped with Cold Patch on 2/26/2024 | |
| 140 | 15 | | | | | | | | |
| 135 | 20 | | | | | | | | |
| 130 | 25 | | | | | | | | |
| 125 | 30 | | | | | | | | |

LGC Geotechnical, Inc.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

**SAMPLE TYPES:**
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

**TEST TYPES:**
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 10/20/2022

# *Appendix C*
# *Laboratory Test Results*

## *APPENDIX C*

## ***<u>Laboratory Testing Procedures and Test Results</u>***

The laboratory testing program was formulated towards providing data relating to the relevant engineering properties of the soils with respect to residential construction. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

<u>Moisture and Density Determination Tests</u>: Moisture content (ASTM D2216) and dry density determinations (ASTM D2937) were performed on relatively undisturbed samples obtained from the test borings. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

<u>Grain Size Distribution/Fines Content</u>: Representative samples were dried, weighed and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve (ASTM D1140). Where applicable, the portion retained on the No. 200 sieve and dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D6913 (sieve).

| Sample Location | Description | % Passing # 200 Sieve |
|---|---|---|
| I-1 @ 8.5 feet | Clayey Sand | 48 |
| I-3 @ 8.5 feet | Sandy Clay | 68 |

<u>Expansion Index</u>: The expansion potential of selected samples was evaluated by the Expansion Index Test, Standard ASTM D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch-thick by 4-inch-diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below.

| Sample Location | Expansion Index | Expansion Potential* |
|---|---|---|
| HS-3 @ 1-5 feet | 112 | High |
| HS-6 @ 1-5 feet | 135 | Very High |

* ASTM D4829

<u>Maximum Density Tests</u>: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM D1557. The results of these tests are presented in the table below:

***Laboratory Testing Procedures and Test Results***

| Sample Location | Sample Description | Maximum Dry Density (pcf) | Optimum Moisture Content (%) |
|---|---|---|---|
| HS-3 @ 1-5 feet | Dark Grayish Brown Clay | 117.5 | 12.5 |
| HS-6 @ 1-5 feet | Dark Brown Sandy Clay | 115.0 | 14.0 |

Collapse/Swell Potential: Six collapse tests were performed per ASTM D4546. A sample (2.4 inches in diameter and 1-inch in height) was placed in a consolidometer and loaded to the approximate in-situ effective stress. The curve is presented in this Appendix.

Chloride Content: Chloride content was tested in accordance with Caltrans Test Method (CTM) 422. The results are presented below.

| Sample Location | Chloride Content, ppm |
|---|---|
| HS-3 @ 1-5 feet | 60 |

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM 417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below.

| Sample Location | Sulfate Content (ppm) | Sulfate Exposure Class * |
|---|---|---|
| HS-3 @ 1-5 feet | 95 | S0 |

*Based on ACI 318R-14, Table 19.3.1.1

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with CTM 643 and standard geochemical methods. The results are presented in the table below.

| Sample Location | pH | Minimum Resistivity (ohms-cm) |
|---|---|---|
| HS-3 @ 1-5 feet | 7.90 | 720 |

# ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

**ASTM D 4546**

| | | | | | |
|---|---|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala | Date: | 05/20/24 |
| Project No.: | 23174-01 | Checked By: | J. Ward | Date: | 05/22/24 |
| Boring No.: | HS-1 | Sample Type: | Ring | | |
| Sample No.: | R-4 | Depth (ft.) | 10.0 | | |
| Sample Description: | Brown lean clay (CL) | | | | |

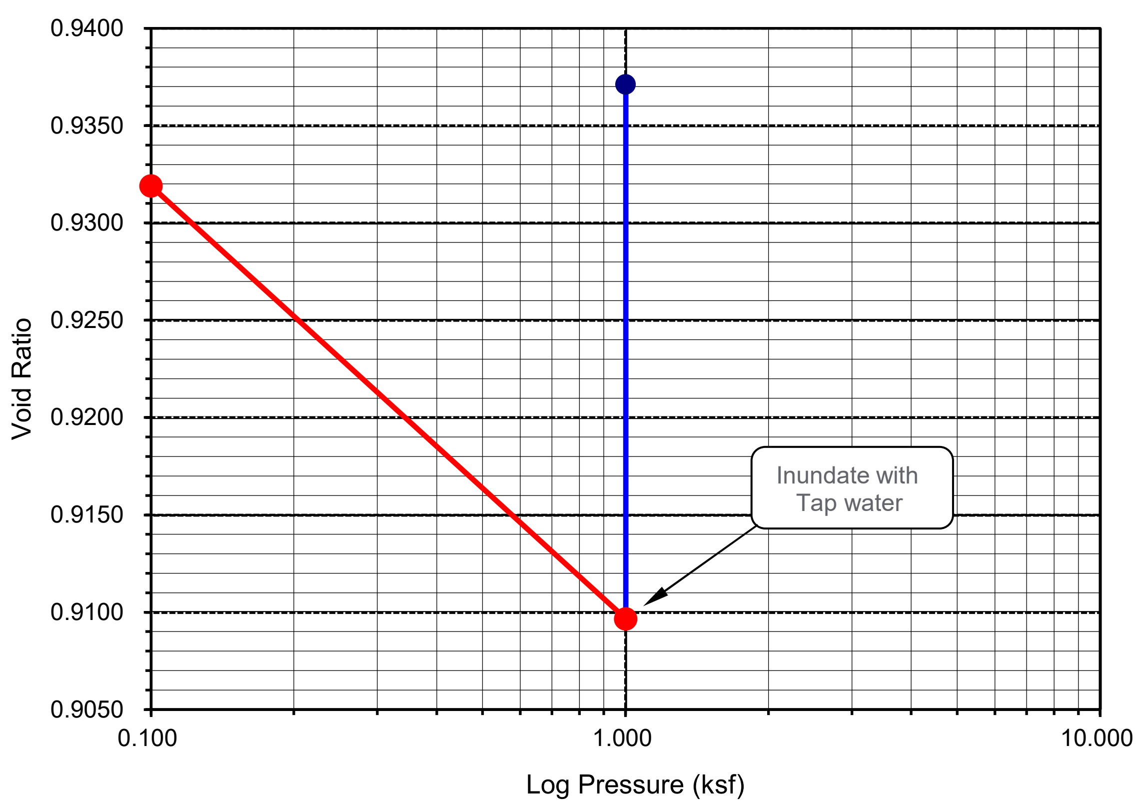
| | |
|---|---|
| Initial Dry Density (pcf): | 87.2 |
| Initial Moisture (%): | 22.94 |
| Initial Length (in.): | 1.0000 |
| Initial Dial Reading: | 0.1187 |
| Diameter(in): | 2.415 |

| | |
|---|---|
| Final Dry Density (pcf): | 87.3 |
| Final Moisture (%) : | 31.6 |
| Initial Void ratio: | 0.9335 |
| Specific Gravity(assumed): | 2.70 |
| Initial Saturation (%) | 66.4 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.1195 | 0.9992 | 0.00 | -0.08 | 0.9319 | -0.08 |
| 1.000 | 0.1325 | 0.9862 | 0.15 | -1.38 | 0.9097 | -1.23 |
| H2O | 0.1183 | 1.0004 | 0.15 | 0.04 | 0.9371 | 0.19 |

**Percent Swell (+) / Settlement (-) After Inundation = 1.44**

Void Ratio - Log Pressure Curve



# ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

**ASTM D 4546**

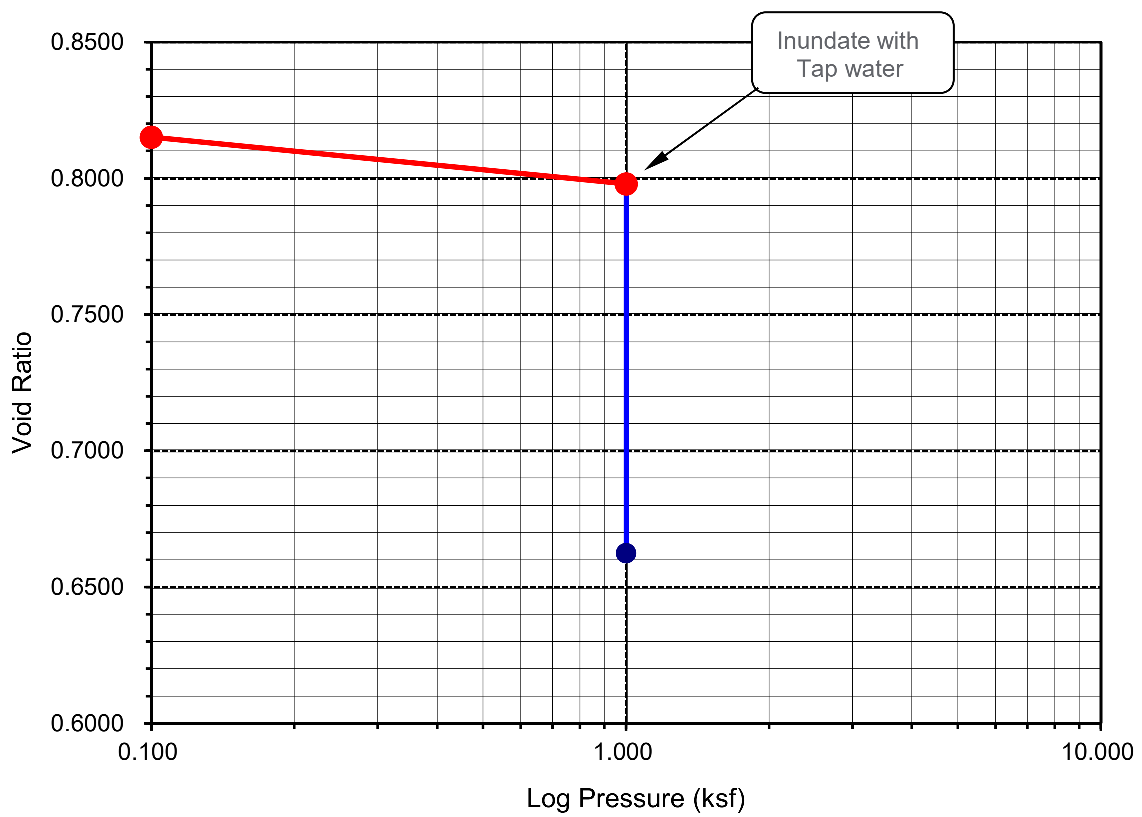
| | | | | | |
|---|---|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala | Date: | 05/21/24 |
| Project No.: | 23174-01 | Checked By: | J. Ward | Date: | 05/23/24 |
| Boring No.: | HS-2 | Sample Type: | Ring | | |
| Sample No.: | R-4 | Depth (ft.) | 10.0 | | |
| Sample Description: | Brown silty clay (CL-ML) | | | | |

| | | | |
|---|---|---|---|
| Initial Dry Density (pcf): | 92.3 | Final Dry Density (pcf): | 101.7 |
| Initial Moisture (%): | 7.54 | Final Moisture (%) : | 20.8 |
| Initial Length (in.): | 1.0000 | Initial Void ratio: | 0.8270 |
| Initial Dial Reading: | 0.1122 | Specific Gravity(assumed): | 2.70 |
| Diameter(in): | 2.415 | Initial Saturation (%) | 24.6 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.1187 | 0.9935 | 0.00 | -0.65 | 0.8151 | -0.65 |
| 1.000 | 0.1296 | 0.9826 | 0.15 | -1.74 | 0.7980 | -1.59 |
| H2O | 0.2038 | 0.9084 | 0.15 | -9.16 | 0.6624 | -9.01 |

**Percent Swell (+) / Settlement (-) After Inundation = -7.54**

Void Ratio - Log Pressure Curve



## ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
### ASTM D 4546

| | | | | | |
|---|---|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala | Date: | 03/13/24 |
| Project No.: | 23174-01 | Checked By: | J. Ward | Date: | 03/21/24 |
| Boring No.: | HS-2 | Sample Type: | Ring | | |
| Sample No.: | R-5 | Depth (ft.) | 15.0 | | |
| Sample Description: | Brown silty sand (SM) | | | | |

