Appendix E

Geotechnical Study

GEOTECHNICAL STUDY IN SUPPORT OF EIR

for

BUENA PARK RESIDENTIAL DEVELOPMENT
BUENA PARK, CA

Prepared For:

MGP XII BUENA PARK CENTER, LLC
4365 EXECUTIVE DRIVE, SUITE 1400
SAN DIEGO, CALIFORNIA

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Langan Project No.: 700105101

LANGAN

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1.0 INTRODUCTION

Langan Engineering and Environmental Services, Inc. (LANGAN) has prepared this report at the authorization of MGP XII Buena Park Center, LLC to provide geotechnical analysis in support of an entitlement submission for the proposed Village at Buena Park residential development (Project) in Buena Park, California (Site). Provided herein is a brief site description, a description of the proposed project, an overview of available geotechnical information, and a preliminary geotechnical analysis for the proposed development. Elevations referenced herein are in feet with respect to North American Vertical Datum of 1988 (NAVD88) unless noted otherwise.

2.0 PROJECT OVERVIEW

2.1 Site Description

The Site is an approximately 25-acre property on the southeast corner of the intersection of Stanton Avenue and La Palma Avenue in the city of Buena Park, California, see Figure 1. The property is bordered by La Palma Avenue to the north, parking lots and the Buena Park Downtown Mall on the east, Mall Avenue on the south and Stanton Avenue to the west.

Based a 2021 Topographic survey provided to us, the existing site grades range from an approximate elevation of 83 feet on the west side of the project limits and rise to an approximate elevation of 89 feet at the east side of the project.

The Site is currently occupied by a former Sears Store and Service Center with associated parking lots. There is a truck ramp leading down into the shipping and receiving area of the former Sears building which is located below ground. Based on the original architectural plans for the former Sears store (Stiles & Robert Clements, 1958), the basement for the Sears has a top of finished floor elevation that is about 13 feet below ground surface.

2.2 Proposed Development

Our understanding of the project is based on the 18 November 2021 Entitlement Submittal by TCA Architects. Based on the submittal, the Project involves the demolition of the existing former Sears store and service center and construction of a new residential development. The development includes four (4) multi-story apartment buildings to be built in phases on four (4) parcels including apartments, lobbies, amenity spaces courtyards, roof deck and a parking garage. The development also includes two (2) parcels containing townhomes with a private garage, a shared amenity building and common open spaces. The development will also include a park, new streets, sidewalks, and street parking. The apartment buildings are proposed to be 5- to 7-stories tall and the town homes are proposed to be 3-stories tall.

3.0 AVAILABLE INFORMATION REVIEW

3.1 Document Review

LANGAN reviewed reports, maps and other public available information from the United States Geological Survey (USGS), California Geological Survey (CGS), City of Buena Park, Federal Emergency Management Agency (FEMA), and Geologic Energy Management Division's (CalGEM). A summary of the available information reviewed is provided below.

3.2 Site Development History

Based on review of historical United State Geologic Survey Topography maps dated 1935, 1949, 1950, 1969, and 1974, the Site was oringal generally flat land referred to as Los Coyotes. Stanton

Avenue and La Palma Avenue are depicted on the maps between 1935 and 1950. Then sometime between 1950 and 1965 the Former Sears building was built.

3.3 Regional and Local Geologic Setting

The subject site is located at the eastern end of the Los Angeles Basin, a northwest trending, alluviated lowland situated at the north end of the Peninsular Ranges geomorphic province of coastal southern California. This basin, which is the surface expression of a deep structural trough, has been subdivided into four primary structural blocks distinguished from one another by contrasting basement rock types and stratigraphy. These structural blocks are generally separated by zones of faulting along which movement has been occurring intermittently since middle Miocene time (Yerkes and others, 1965). The site is located near the middle of the Central Block of the Los Angeles Basin, a wedge-shaped area that extends from the Santa Monica Mountains at its northwest end to the San Joaquin Hills at its southeast end.

3.4 Geologic Hazards Review

Our geologic hazard review was performed in general accordance with CGS Special Publication 117A, "Guidelines for Evaluating and Mitigating Seismic Hazards in California." The following subsections present the results of our review of the potential geologic hazards as they pertain to the Site.