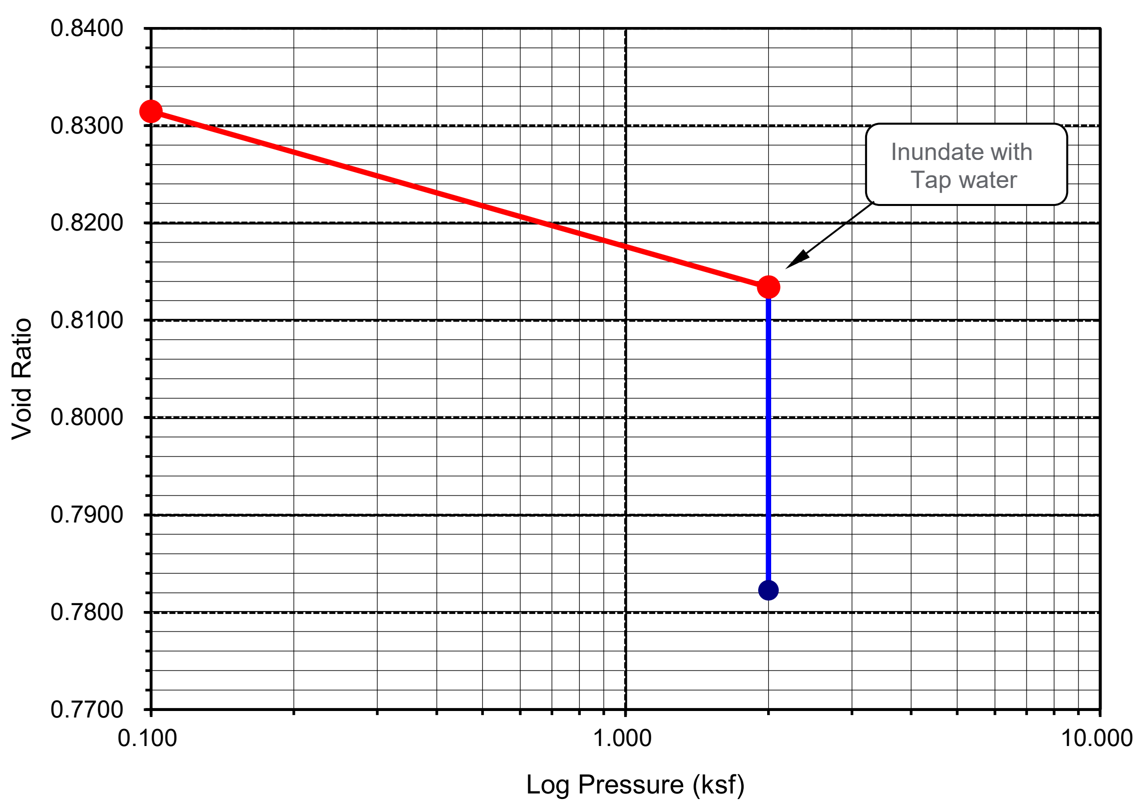
| | |
|---|---|
| Initial Dry Density (pcf): | 92.0 |
| Initial Moisture (%): | 8.87 |
| Initial Length (in.): | 1.0000 |
| Initial Dial Reading: | 0.0680 |
| Diameter(in): | 2.415 |

| | |
|---|---|
| Final Dry Density (pcf): | 95.9 |
| Final Moisture (%) : | 22.0 |
| Initial Void ratio: | 0.8325 |
| Specific Gravity(assumed): | 2.70 |
| Initial Saturation (%) | 28.8 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.06855 | 0.9995 | 0.00 | -0.06 | 0.8315 | -0.06 |
| 2.000 | 0.0849 | 0.9831 | 0.65 | -1.69 | 0.8134 | -1.04 |
| $H_2O$ | 0.1019 | 0.9661 | 0.65 | -3.39 | 0.7823 | -2.74 |

**Percent Swell (+) / Settlement (-) After Inundation = -1.72**

Void Ratio - Log Pressure Curve



**ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS**

**ASTM D 4546**

| | | | | | |
|---|---|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala | Date: | 05/20/24 |
| Project No.: | 23174-01 | Checked By: | J. Ward | Date: | 05/22/24 |
| Boring No.: | HS-3 | Sample Type: | Ring | | |
| Sample No.: | R-4 | Depth (ft.) | 10.0 | | |
| Sample Description: | Brown lean clay with sand (CL)s | | | | |

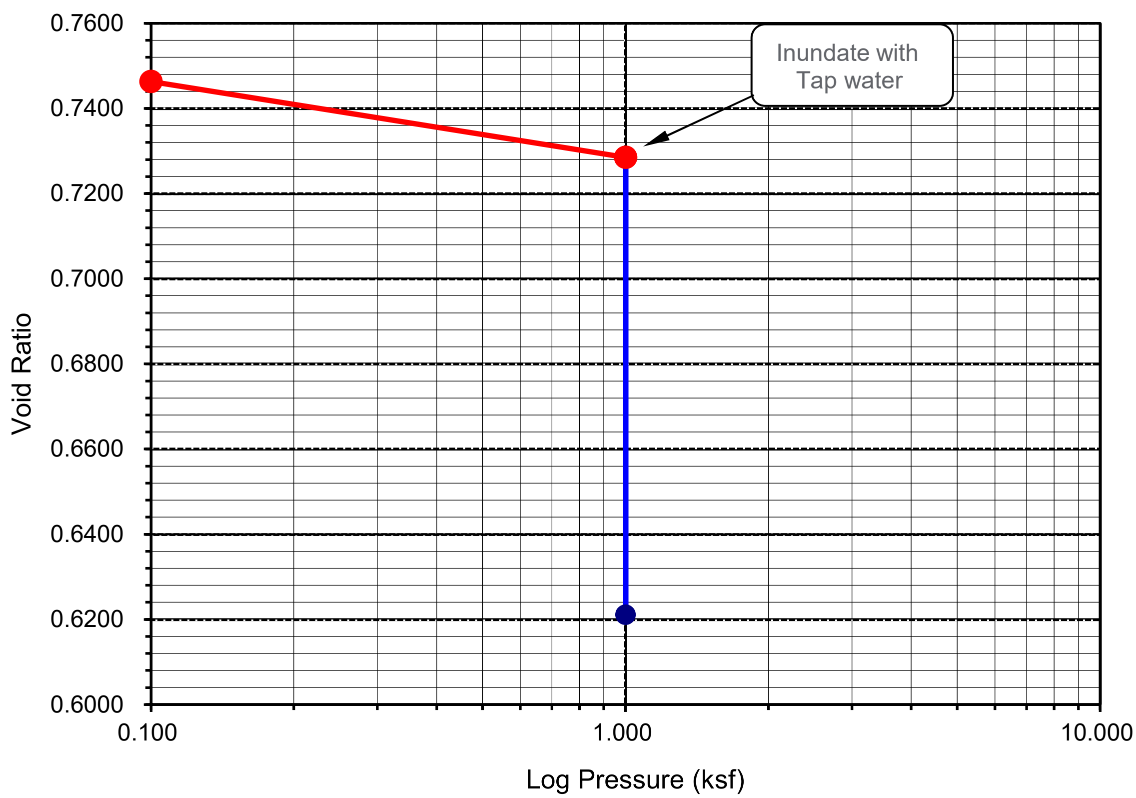
| | |
|---|---|
| Initial Dry Density (pcf): | 96.5 |
| Initial Moisture (%): | 5.37 |
| Initial Length (in.): | 1.0000 |
| Initial Dial Reading: | 0.0967 |
| Diameter(in): | 2.415 |

| | |
|---|---|
| Final Dry Density (pcf): | 104.3 |
| Final Moisture (%) : | 19.1 |
| Initial Void ratio: | 0.7469 |
| Specific Gravity(assumed): | 2.70 |
| Initial Saturation (%) | 19.4 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.0970 | 0.9997 | 0.00 | -0.03 | 0.7464 | -0.03 |
| 1.000 | 0.1087 | 0.9880 | 0.15 | -1.20 | 0.7285 | -1.05 |
| H2O | 0.1702 | 0.9265 | 0.15 | -7.35 | 0.6211 | -7.20 |

**Percent Swell (+) / Settlement (-) After Inundation = -6.22**

Void Ratio - Log Pressure Curve



Leighton

# ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

## ASTM D 4546

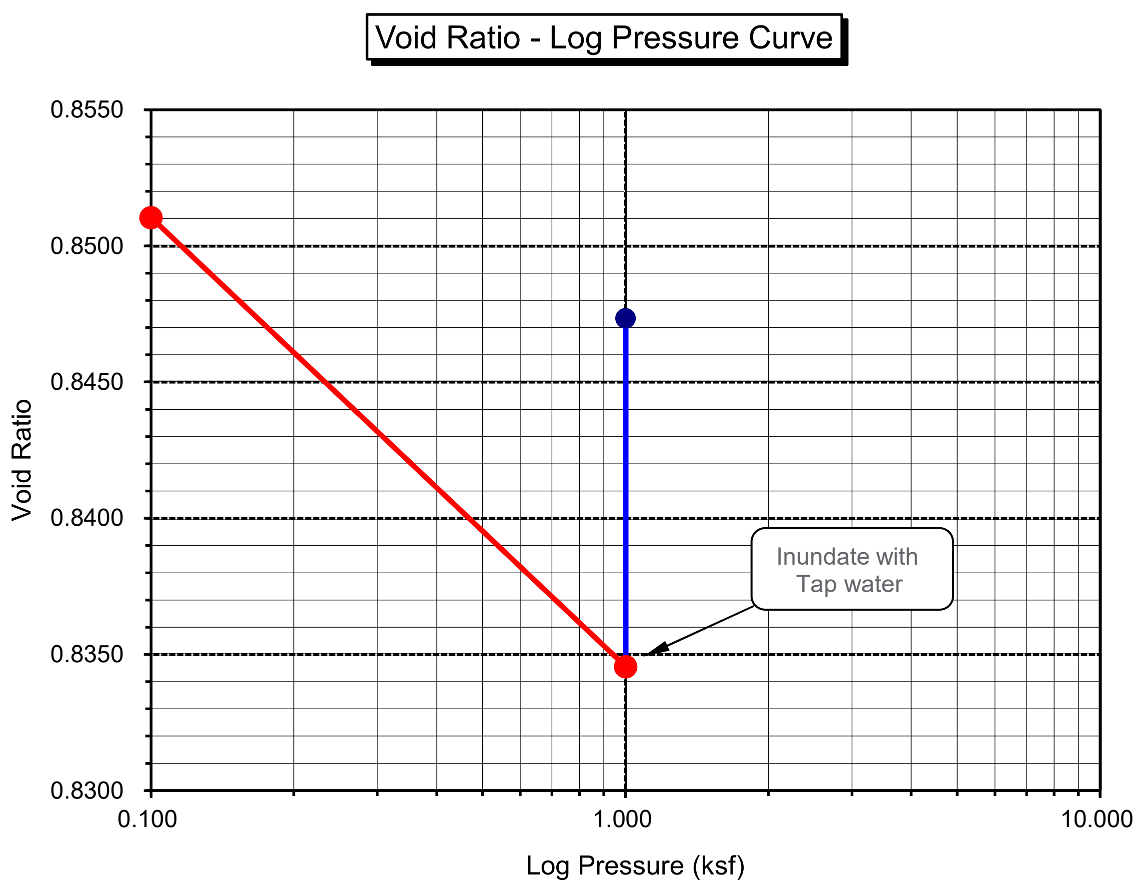
| | | | | | |
|---|---|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala | Date: | 05/21/24 |
| Project No.: | 23174-01 | Checked By: | J. Ward | Date: | 05/23/24 |
| Boring No.: | HS-4 | Sample Type: | Ring | | |
| Sample No.: | R-4 | Depth (ft.) | 10.0 | | |
| Sample Description: | Brown lean clay (CL) | | | | |

| | |
|---|---|
| Initial Dry Density (pcf): | 91.0 |
| Initial Moisture (%): | 13.63 |
| Initial Length (in.): | 1.0000 |
| Initial Dial Reading: | 0.0961 |
| Diameter(in): | 2.415 |

| | |
|---|---|
| Final Dry Density (pcf): | 91.5 |
| Final Moisture (%) : | 24.3 |
| Initial Void ratio: | 0.8522 |
| Specific Gravity(assumed): | 2.70 |
| Initial Saturation (%) | 43.2 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.0967 | 0.9994 | 0.00 | -0.06 | 0.8510 | -0.06 |
| 1.000 | 0.1071 | 0.9890 | 0.15 | -1.10 | 0.8346 | -0.95 |
| H2O | 0.1002 | 0.9959 | 0.15 | -0.41 | 0.8473 | -0.26 |

**Percent Swell (+) / Settlement (-) After Inundation = 0.70**



# ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

### ASTM D 4546

| | | | | | |
|---|---|---|---|---|---|
| Project Name: | Shea - Our Lady of Peace, Irvine | Tested By: | G. Bathala | Date: | 03/13/24 |
| Project No.: | 23174-01 | Checked By: | J. Ward | Date: | 03/21/24 |
| Boring No.: | HS-5 | Sample Type: | Ring | | |
| Sample No.: | R-4 | Depth (ft.) | 10.0 | | |
| Sample Description: | Olive brown silty clay (CL-ML) | | | | |

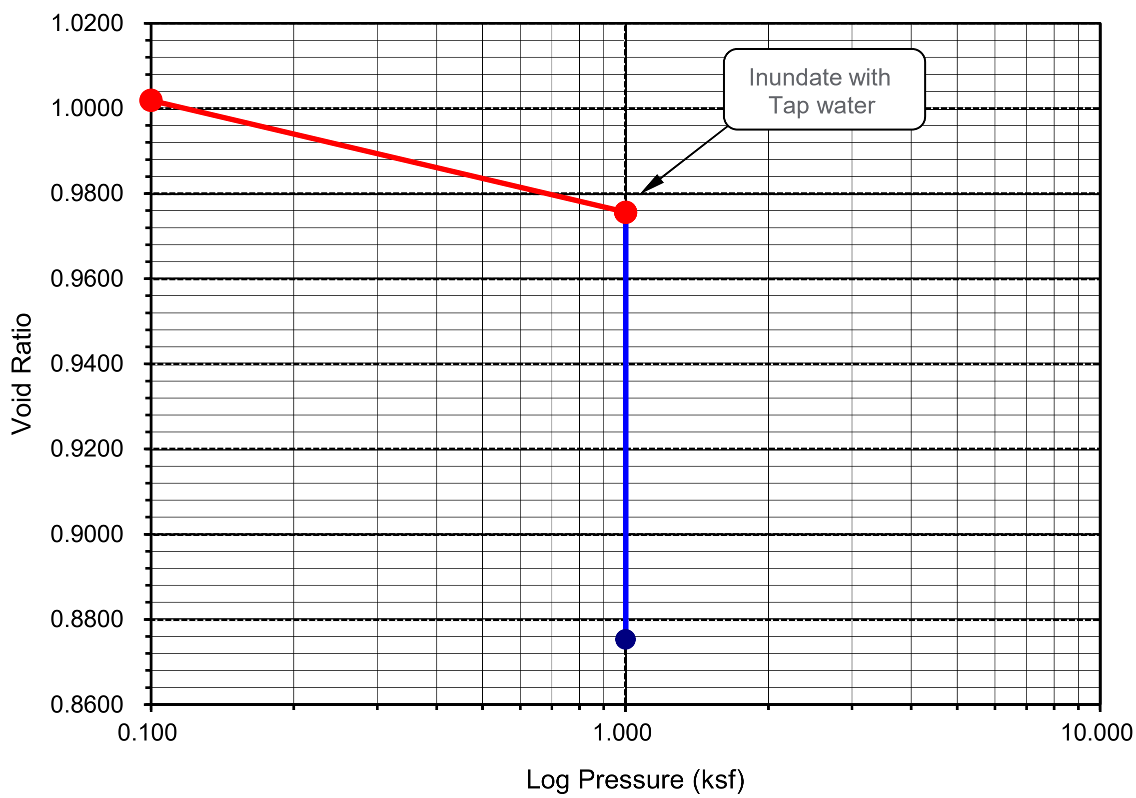
| | |
|---|---|
| Initial Dry Density (pcf): | 84.2 |
| Initial Moisture (%): | 12.07 |
| Initial Length (in.): | 1.0000 |
| Initial Dial Reading: | 0.1519 |
| Diameter(in): | 2.415 |

| | |
|---|---|
| Final Dry Density (pcf): | 90.2 |
| Final Moisture (%) : | 25.5 |
| Initial Void ratio: | 1.0029 |
| Specific Gravity(assumed): | 2.70 |
| Initial Saturation (%) | 32.5 |

| Pressure (p) (ksf) | Final Reading (in) | Apparent Thickness (in) | Load Compliance (%) | Swell (+) Settlement (-) % of Sample Thickness | Void Ratio | Corrected Deformation (%) |
|---|---|---|---|---|---|---|
| 0.100 | 0.1524 | 0.9995 | 0.00 | -0.05 | 1.0019 | -0.05 |
| 1.000 | 0.1670 | 0.9849 | 0.15 | -1.51 | 0.9757 | -1.36 |
| H2O | 0.2171 | 0.9348 | 0.15 | -6.52 | 0.8753 | -6.37 |

**Percent Swell (+) / Settlement (-) After Inundation = -5.08**

Void Ratio - Log Pressure Curve



# *Appendix D*
# *Infiltration Test Data*

# Infiltration Test Data Sheet

## LGC Geotechnical, Inc

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

**Project Name:** Our Lady of Peace, Irvine
**Project Number:** 23174-01
**Date:** 2/26/2024
**Boring Number:** I-1

| Test hole dimensions (if circular) | |
|---|---|
| Boring Depth (feet)*: | 10 |
| Boring Diameter (inches): | 8 |
| Pipe Diameter (inches): | 3 |

*measured at time of test

| Test pit dimensions (if rectangular) | |
|---|---|
| Pit Depth (feet): | |
| Pit Length (feet): | |
| Pit Breadth (feet): | |

Minimum test Head ($D_o$):

(What the sounder tape should read) Boring Depth - (5 x Boring Radius) 8.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

### *Pre-Test (Sandy Soil Criteria)**

| Trial No. | Start Time (24:HR) | Stop Time (24:HR) | Time Interval (min) | Initial Depth to Water (feet) | Final Depth to Water (feet) | Total Change in Water Level (feet) | Greater Than or Equal to 0.5 feet (yes/no) |
|---|---|---|---|---|---|---|---|
| 1 | 7:08 | 7:33 | 25.0 | 7.21 | 7.25 | 0.04 | Yes |
| 2 | 7:35 | 8:00 | 25.0 | 7.23 | 7.27 | 0.04 | Yes |

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight, and then obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25 inches

### *Main Test Data*

| Trial No. | Start Time (24:HR) | Stop Time (24:HR) | Time Interval, Dt (min) | Initial Depth to Water, $D_o$ (feet) | Final Depth to Water, $D_f$ (feet) | Change in Water Level, DD (feet) | Measured Infiltration Rate(in/hr) |
|---|---|---|---|---|---|---|---|
| 1 | 8:00 | 8:30 | 30.0 | 7.20 | 7.24 | 0.04 | 0.1 |
| 2 | 8:32 | 9:02 | 30.0 | 7.24 | 7.28 | 0.04 | 0.1 |
| 3 | 9:04 | 9:34 | 30.0 | 7.28 | 7.31 | 0.03 | 0.0 |
| 4 | 9:37 | 10:07 | 30.0 | 7.31 | 7.34 | 0.03 | 0.0 |
| 5 | 10:09 | 10:39 | 30.0 | 7.25 | 7.29 | 0.04 | 0.1 |
| 6 | 10:41 | 11:11 | 30.0 | 7.21 | 7.24 | 0.03 | 0.0 |
| 7 | 11:13 | 11:43 | 30.0 | 7.24 | 7.27 | 0.03 | 0.0 |
| 8 | 11:45 | 12:15 | 30.0 | 7.27 | 7.29 | 0.02 | 0.0 |
| 9 | 12:18 | 12:48 | 30.0 | 7.18 | 7.21 | 0.03 | 0.0 |
| 10 | 12:50 | 13:20 | 30.0 | 7.21 | 7.23 | 0.02 | 0.0 |
| 11 | 13:22 | 13:52 | 30.0 | 7.23 | 7.25 | 0.02 | 0.0 |
| 12 | 13:52 | 14:22 | 30.0 | 7.25 | 7.27 | 0.02 | 0.0 |
| | | | | | | **Measured Infiltration Rate (No Factor of Safety)** | **0.03** |

***Sketch:***

***Notes:***

Based on Guidelines from: South Orange County 9/28/2017

Spreadsheet Revised on: 10/30/2019

# Infiltration Test Data Sheet

**LGC Geotechnical, Inc**

**131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141**

**Project Name:** Our Lady of Peace, Irvine
**Project Number:** 23174-01
**Date:** 2/26/2024
**Boring Number:** I-2

| Test hole dimensions (if circular) | |
|---|---|
| Boring Depth (feet)*: | 10 |
| Boring Diameter (inches): | 8 |
| Pipe Diameter (inches): | 3 |

*measured at time of test

| Test pit dimensions (if rectangular) | |
|---|---|
| Pit Depth (feet): | |
| Pit Length (feet): | |
| Pit Breadth (feet): | |

Minimum test Head ($D_o$):

(What the sounder tape should read) Boring Depth - (5 x Boring Radius) 8.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

### *Pre-Test (Sandy Soil Criteria)**

| Trial No. | Start Time (24:HR) | Stop Time (24:HR) | Time Interval (min) | Initial Depth to Water (feet) | Final Depth to Water (feet) | Total Change in Water Level (feet) | Greater Than or Equal to 0.5 feet (yes/no) |
|---|---|---|---|---|---|---|---|
| 1 | 7:10 | 7:35 | 25.0 | 7.44 | 7.47 | 0.03 | Yes |
| 2 | 7:37 | 8:02 | 25.0 | 7.39 | 7.42 | 0.03 | Yes |

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight, and then obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25 inches

### *Main Test Data*

| Trial No. | Start Time (24:HR) | Stop Time (24:HR) | Time Interval, Dt (min) | Initial Depth to Water, $D_o$ (feet) | Final Depth to Water, $D_f$ (feet) | Change in Water Level, DD (feet) | Measured Infiltration Rate(in/hr) |
|---|---|---|---|---|---|---|---|
| 1 | 8:04 | 8:34 | 30.0 | 7.42 | 7.45 | 0.03 | 0.0 |
| 2 | 8:36 | 9:06 | 30.0 | 7.35 | 7.38 | 0.03 | 0.0 |
| 3 | 9:08 | 9:38 | 30.0 | 7.36 | 7.39 | 0.03 | 0.0 |
| 4 | 9:40 | 10:10 | 30.0 | 7.31 | 7.33 | 0.02 | 0.0 |
| 5 | 10:12 | 10:42 | 30.0 | 7.33 | 7.36 | 0.03 | 0.0 |
| 6 | 10:45 | 11:15 | 30.0 | 7.36 | 7.38 | 0.02 | 0.0 |
| 7 | 11:17 | 11:47 | 30.0 | 7.33 | 7.35 | 0.02 | 0.0 |
| 8 | 11:49 | 12:19 | 30.0 | 7.34 | 7.36 | 0.02 | 0.0 |
| 9 | 12:22 | 12:52 | 30.0 | 7.32 | 7.34 | 0.02 | 0.0 |
| 10 | 12:54 | 13:24 | 30.0 | 7.34 | 7.35 | 0.01 | 0.0 |
| 11 | 13:26 | 13:56 | 30.0 | 7.35 | 7.36 | 0.01 | 0.0 |
| 12 | 13:57 | 14:27 | 30.0 | 7.36 | 7.37 | 0.01 | 0.0 |
| | | | | | | **Measured Infiltration Rate (No Factor of Safety)** | **0.01** |

***Sketch:***

***Notes:***

Based on Guidelines from: South Orange County 9/28/2017

Spreadsheet Revised on: 10/30/2019

LGC Geotechnical, Inc.

# *Appendix E*
# *General Earthwork and Grading Specifications for Rough Grading*

## ***General Earthwork and Grading Specifications for Rough Grading***

### *1.0 General*

#### *1.1 Intent*

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

#### *1.2 The Geotechnical Consultant of Record*

Prior to commencement of work, the owner shall employ a qualified Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to confirm that the attained level of compaction is being accomplished as specified. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

#### *1.3 The Earthwork Contractor*

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the project plans and specifications. The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of “equipment” of work and the estimated quantities of daily earthwork

contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the
Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate personnel will be available for observation and testing. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified. It is the contractor's sole responsibility to provide proper fill compaction.

## *2.0 Preparation of Areas to be Filled*

### *2.1 Clearing and Grubbing*

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

### *2.2 Processing*

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be over-excavated as specified in the following section. Scarification shall continue until soils are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

### *2.3 Over-excavation*

In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by the Geotechnical Consultant during grading.