- Regional Faulting - Recognized and mapped faults that are located within a 100-kilometer (km) radius of the Site based on the CGS "2010 Fault Activity Map of California" and "2014 USGS National Seismic Hazards Maps" are shown on Figures 2A and 2B. Based on our review, the closest known fault to the Site is the Anaheim Fault located approximately 1.5 miles (2.4 km) south of the Site. The next closest faults are the Elysian Park (lower CFM) Fault located approximately 1.8 miles (3.0 km) north of the west and Puente Hills (Coyote Hills) Fault located approximately 2.2 miles (3.6 km) north of the Site. See Table A.1 in Appendix A.

- Regional Seismicity - A search of the Uniform California Earthquake Rupture Forecast (UCERF3), accessed on 11 October 2021, using a web-based Earthquake Archive Search and URL Builder tool, found that 36 earthquakes with magnitudes greater than 5.0 have occurred within a 100-km radius of the Site since 1800.

  The Site is located in an active seismic area that has historically been affected by generally moderate to occasionally high levels of ground motion. Therefore, the proposed development will probably experience moderate to occasionally high levels of ground motion from nearby faults as well as ground motions from other active seismic areas of the southern California region

- Surface Rupture – The Site is not within a mapped Alquist-Priolo (AP) Earthquake Fault Zone as defined by the AP Act, as shown in Figure 3. Geologic review does not indicate the presence of active surface faulting within or adjacent to the Site.

- Liquefaction – Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium-dense sand and gravel, and low-plasticity silts below the groundwater table. According to the CGS, the Site is within a mapped, currently established liquefaction-potential investigation zone, as shown in

Figure 3. Liquefaction potential at the site and recommendations for mitigation are discussed further in this this report.

- Historical High Groundwater – Based on the SHZR 003, the historically highest groundwater at the Site is about 10 feet below ground surface, as shown in Figure 4. The Department of Water Resources database contains measurements from a well approximately 0.25 miles southeast of the Site. The groundwater depth measurements from 2008 to 2010 ranged from 5 to 25 feet below ground surface corresponding to elevations ranging from elevation 63 to elevation.
- Landslides – According to the CGS and SHZR 003 Landslide Inventory, the Site is not within a mapped Earthquake-Induced Landslide Hazard Zone or a mapped landslide area, as shown in Figure 3.
- Seismic-Induced Ground Deformations – Seismic-induced ground deformations include ground-surface settlement, differential settlement and lateral spreading resulting from liquefaction and cyclic densification of unsaturated sands and gravels. The Site is mapped within a liquefaction potential investigation zone and, based on the 2035 Buena Park General Plan Environmental Impact Report, the Site is located within a zone of moderate differential settlement potential. Therefore, differential seismic-induced ground deformations are expected, mitigations of this hazard are described later in this report.
- Flood Mapping – According to the City of Buena Park's Safety Element, within the general plan, the Site is within a mapped 0.2 percent annual chance flood hazard 500-year flood zone. Based on FEMA's National Flood Hazard Layer FIRMette Number 06059C0126F (October 2020), the Site is inside an area of 0.2 percent annual chance flood; 1 percent annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile as shown in Figure 5.
- Tsunami and Seiche – According to information and maps available from the California Department of Conservation, the Site is not within a mapped tsunami inundation- hazard zone.
- Subsidence – Land subsidence may be induced from withdrawal of oil, gas, or water from wells. According to a search of the California Geologic Energy Management Division's (CalGEM) Well Finder online tool, no active oil, gas, or geothermal wells are mapped within the Site. According to our review of the available information from CalGEM, the Site is not considered to be subject to land subsidence from oil, gas, or water withdrawal from oil wells.
- Expansive Soils – Expansive soils occur when the moisture content in the soil causes swelling or shrinking as a result of cyclic wet/dry weather cycles, installation of irrigation systems, change in landscape plantings, or changes in grading. Swelling and shrinking soils can result in differential movement of structures including floor slabs and foundations, and site work including hardscape, utilities, and sidewalks. The 2019 CBC defines potentially expansive soils as soils with expansion indices (EI) greater than 20. Based on Buena Park General Plan Exhibit SAF-3 the Site is not located within a zone of moderately expansive soil potential, see Figure 6. Expansive soils are anticipated to have a less than significant impact on the Project.