### *2.4 Benching*

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

### *2.5 Evaluation/Acceptance of Fill Areas*

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

## *3.0 Fill Material*

### *3.1 General*

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

### *3.2 Oversize*

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

### *3.3 Import*

If importing of fill material is required for grading, proposed import material shall meet the requirements of the geotechnical consultant. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

## *4.0 Fill Placement and Compaction*

### *4.1 Fill Layers*

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

### *4.2 Fill Moisture Conditioning*

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

### *4.3 Compaction of Fill*

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

### *4.4 Compaction of Fill Slopes*

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

### *4.5 Compaction Testing*

Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

### *4.6 Frequency of Compaction Testing*

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

### *4.7 Compaction Test Locations*

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than
5 feet apart from potential test locations shall be provided.

## *5.0 Subdrain Installation*

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

## *6.0 Excavation*

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

## *7.0 Trench Backfills*

***7.1*** The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

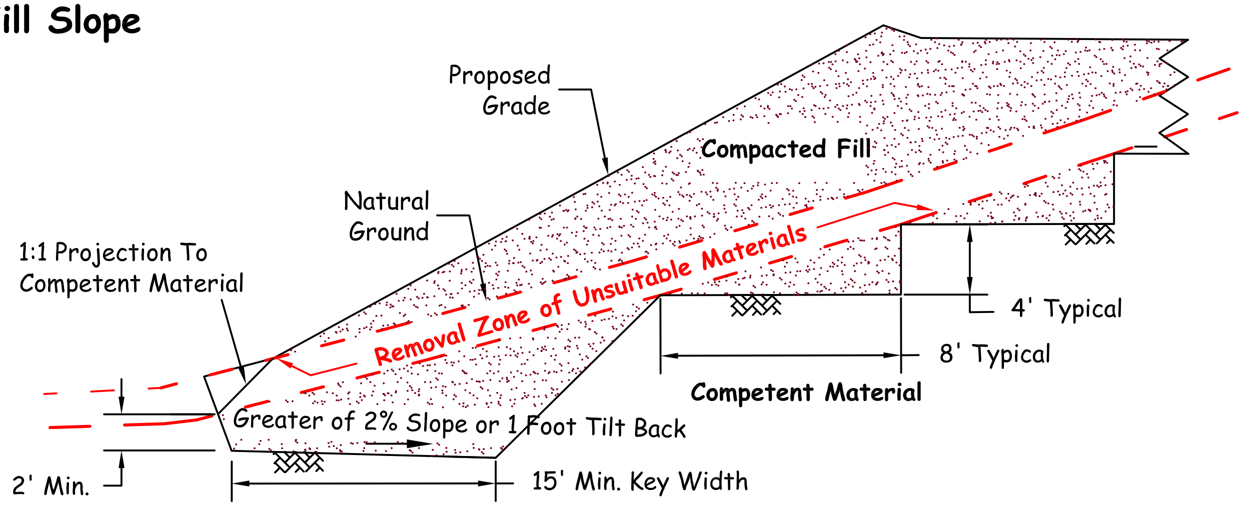
***7.2*** All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

***7.3*** The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

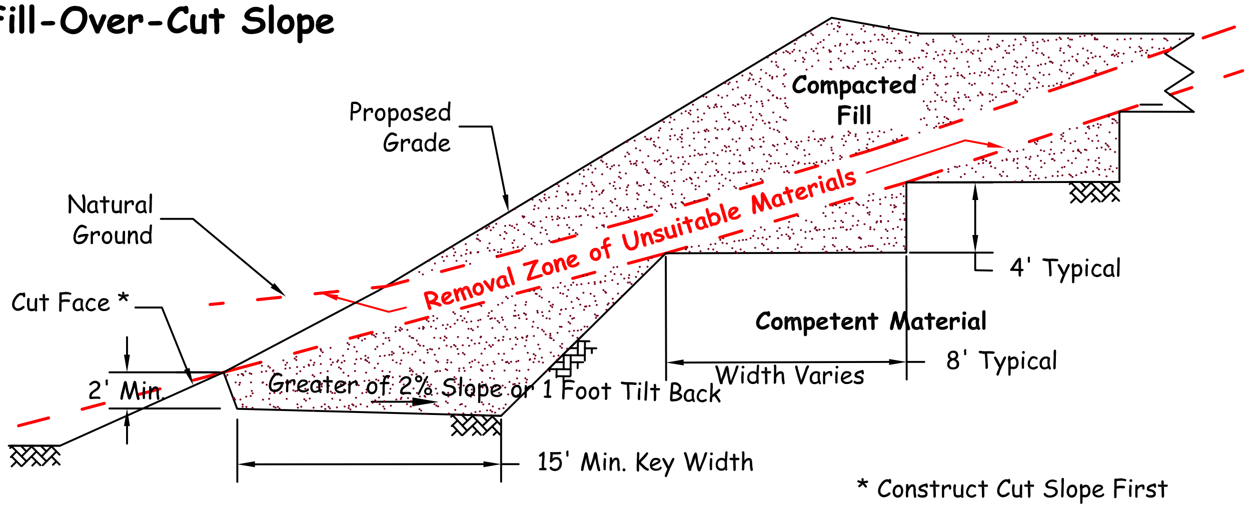
***7.4*** The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

***7.5*** Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

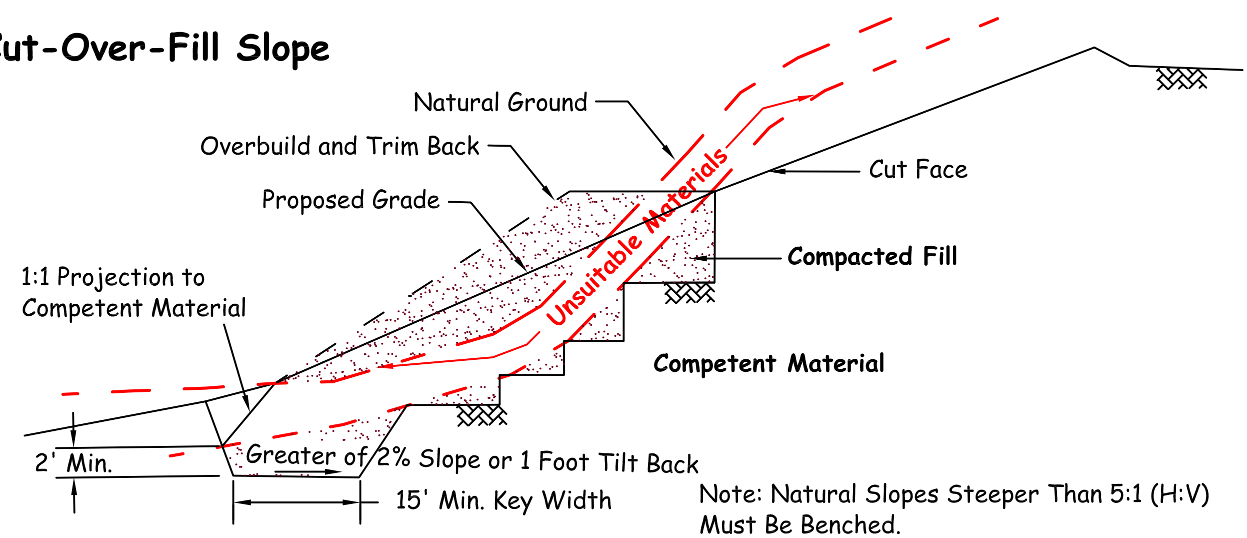
## Fill Slope




## Fill-Over-Cut Slope




## Cut-Over-Fill Slope

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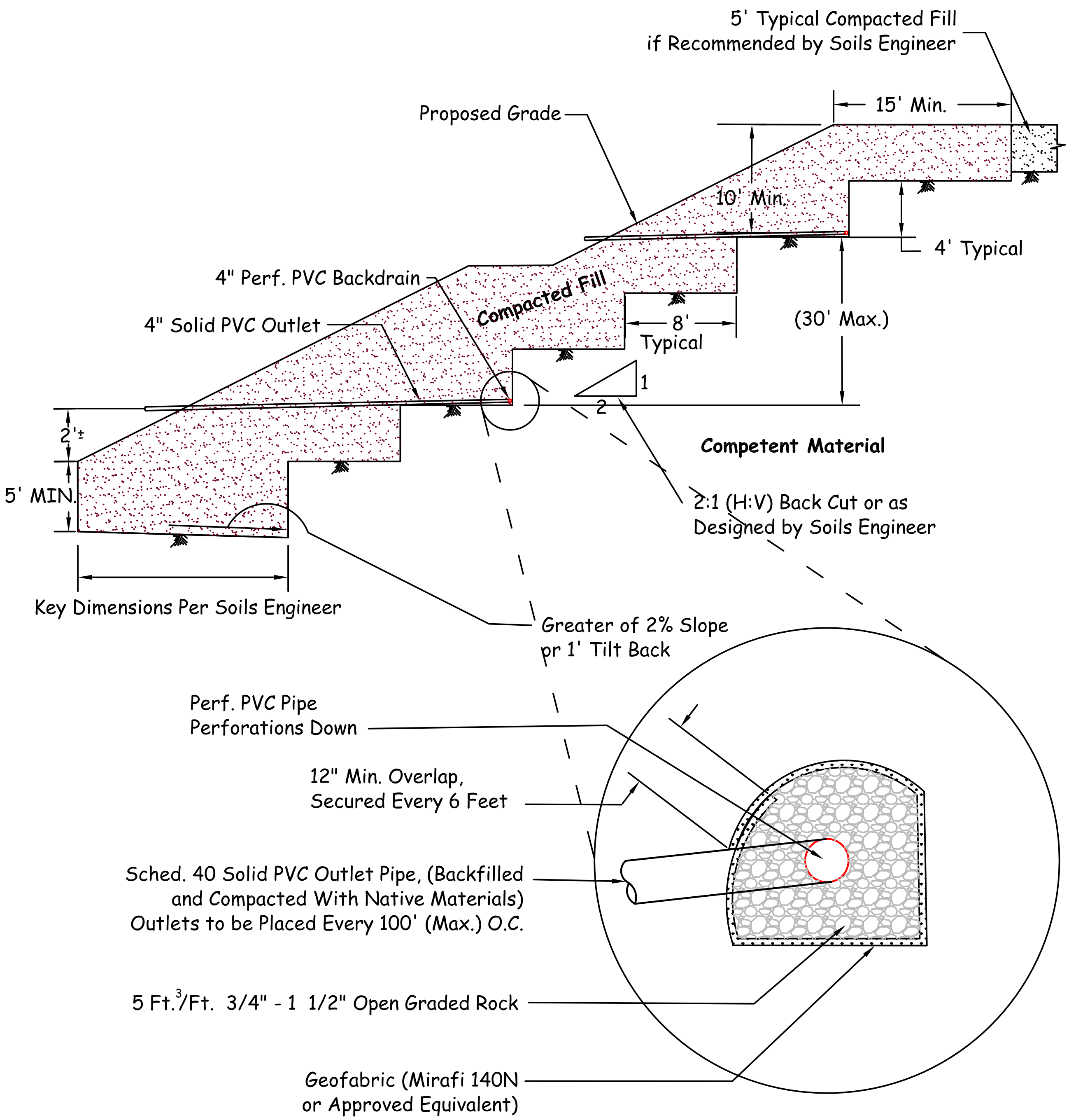
**KEYING AND BENCHING**

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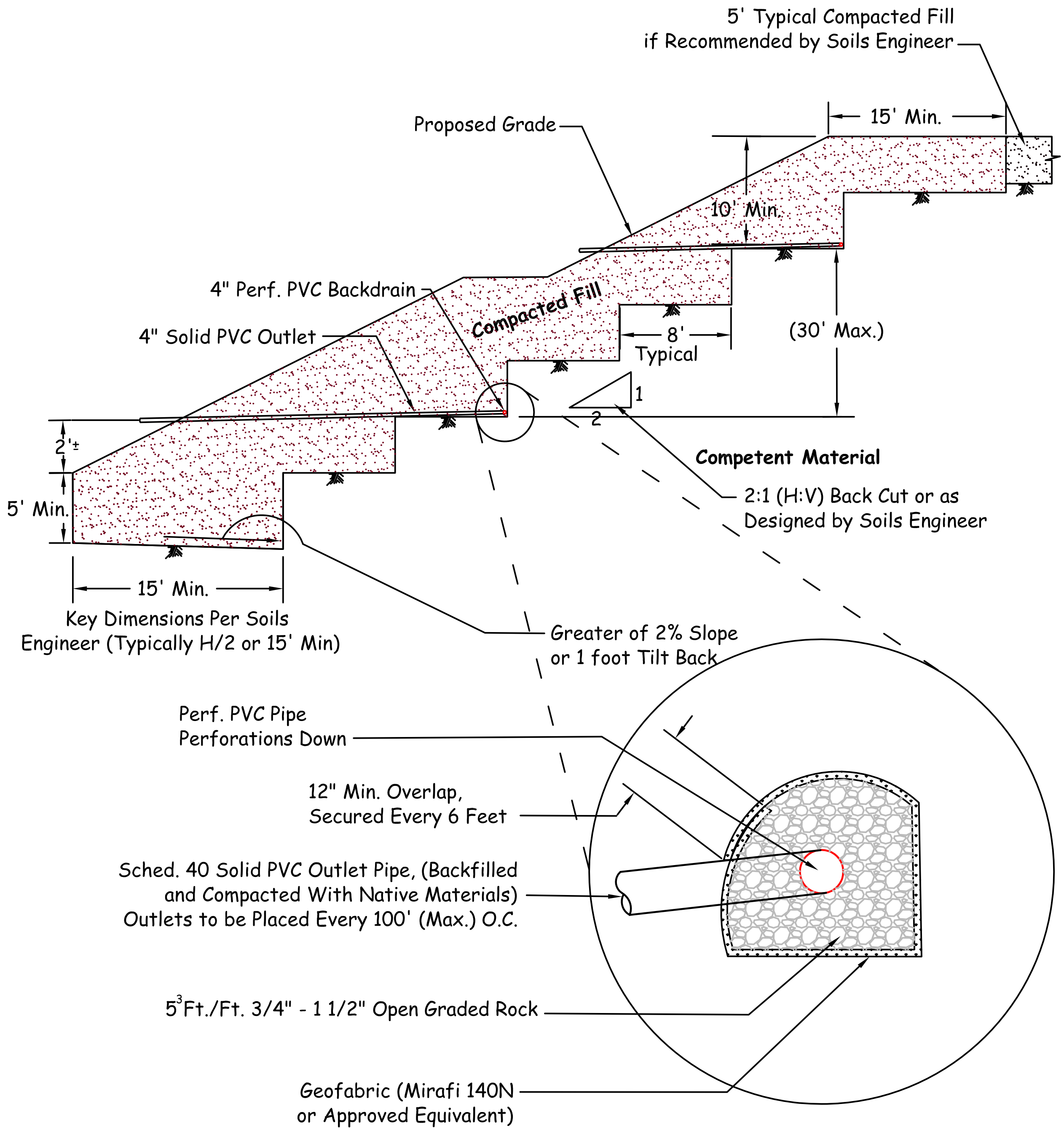
# TYPICAL BUTTRESS DETAIL

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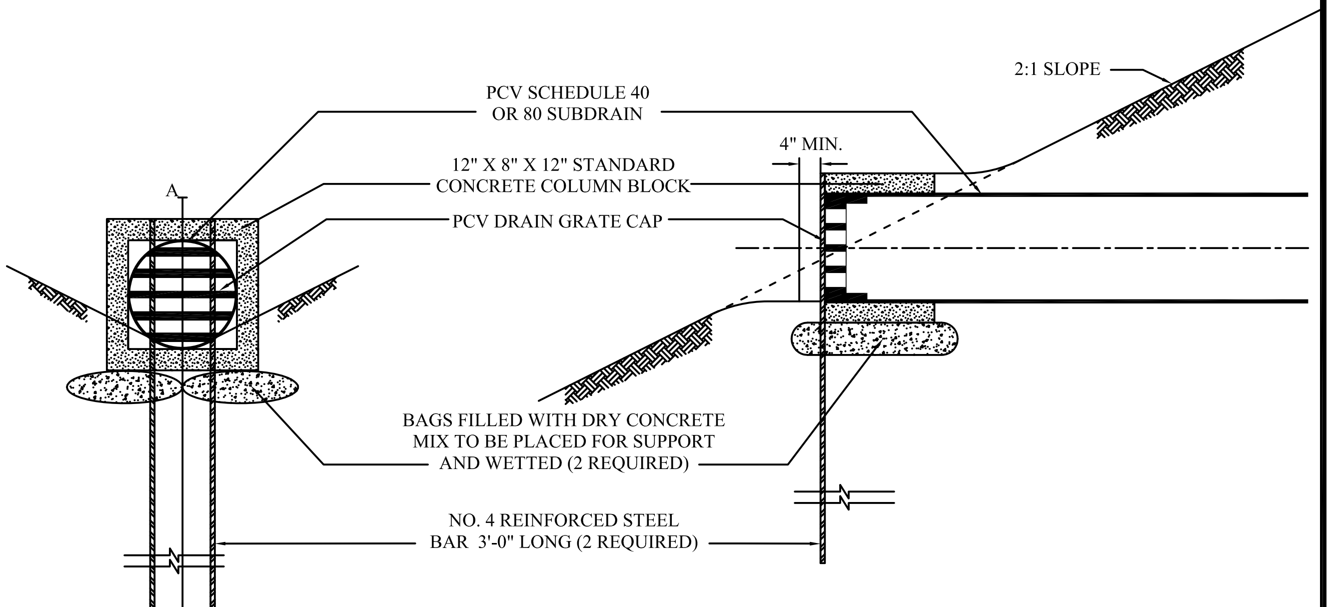
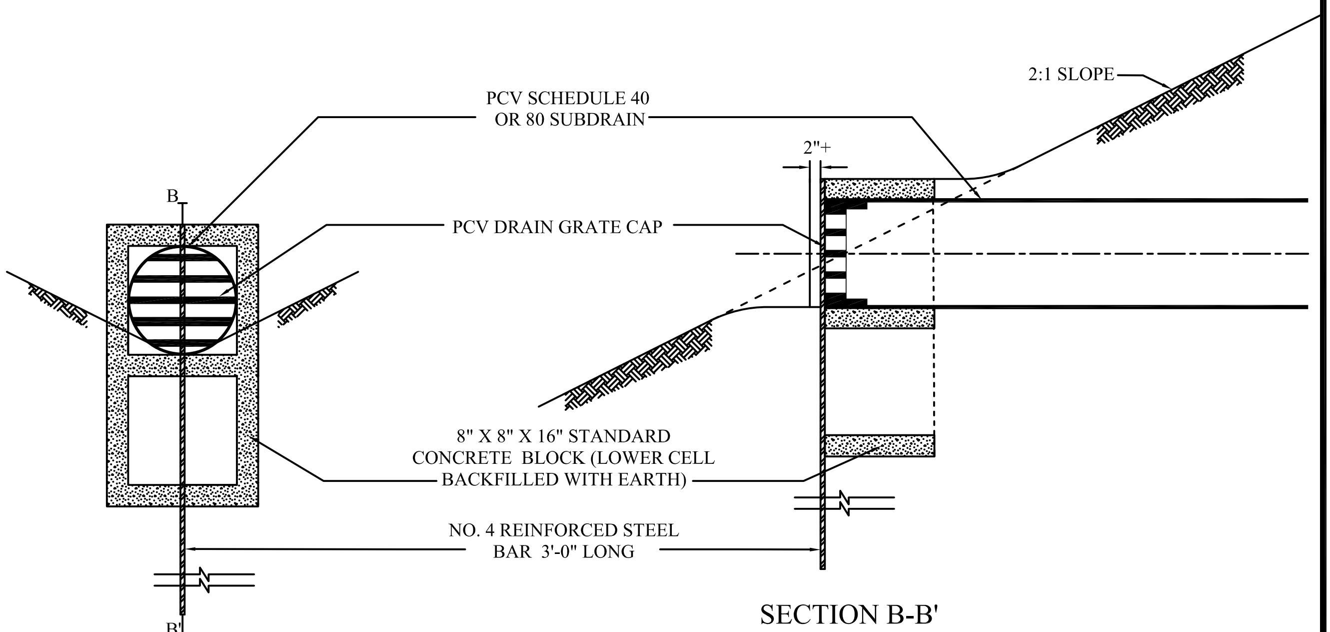
# TYPICAL STABILIZATION FILL DETAIL

# SUBDRAIN OUTLET MARKER -6" & 8" PIPE




SECTION A-A'

# SUBDRAIN OUTLET MARKER -4" PIPE




SECTION B-B'

NOT TO SCALE

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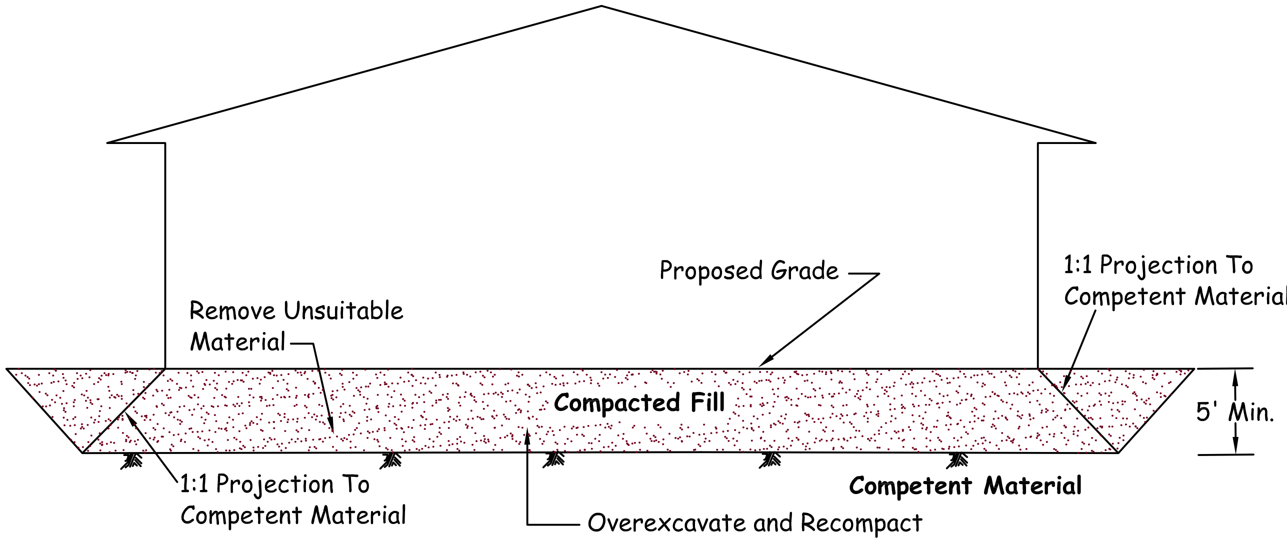
**SUBDRAIN OUTLET MARKER DETAIL**

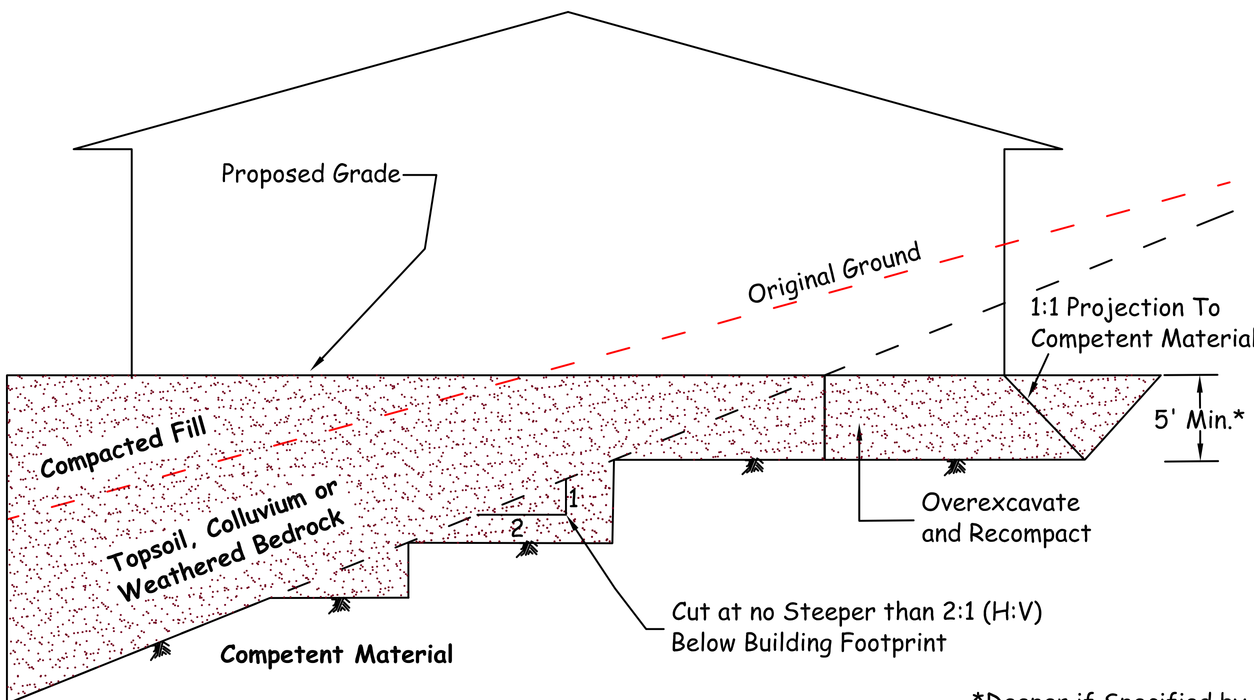
## Cut Lot
(Exposing Unsuitable Soils at Design Grade)




Note 1: Removal Bottom Should be Graded With Minimum 2% Fall Towards Street or Other Suitable Area (as Determined by Soils Engineer) to Avoid Ponding Below Building

Note 2: Where Design Cut Lots are Excavated Entirely Into Competent Material, Overexcavation May Still be Required for Hard-Rock Conditions or for Materials With Variable Expansion Characteristics.