3.5 Available Document Information

The following table summarizes reports and historic documents that were reviewed by LANGAN and that pertain or are in proximity to the Site:

Document	Summary of Document Findings
Clements Architects–Engineers, "Retail Store and Service Station Building for Sears" sheets 2 through 41.	Based on the plan set provided, the plans indicate that the basement floor is approximately 13 feet below existing grade and the loading dock approximately 17 feet below existing grade. From various elevation sections on the plans it seems the basement extents are under the entire portion of the west wing structure of the mall.

4.0 SUBSURFACE INVESTIGATION

LANGAN's geotechnical field investigation included six (6) borings, identified as LB-1 through LB-6, six (6) Cone Penetrometer Tests (CPTs), identified as LCPT-1 through LCPT-6, and one percolation test borehole, identified as LP-1. Borings were drilled by Martini Drilling using hollow-stem auger techniques with a truck-mounted CME-75 drill rig on 28 and 29 September 2021 to approximate depths of 36.5 to 51.5 feet. CPT's were performed by Kehoe Testing and Engineering on 28 September 2021. The borings and CPT's were performed under full-time observation of a LANGAN field engineer. Exploration locations are shown on Figure 7.

At select boring locations, bulk samples were collected from the upper 5 feet. Standard Penetration Tests (SPT[1]) were performed and samples were collected at typically 5-feet intervals. Ring samples were collected at select depths using a 3.0-inch-outer-diameter split-barrel California sampler lined with 2.42-inch-inner-diameter brass rings in accordance with ASTM D3550. Soil samples were visually examined and classified in the field in accordance with the Unified Soil Classification System (USCS). Upon completion, the borings were backfilled with cement-grout-slurry and patched with concrete, the surface was brought to approximately pre-existing condition. Excess soil cuttings were placed in 55-gallon drums. Boring logs are included in Appendix B.

The CPTs were performed using the guidelines of ASTM D5778 by hydraulically pushing a 1.4-inch-diameter cone-tipped probe into the ground. Electrical strain gauges within the cone continuously measured soil data for the entire depth advanced, including tip resistance at the cone tip and frictional resistance on the friction sleeve behind the cone. Copies of the CPT logs are provided in Appendix C.

Prior to performing the subsurface investigation, borings were located and marked by a field engineer from our office. Underground Service Alert of Southern California (DigAlert) was

[1] The Standard Penetration Test is a measure of the soil density and consistency. The SPT N-value is defined as the number of blows required to drive a 2-inch outer diameter split-barrel sampler 12-inches, after an initial penetration of 6 inches, using a 140-pound hammer free falling of a height of 30-inches (ASTM D1586).

contacted to locate and mark known public underground utilities present within the public rights-of-way.

4.1 Percolation Testing

Percolation testing was performed in boring LB-7 between depths of approximately 5 and 10 feet. The test was performed on 29 September 2021 under full-time observation of a LANGAN field engineer. The percolation test consisted of measuring the drop in water over time in a 2-inch diameter polyvinyl chloride (PVC) pipe with a solid end cap and 5 feet of slotted PVC section at the bottom. A sand filter pack was poured at the base and around the PVC slotted section.

The percolation test was performed in general accordance with the County of Orange, north and central portions of Orange County "Exhibit 7.III - Technical Guidance Document for the preparation of conceptual/preliminary and/or project water quality management plans (WQMPs)" dated 20 December 2013. Percolation test results are provided in Appendix D. Listed below isthe measured percolation rate at the tested location. A minimum factor of safety of 2 is recommended to determine the design infiltration rate. Additional recommendations regarding site infiltration can be found in section 5.10

Table 1 – Percolation Results

Boring Location	Test Depth	Calculated Infiltration Rate (inch per hour)
LB-7	5 to 10 feet	0.53

4.2 Laboratory Testing

Select soil samples were select for laboratory testing to confirm index and measure strength properties of the soils. Laboratory testing was performed by GeoLogic Associates, Inc. and laboratory test results are attached in Appendix E. The laboratory test program included the following tests:

- Direct Shear – ASTM D3080
- Passing #200 Sieve – ASTM D1140
- Modified Proctor – ASTM D1557
- Expansion Index – ASTM D4829
- Electrical Resistivity – CTM 643
- Chloride Content – CTM 422
- Sulfate Content – CTM 417
- Soil pH – CTM 643

4.3 Subsurface Conditions

Based on the field exploration performed to date, the Site is underlain by undocumented fill overlying alluvial deposits. The undocumented fill generally consists of silty sands with trace amounts of clay. The alluvial deposits underlying the undocumented fill consists of interlayered sands, sands and silts, silty sands, silt, and clays. These layered alluvial deposits range in thickness from approximately 3 to 10 feet thick. More details on the undocumented fill and alluvial deposits are presented below with detailed boring logs included in Appendix B. See Figure 8A for generalized subsurface cross-sections A-A', B-B' and 8B for generalized subsurface cross-sections C-C', D-D'.

- **Fill:** We encountered fill within all borings. In most locations up to 5 feet of fill was encountered, however deeper fills (15 feet or more) should be expected adjacent to the former Sears basement walls and truck ramp. Existing fill consisted of very loose to medium dense silty sands with up to trace amounts of clay. SPT N-values recorded in the fill range from 2 to 10 blows per foot.

Direct Shear testing on remolded samples of the fill indicates the undocumented fill when recompacted to 90 percent has a peak and ultimate friction angle of 30 degrees and peak and ultimate cohesion of 100 pounds per square foot (psf). Expansion index testing indicates the undocumented fill has an expansion index of 0. Modified Proctor testing on collected near-surface soil was performed and indicates the maximum dry density and optimum moisture content of the fill when recompacted is 117 pounds per cubic foot (pcf) and 7.5 percent.

- **Alluvial Deposits:** Alluvial deposits were encountered underlying the undocumented fill, in all borings. The alluvial soils were observed to consist of stiff to very stiff silts and clays, and loose to medium dense sands, sands with silt, and silty sands. SPT N-values recorded in cohesive soils were from 6 blows per foot to 29 blows per foot. SPT N-values recorded in granular soils were from 4 blows per foot to 29 blows per foot.

  Laboratory tests performed on select samples measured percent fines content of 23, 29, 60, 84, and 97 percent passing the No. 200 sieve. Direct shear testing of one sample measured peak and ultimate cohesion of 300 psf and 100 psf with peak and ultimate internal friction angle of 31.5 degrees, respectively.

- **Groundwater:** Groundwater was encountered at depths ranging from 22 to 28 feet below ground surface. While not encountered during this investigation, localized shallower pockets of perched water should be anticipate in the sand layers above the silt areas during rainy periods. In addition, the regional groundwater levels are generally lower than historic highs, partly due to current drought conditions. Shallower groundwater levels, consistent with historic high groundwater levels, are feasible at the site. For design purposes, we recommend using a design groundwater level consistent with the historic high level of 10 feet below ground surface.

5.0 PRELIMINARY GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

Based on our subsurface investigation, preliminary engineering analyses, and laboratory testing the proposed development is considered feasible from a geotechnical engineering standpoint. The primary geotechnical issue associated with the proposed development is related to presence of potentially liquefiable soils and potential for liquefaction-induced settlement to occur during a moderate to major earthquake at the site. After ground improvement to bring the settlement related to liquefaction within tolerable limits, the proposed structures are anticipated to be supported on shallow foundations. Presented below are preliminary geotechnical evaluation and recommendations based on data obtained to date.

5.1 Expansive Soils

Based on the review of geotechnical reports for adjacent sites and our laboratory testing results, surficial soils at the Site had a very low expansion potential (EI=0).

5.2 Soil Corrosion

Based on laboratory results the surficial soils had a low water-soluble sulfate concentration. The mitigation of corrosive soils could include use of specific types of pipe, insulation, coatings, and cathodic protection or concrete admixtures. The potentially corrosive soils are anticipated to have a less than significant impact on the Project if mitigated through site specific testing and design in accordance with American Concrete Institute and Caltrans Corrosion Design Standards. See Appendix E for soil corrosivity lab results.