## Cut/Fill Transition Lot




*Deeper if Specified by Soils Engineer

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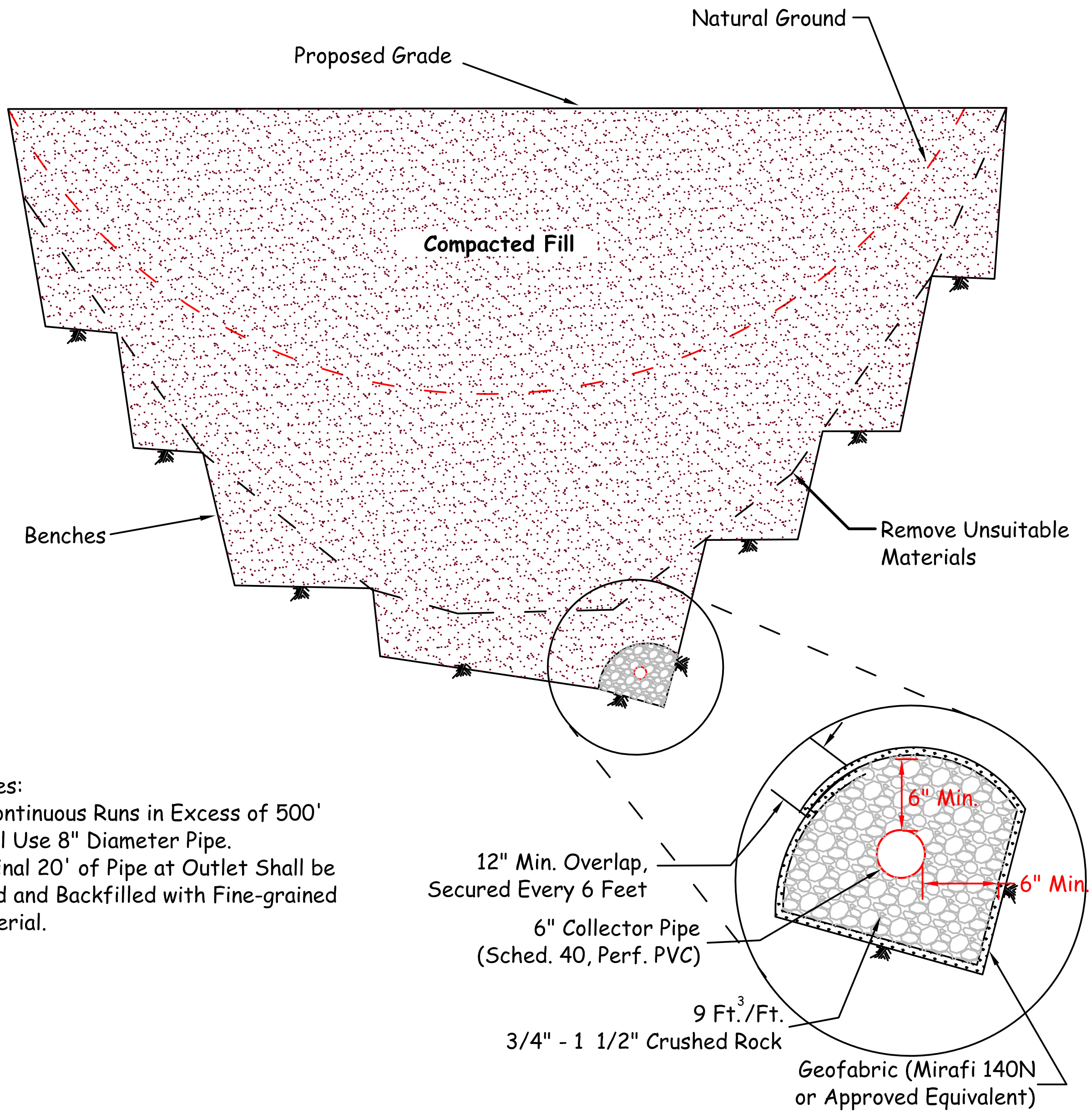
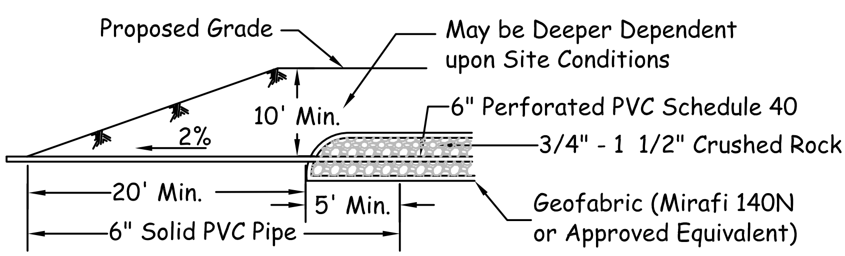
# CUT AND TRANSITION LOT OVEREXCAVATION DETAIL




Notes:
1) Continuous Runs in Excess of 500' Shall Use 8" Diameter Pipe.
2) Final 20' of Pipe at Outlet Shall be Solid and Backfilled with Fine-grained Material.

## Proposed Outlet Detail

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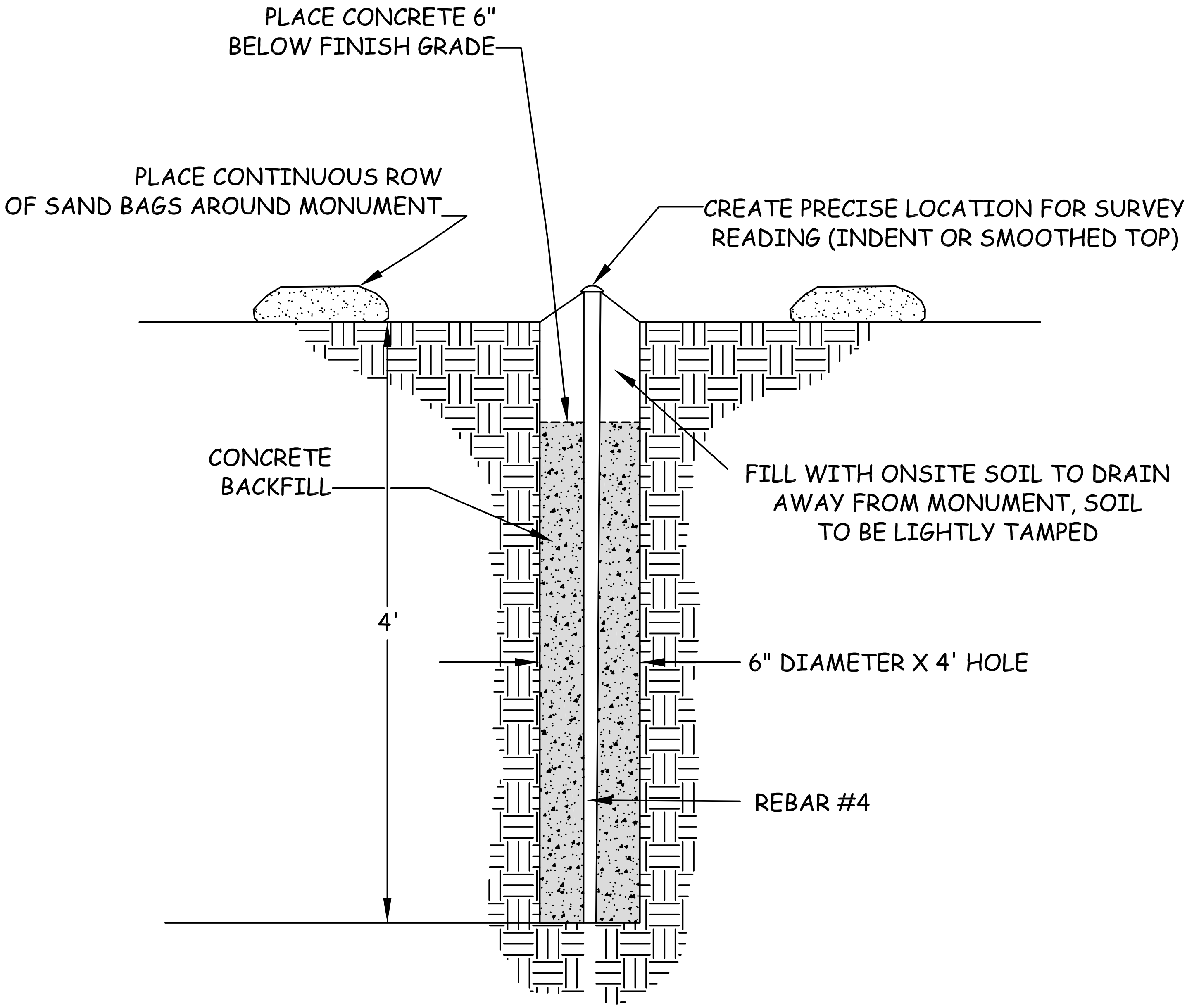
# CANYON SUBDRAINS




NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS

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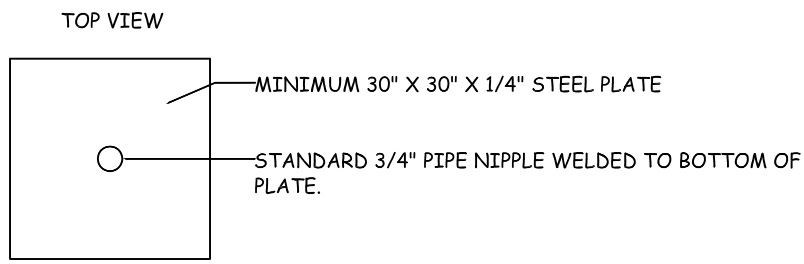
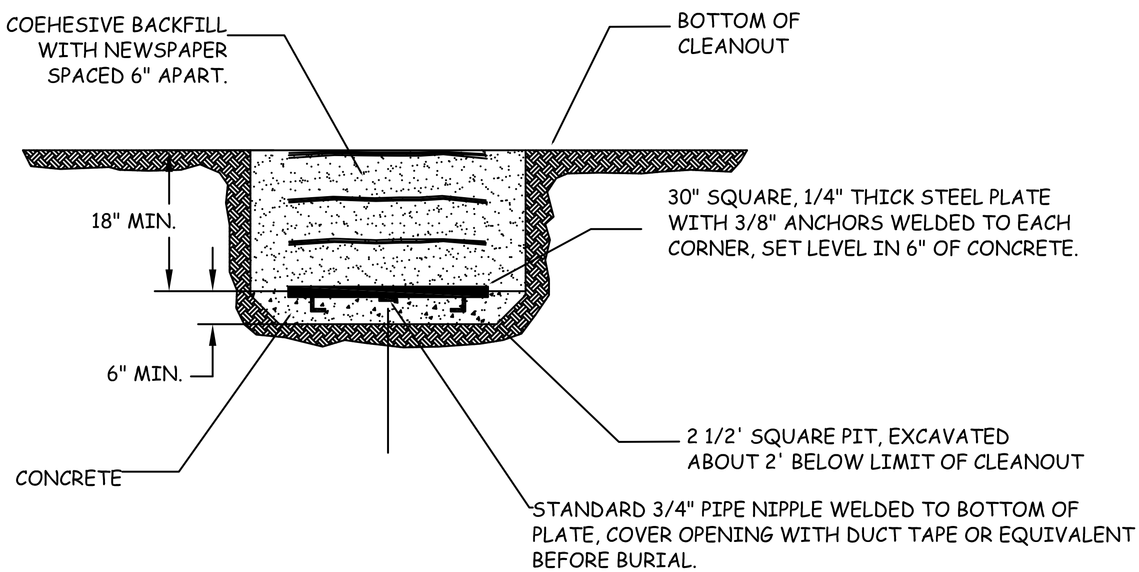
# TYPICAL SURFACE SETTLEMENT MONUMENT




1. SURVEY FOR HORIZONTAL AND VERTICAL LOCATION TO NEAREST .01 INCH PRIOR TO BACKFILL USING KNOW LOCATIONS THAT WILL REMAIN INTACT DURING THE DURATION OF THE MONITORING PROGRAM. KNOW POINTS EXPLICITELY NOT ALLOWED ARE THOSE LOCATED ON FILL OR THAT WILL BE DESTROYED DURING GRADING.
2. IN THE EVENT OF DAMAGE TO SETTLEMENT PLATE DURING GRADING, CONTRACTOR SHALL IMMEDIATELY NOTIFY THE GEOTECHNICAL ENGINEER AND SHALL BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.
3. DRILL TO RECOVER AND ATTACH RISER PIPE.