5.3 Seismic Design Criteria

Seismic design of structures can be designed in accordance with the provisions of ASCE/SEI 7-16 and 2019 California Building Code. Based on the available subsurface information, site-specific shear wave velocities from the Seismic CPTs, and in accordance with the seismic provisions of these codes, the soils underlying the site can be characterized as Site Class D. Our evaluation of the site class considered the residual strength of the potentially liquefiable layers to estimate the average shear wave velocity. As such, the following preliminary seismic design criteria are recommended for structures bearing on shallow foundations with ground improvement to mitigate the settlement hazard from liquefaction.

Table 2 – Recommended Seismic Design Criteria

Criteria	Site-Specific	Mapped (Site Class D)
MCE_R Spectral response acceleration at Short Periods, S_S	---	1.485g
MCE_R Spectral response acceleration at 1 second period, S_1	---	0.524g
Site-modified MCE_R Spectral Response Acceleration at Short Periods, S_{MS}	1.887g	1.485g
Site-modified MCE_R Spectral Response Acceleration at 1 second period, S_{M1}	1.777g	0.891g
Design Spectral Response Acceleration at short periods, S_{DS}	1.258g	0.990g
Design Spectral Response Acceleration at 1 second period, S_{D1}	1.185g	0.594g
MCE_G Peak Ground Acceleration, PGA_M	0.677g	0.696g

Notes:

1. Recommended site-specific S_{DS} taken as 90 percent of the maximum spectral acceleration value between 0.2 and 5 seconds per section 21.3 of ASCE 7-16.
2. Recommended site-specific S_{D1} governed by 3.0 x spectral acceleration value at 1.0 second.
3. Recommended mapped values are based on F_a and F_v of 1.0 and 1.7, respectively.

The recommend mapped values above assume Exception No. 2 of Section 11.4.8 of ASCE 7-16 will be used for seismic design, and that the structures will not be classified as a seismically isolated structure or structure with damping systems. Details of our site-specific response spectra development are summarized in Appendix F.

5.4 Preliminary Liquefaction Analysis

Liquefaction evaluation was performed in accordance with ASCE 7-16, and CGS SP-117A. For this analysis performed to estimate seismic-induced settlements under a Maximum Considered Earthquake geometric mean (MCE_G) level of ground shaking. We utilized a factor of safety (FS) of 1.0, a modal magnitude of 7.3 for a 2 percent probability of exceedance in 50-year earthquake, and a MCE_G Peak Ground Acceleration (PGA_M) of 0.70g. To evaluate the liquefaction potential at the site, we utilized the Boulanger and Idriss (2014) method the computer program *CLiq*, a soil profile from Boring LB-2 and CPT data from LCPT-1, with a groundwater depth of 22 feet. Based on the results of our analysis, there are potentially liquefiable soil layers at depths ranging from 25 to 35 feet below ground surface. When the liquefaction analysis is run with a groundwater depth of 10 feet based on historic high groundwater, there is another potentially liquefiable sand layer at 15 feet below ground surface. The results of this analysis are attached in Appendix G.

5.5 Seismically Induced Settlement

Based on existing soils anticipated settlements due to liquefaction range from about 3.5 to 4.5 inches with an additional 4 to 5 inches of settlement in the shallower soils due to cyclic densification. This results in a total calculated seismically induced settlements ranging from about 8 to 9 inches. Ground improvement as discussed below could be used to mitigate these seismic induced settlements. The results of this analysis are attached in Appendix G.

5.6 Ground Improvement

Due to the anticipated amount of settlement under seismic loading, ground improvement would be required to mitigate the potential impacts of the liquefaction at the site. Due the depth of the soils subject to liquefaction, ground improvement would need to extend to 30 to 35 feet below existing grade to reduce settlements to less than 2 inches. There are several methods that can be used to mitigate these soils, two common methods that would be suitable to this site include rammed aggregate piers (RAP) and drilled displacement columns (DDC).