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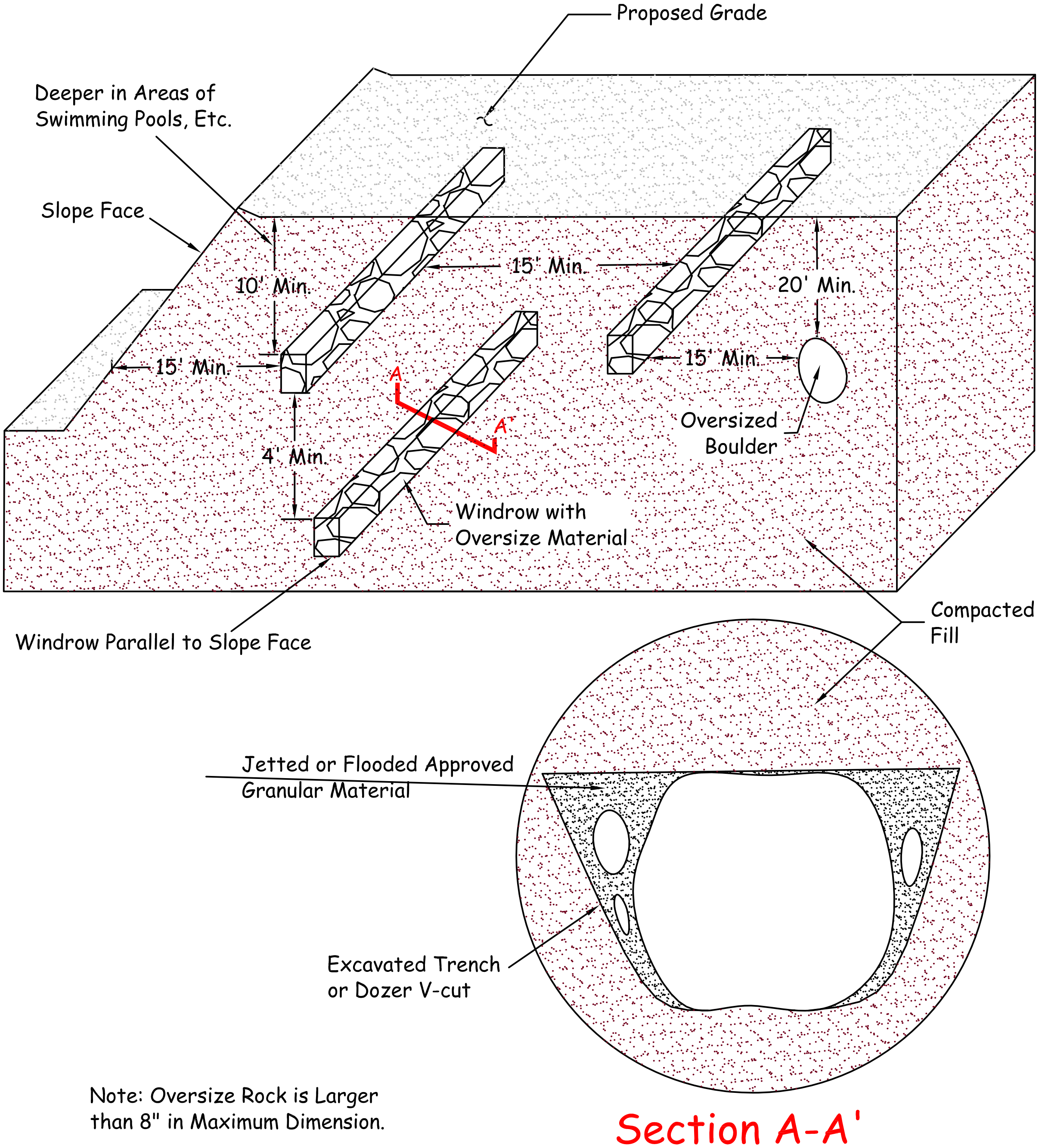
# TYPICAL SETTLEMENT PLATE AND RISER

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# OVERSIZE ROCK DISPOSAL DETAIL

# Attachment E: 2 Year Storm Hydrology


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27042 Towne Centre Drive, Suite 110, Foothill Ranch, CA 92610

**OUR LADY OF PEACE - HYDROLOGY MAP EXISTING RESIDENTIAL**

11/10/2025

```
****************************************************************************
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            (Reference: 1986 ORANGE COUNTY HYDROLOGY CRITERION)
         (c) Copyright 1983-2015 Advanced Engineering Software (aes)
             Ver. 22.0  Release Date: 07/01/2015  License ID 1673

                           Analysis prepared by:





 ************************** DESCRIPTION OF STUDY **************************
* OLP RESIDENTIAL                                                          *
* EXISTING                                                                 *
* 2 YEAR                                                                   *
 **************************************************************************

  FILE NAME: RES2EX.DAT
  TIME/DATE OF STUDY: 13:59 11/08/2025
============================================================================
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
============================================================================
                    --*TIME-OF-CONCENTRATION MODEL*--

  USER SPECIFIED STORM EVENT(YEAR) =    2.00
  SPECIFIED MINIMUM PIPE SIZE(INCH) =  18.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.95
  *DATA BANK RAINFALL USED*
  *ANTECEDENT MOISTURE CONDITION (AMC) II ASSUMED FOR RATIONAL METHOD*

  *USER-DEFINED STREET-SECTIONS FOR COUPLED PIPEFLOW AND STREETFLOW MODEL*
     HALF-  CROWN TO   STREET-CROSSFALL:   CURB  GUTTER-GEOMETRIES:  MANNING
     WIDTH  CROSSFALL  IN-  / OUT-/PARK-  HEIGHT  WIDTH  LIP   HIKE  FACTOR
NO.   (FT)     (FT)    SIDE / SIDE/ WAY    (FT)    (FT)  (FT)  (FT)    (n)
===  =====  =========  =================  ======  ===== ====== ===== =======
  1   30.0     20.0    0.018/0.018/0.020   0.67    2.00 0.0312 0.167 0.0150

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.00 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint =  6.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*
  *USER-SPECIFIED MINIMUM TOPOGRAPHIC SLOPE ADJUSTMENT NOT SELECTED

****************************************************************************
  FLOW PROCESS FROM NODE    100.00 TO NODE    101.00 IS CODE =  21
```

```
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
  >>USE TIME-OF-CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA<<
============================================================================
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   270.00
  ELEVATION DATA: UPSTREAM(FEET) =    164.50  DOWNSTREAM(FEET) =    160.00

  Tc = K*[(LENGTH** 3.00)/(ELEVATION CHANGE)]**0.20
  SUBAREA ANALYSIS USED MINIMUM Tc(MIN.) =   11.177
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.427
  SUBAREA Tc AND LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS   Tc
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN  (MIN.)
  NATURAL POOR COVER
  "BARREN"                   C        0.46      0.25     1.000    91   11.18
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  1.000
  SUBAREA RUNOFF(CFS) =      0.49
  TOTAL AREA(ACRES) =      0.46   PEAK FLOW RATE(CFS) =      0.49

****************************************************************************
  FLOW PROCESS FROM NODE    101.00 TO NODE    102.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    160.00  DOWNSTREAM(FEET) =    155.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =   275.00   CHANNEL SLOPE =  0.0182
  CHANNEL BASE(FEET) =  100.00   "Z" FACTOR =   5.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   1.00
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.005
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  NATURAL POOR COVER
  "BARREN"                   C        1.35      0.25     1.000    91
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  1.000
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.98
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   0.49
  AVERAGE FLOW DEPTH(FEET) =   0.02   TRAVEL TIME(MIN.) =   9.40
  Tc(MIN.) =   20.57
  SUBAREA AREA(ACRES) =     1.35       SUBAREA RUNOFF(CFS) =    0.92
  EFFECTIVE AREA(ACRES) =      1.81     AREA-AVERAGED Fm(INCH/HR) =   0.25
  AREA-AVERAGED Fp(INCH/HR) =   0.25  AREA-AVERAGED Ap =   1.00
  TOTAL AREA(ACRES) =        1.8         PEAK FLOW RATE(CFS) =       1.23

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.02   FLOW VELOCITY(FEET/SEC.) =   0.61
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    102.00 =     545.00 FEET.
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    102.00 TO NODE    103.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   155.00  DOWNSTREAM(FEET) =   152.00
  FLOW LENGTH(FEET) =   295.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   4.2 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   3.92
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       1.23
  PIPE TRAVEL TIME(MIN.) =   1.25    Tc(MIN.) =   21.83
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    103.00 =     840.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    103.00 TO NODE    103.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  MAINLINE Tc(MIN.) =   21.83
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  0.971
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  COMMERCIAL                 C       1.82      0.25     0.100    69
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.100
  SUBAREA AREA(ACRES) =    1.82      SUBAREA RUNOFF(CFS) =    1.55
  EFFECTIVE AREA(ACRES) =      3.63   AREA-AVERAGED Fm(INCH/HR) =  0.14
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.55
  TOTAL AREA(ACRES) =        3.6       PEAK FLOW RATE(CFS) =       2.73
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        3.6  TC(MIN.) =     21.83
  EFFECTIVE AREA(ACRES) =      3.63  AREA-AVERAGED Fm(INCH/HR)=  0.14
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap = 0.549
  PEAK FLOW RATE(CFS)   =      2.73
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```

## LEGEND

DRAINAGE BOUNDARY

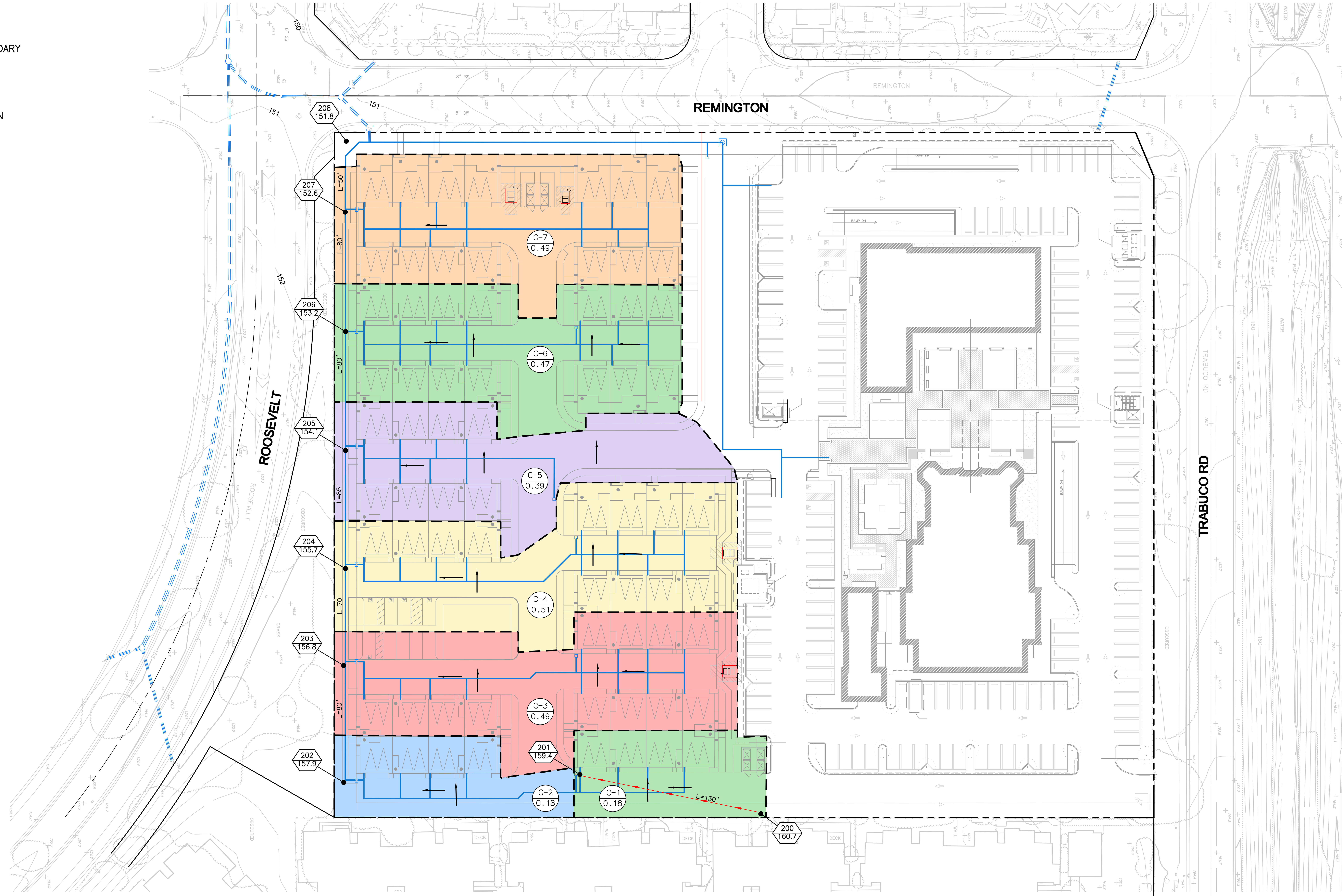
L=275' FLOW PATH

STORM DRAIN

EX STORM DRAIN

200 160.7 NODE

C-1 0.18 AREA




30 15 0 30 60

SCALE: 1"=30'

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11/10/2025

Plotted: Nov 10, 2025 – 8:30am by: Jvtwist    Drawing Name: N:\20.151.000\DLV\REPORTS\H&H\ENT\OLP Hydro Map–PROP Res.dwg

```
****************************************************************************
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              Ver. 22.0  Release Date: 07/01/2015  License ID 1673

                           Analysis prepared by:






 ************************** DESCRIPTION OF STUDY **************************
* OLP RESIDENTIAL                                                          *
* PROPSED                                                                  *
* 2 YEAR                                                                   *
 **************************************************************************

  FILE NAME: RES2PR.DAT
  TIME/DATE OF STUDY: 14:08 11/08/2025
============================================================================
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
============================================================================
                    --*TIME-OF-CONCENTRATION MODEL*--

  USER SPECIFIED STORM EVENT(YEAR) =    2.00
  SPECIFIED MINIMUM PIPE SIZE(INCH) =  18.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.95
  *DATA BANK RAINFALL USED*
  *ANTECEDENT MOISTURE CONDITION (AMC) II ASSUMED FOR RATIONAL METHOD*

  *USER-DEFINED STREET-SECTIONS FOR COUPLED PIPEFLOW AND STREETFLOW MODEL*
     HALF-  CROWN TO   STREET-CROSSFALL:   CURB  GUTTER-GEOMETRIES:  MANNING
     WIDTH  CROSSFALL  IN-  / OUT-/PARK-  HEIGHT  WIDTH  LIP   HIKE  FACTOR
NO.   (FT)     (FT)    SIDE / SIDE/ WAY    (FT)    (FT)  (FT)  (FT)    (n)
===  =====  =========  =================  ======  ===== ====== ===== =======
  1   30.0     20.0    0.018/0.018/0.020   0.67    2.00 0.0312 0.167 0.0150

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.00 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint =  6.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*
  *USER-SPECIFIED MINIMUM TOPOGRAPHIC SLOPE ADJUSTMENT NOT SELECTED

****************************************************************************
  FLOW PROCESS FROM NODE    200.00 TO NODE    201.00 IS CODE =  21
```

```
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
  >>USE TIME-OF-CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA<<
============================================================================
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   130.00
  ELEVATION DATA: UPSTREAM(FEET) =    160.70  DOWNSTREAM(FEET) =    159.40

  Tc = K*[(LENGTH** 3.00)/(ELEVATION CHANGE)]**0.20
  SUBAREA ANALYSIS USED MINIMUM Tc(MIN.) =    5.703
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  2.099
  SUBAREA Tc AND LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS   Tc
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN  (MIN.)
  APARTMENTS                 C        0.18      0.25     0.200    69    5.70
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.200
  SUBAREA RUNOFF(CFS) =      0.33
  TOTAL AREA(ACRES) =      0.18   PEAK FLOW RATE(CFS) =      0.33

****************************************************************************
  FLOW PROCESS FROM NODE    201.00 TO NODE    202.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   159.40  DOWNSTREAM(FEET) =   157.90
  FLOW LENGTH(FEET) =   160.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   2.3 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   2.58
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       0.33
  PIPE TRAVEL TIME(MIN.) =   1.03    Tc(MIN.) =    6.74
  LONGEST FLOWPATH FROM NODE    200.00 TO NODE    202.00 =     290.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    202.00 TO NODE    202.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  MAINLINE Tc(MIN.) =    6.74
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.907
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  APARTMENTS                 C        0.18      0.25     0.200    69
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.200
  SUBAREA AREA(ACRES) =    0.18      SUBAREA RUNOFF(CFS) =    0.30
  EFFECTIVE AREA(ACRES) =      0.36   AREA-AVERAGED Fm(INCH/HR) =  0.05
```

```
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.20
  TOTAL AREA(ACRES) =        0.4       PEAK FLOW RATE(CFS) =       0.60

****************************************************************************
  FLOW PROCESS FROM NODE    202.00 TO NODE    203.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   157.90  DOWNSTREAM(FEET) =   156.80
  FLOW LENGTH(FEET) =    80.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   2.7 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   3.56
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       0.60
  PIPE TRAVEL TIME(MIN.) =   0.37    Tc(MIN.) =    7.11
  LONGEST FLOWPATH FROM NODE    200.00 TO NODE    203.00 =     370.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    203.00 TO NODE    203.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  MAINLINE Tc(MIN.) =    7.11
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.849
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  APARTMENTS                 C        0.49      0.25     0.200    69
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.200
  SUBAREA AREA(ACRES) =    0.49      SUBAREA RUNOFF(CFS) =    0.79
  EFFECTIVE AREA(ACRES) =      0.85   AREA-AVERAGED Fm(INCH/HR) =  0.05
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.20
  TOTAL AREA(ACRES) =        0.9       PEAK FLOW RATE(CFS) =       1.38

****************************************************************************
  FLOW PROCESS FROM NODE    203.00 TO NODE    204.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   156.80  DOWNSTREAM(FEET) =   155.70
  FLOW LENGTH(FEET) =    70.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   4.0 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   4.73
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       1.38
```

```
  PIPE TRAVEL TIME(MIN.) =   0.25    Tc(MIN.) =    7.36
  LONGEST FLOWPATH FROM NODE    200.00 TO NODE    204.00 =     440.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    204.00 TO NODE    204.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  MAINLINE Tc(MIN.) =    7.36
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.813
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  APARTMENTS                  C        0.51      0.25     0.200    69
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.200
  SUBAREA AREA(ACRES) =    0.51      SUBAREA RUNOFF(CFS) =    0.81
  EFFECTIVE AREA(ACRES) =      1.36   AREA-AVERAGED Fm(INCH/HR) =  0.05
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.20
  TOTAL AREA(ACRES) =        1.4       PEAK FLOW RATE(CFS) =       2.16

****************************************************************************
  FLOW PROCESS FROM NODE    204.00 TO NODE    205.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   155.70  DOWNSTREAM(FEET) =   154.10
  FLOW LENGTH(FEET) =    85.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   4.8 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   5.76
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       2.16
  PIPE TRAVEL TIME(MIN.) =   0.25    Tc(MIN.) =    7.60
  LONGEST FLOWPATH FROM NODE    200.00 TO NODE    205.00 =     525.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    205.00 TO NODE    205.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  MAINLINE Tc(MIN.) =    7.60
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.779
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  APARTMENTS                  C        0.39      0.25     0.200    69
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.200
```

```
  SUBAREA AREA(ACRES) =    0.39      SUBAREA RUNOFF(CFS) =    0.61
  EFFECTIVE AREA(ACRES) =      1.75   AREA-AVERAGED Fm(INCH/HR) =  0.05
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.20
  TOTAL AREA(ACRES) =        1.8       PEAK FLOW RATE(CFS) =       2.72

****************************************************************************
  FLOW PROCESS FROM NODE    205.00 TO NODE    206.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   154.10  DOWNSTREAM(FEET) =   153.20
  FLOW LENGTH(FEET) =    80.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   6.1 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   5.11
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       2.72
  PIPE TRAVEL TIME(MIN.) =   0.26    Tc(MIN.) =    7.87
  LONGEST FLOWPATH FROM NODE    200.00 TO NODE    206.00 =     605.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    206.00 TO NODE    206.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  MAINLINE Tc(MIN.) =    7.87
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.745
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  APARTMENTS                 C        0.47      0.25     0.200    69
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.200
  SUBAREA AREA(ACRES) =    0.47      SUBAREA RUNOFF(CFS) =    0.72
  EFFECTIVE AREA(ACRES) =      2.22   AREA-AVERAGED Fm(INCH/HR) =  0.05
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.20
  TOTAL AREA(ACRES) =        2.2       PEAK FLOW RATE(CFS) =       3.39

****************************************************************************
  FLOW PROCESS FROM NODE    206.00 TO NODE    207.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   153.20  DOWNSTREAM(FEET) =   152.60
  FLOW LENGTH(FEET) =    80.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   7.7 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   4.68
```

```
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       3.39
  PIPE TRAVEL TIME(MIN.) =   0.28    Tc(MIN.) =    8.15
  LONGEST FLOWPATH FROM NODE    200.00 TO NODE    207.00 =     685.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    207.00 TO NODE    207.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  MAINLINE Tc(MIN.) =    8.15
  *   2 YEAR RAINFALL INTENSITY(INCH/HR) =  1.710
  SUBAREA LOSS RATE DATA(AMC  II):
   DEVELOPMENT TYPE/      SCS SOIL   AREA      Fp         Ap     SCS
       LAND USE            GROUP   (ACRES)  (INCH/HR)  (DECIMAL)  CN
  APARTMENTS                 C        0.49      0.25     0.200    69
  SUBAREA AVERAGE PERVIOUS LOSS RATE, Fp(INCH/HR) =  0.25
  SUBAREA AVERAGE PERVIOUS AREA FRACTION, Ap =  0.200
  SUBAREA AREA(ACRES) =    0.49      SUBAREA RUNOFF(CFS) =    0.73
  EFFECTIVE AREA(ACRES) =      2.71   AREA-AVERAGED Fm(INCH/HR) =  0.05
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap =  0.20
  TOTAL AREA(ACRES) =        2.7       PEAK FLOW RATE(CFS) =       4.05

****************************************************************************
  FLOW PROCESS FROM NODE    207.00 TO NODE    208.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   152.60  DOWNSTREAM(FEET) =   151.80
  FLOW LENGTH(FEET) =    50.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 18.000
  DEPTH OF FLOW IN  18.0 INCH PIPE IS   6.9 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   6.48
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       4.05
  PIPE TRAVEL TIME(MIN.) =   0.13    Tc(MIN.) =    8.28
  LONGEST FLOWPATH FROM NODE    200.00 TO NODE    208.00 =     735.00 FEET.
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        2.7  TC(MIN.) =      8.28
  EFFECTIVE AREA(ACRES) =      2.71  AREA-AVERAGED Fm(INCH/HR)=  0.05
  AREA-AVERAGED Fp(INCH/HR) =  0.25  AREA-AVERAGED Ap = 0.200
  PEAK FLOW RATE(CFS)   =       4.05
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```