Rammed Aggregate Piers (RAPs) or Vibropiers

RAPs are typically constructed by drilling a 30- to 36-inch-diameter hole with an auger and backfilling the hole with aggregate. The design depth of the RAP is based on the thickness of the improved layer. The bottom of the RAP is compacted with a hydraulic tamper before the column is filled with aggregate. The aggregate for the RAP is compacted in 12-inch lifts using a hydraulic tamper attached to an excavator. The purpose of the RAP is to reduce settlement and increase allowable bearing capacities of the soil by strengthening and creating a stiffer soil matrix. RAP can also be designed to resist uplift loads. RAP systems are generally installed under design-build contracts by specialized contractors and can have allowable bearing capacities of 6,000 to 8,000 psf.

A pre-construction RAP modulus load test program is recommended to verify the RAP elements and structural capacities, based on the contractor's construction means and methods. A full-scale static axial load test program should be developed and performed in accordance with appropriate portions of ASTM D1143 and ASTM D1194.

Vibropiers are constructed similar to rammed aggregate piers, but instead of a hydraulic tamper, the aggregate is densified using vibrations.

Drilled Displacement Columns

DDCs are constructed by using a displacement auger to create a soil shaft that is filled with Controlled Low Strength Material (CLSM) injected under pressure as the displacement auger is withdrawn from the hole. Because of the installation pressures, DDCs vary between 20 to 24 inches in diameter. Installation of DDCs produces minimal soil cuttings because the soil is displaced during column installation. The purpose of the drilled displacement columns is to eliminate densify the upper soils and transfer building loads to a deeper bearing stratum. The structure can then be supported on a shallow foundation bearing on the DDC columns. DDCs can also be constructed to resist uplift loads by drilling them deeper into the bearing layer and installing a central bar.

Because DDCs inject the CLSM under pressure, there is the potential for soil heave near the column. To eliminate the potential to damage nearby improvements, DDCs may need to be set back a horizontal distance from adjacent structures.

5.7 Preliminary Shallow Foundation Design – Apartment Buildings

Based on our experience with similar projects, we anticipate the apartment structures would have individual column loads of 500 to 800 kips and lighter column and wall loads from the town houses.

Following completion of ground improvement, a shallow foundation system (spread or continuous footings), bearing on properly prepared and compacted subgrade can be designed with a preliminary bearing pressures of 4,000 to 6,000 pounds per square foot (psf) or higher depending on the type of ground improvement used for liquefaction mitigation. Recommended allowable bearing values including both dead and live loads, and may be increased by one-third for transient loads such as wind or seismic forces.

Spread or continuous footings bearing on improved ground and design in accordance with the above parameters are anticipated to settle less than one-inch under static loading, with differential settlements of less than ½-inch between adjacent columns. Settlements under dynamic loading are calculated to be 1 to 2 inches depending on depth of ground improvement.

An ultimate coefficient of friction of 0.35 times the dead load forces may also be used between concrete and the supporting soils for lateral sliding resistance for concrete footings on approved subgrade. If additional lateral resistance is required, a passive earth pressure of 150 pounds per square foot per foot of depth may be used for lateral resistance against the side of the footing. The passive pressure may be increased by 1/3 for transient loading conditions, such as earthquake or wind; however, no increase should be applied to the friction factor.

Footing excavations should be performed using a backhoe bucket fitted with a smooth steel plate welded across the bucket teeth to minimize disturbance during excavation and to provide a smooth bearing surface. Any areas loosened by excavation should be over excavated and recompacted or replaced with structural fill, placed in accordance with the recommendations included in this report, or lean concrete.

The foundation subgrade should be observed and approved by a qualified Geotechnical Engineer prior to steel or concrete placement. The foundations should be constructed as soon as possible following subgrade approval. The contractor shall be responsible for maintaining the subgrade in its approved condition (i.e. free of water, debris, etc.) until the footing is constructed.

5.8 Preliminary Shallow Foundation Design – Townhouses

The proposed townhouses are anticipated to be up three-story structures with low column and wall loads. If the anticipated seismically induced settlements are acceptable, then a post-tensioned concrete slab and foundation system may be used for support of the proposed structure. The post-tensioned system should be designed by a structural engineer experienced in these types of structures and following the design criteria of the WRI/CRSI Design of Slab-on-Ground Foundations or Post Tension Institute DC 10.5.

Perimeter edge beams should be founded at a minimum depth of 12 inches below the lowest adjacent final ground surface. If a post-tensioned mat is used, the outer 12 inches should be thickened to provide a minimum embedment of 12 inches below lowest grade. Interior beams may be founded at a minimum depth of 12 inches below the tops of the finish floor slabs. Additional recommendations can be provided during final design if this system is selected for the townhouse support.

Our experience indicates post-tensioned slabs may be susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. During the construction of the post-tension foundation system, the concrete should be placed monolithically. Cold joints should not be allowed to form between the footings/grade beams and the slab during the construction of the post-tension foundation system unless specifically designed by the structural engineer.

5.9 Floor Slabs

Ground floor slabs can be designed as slabs-on-grade bearing on compacted fill after performing subgrade preparation discussed herein. The slabs can be designed using the following minimum recommendations.

- Subgrade modulus, k, equal to 125 pounds per cubic inch (pci)
- Four (4) inches of clean sand with a 15-mil polyethylene capillary break moisture barrier placed at mid-thickness and overlapping at least six (6) inches between joints.
- Steel reinforcing should be designed by the structural engineer and sufficient enough to meet shrinkage reinforcement limits and for soils with low to medium expansion potential.

5.10 Site Infiltration

As summarized earlier in this report, the published historic groundwater level at the site based on CalGEM mapping is approximately 10 feet below ground surface. Orange County Water Districts maps reporting 1997 Depth to Shallowmost Groundwater are based on the CalGEM mapping and similarly show a depth to groundwater of 10 feet below ground surface (OCWD, 2015).

The "Groundwater-Related Infiltration Feasibility Criteria in Appendix VIII of the "Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (OC, 2013) has a required minimum separation between the infiltrating surface and the seasonally high mounded groundwater. The separation is required to be 10 feet for infiltration devices that inject water below the subsurface. Based on the reported historic high groundwater levels and regional well information, these types of infiltration devices are likely not feasible at the site.

A minimum 5 feet separation is required for BMPS such as:

- Rain gardens and dispersion trenches (small, residential applications)
- Bioretention and planters
- Permeable Pavement
- Similar BMPs infiltrating over and extensive surface area and providing robust pretreatment or embedded treatment processes.

Depending on final site grading, these types of infiltration BMPs may be feasible some areas of the site.

The near surface soils at the site are generally silty sands, and based on one percolation test, the measured infiltration rate was approximately 0.5 inches per hour. Based on site subsurface conditions and amount of testing performed to date, we recommend using a factor of safety of 2 for design of any shallow infiltration BMPs.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Site Preparation

Prior to work on the Site, the former Sears building including the below grade level and truck loading ramp, the asphalt parking lot, the former service building, and site utilities that currently cross the proposed development footprint will need to be removed and disposed of in accordance with state and local regulations. Due to the anticipated demolition of the existing Sears Building including complete removal of the existing basement and loading dock, engineering fill in the order of 15 to 20 feet may be required to meet proposed grade.

Any foundation remnants associated with former site structures encountered within excavations should be fully removed, and any void spaces that may be created should be backfilled with approved compacted structural fill meeting the requirements discussed below.

After completion of excavation, including removal of any below-grade remnants, any soft, loose, or unsuitable soils identified by the geotechnical engineer should be removed and replaced with engineered fill. Any environmentally unsuitable soils encountered during the excavation process should be properly disposed of off-site in accordance with all state and local regulations.

6.2 Fill Material and Compaction Criteria

The onsite soils are geotechnically suitable for use on site as engineered fill. Fill material (imported or re-used) should be free of organic, frozen, and other deleterious materials and have a maximum particle size no greater than 6 inches. Imported fill should be free of deleterious materials, be non-corrosive, contain no more than 12 percent passing the no. 200 sieve by dry weight and have a plasticity index less than 7. Grain size distributions, Atterberg Limits, maximum dry density, and optimum water content determinations should be made on representative samples of the proposed fill material.

6.3 Site Drainage and Temporary Construction Dewatering

Proper drainage should be maintained at all times. Ponding or trapping of water in localized areas can cause differing moisture levels in the subsurface soil. Drainage should be directed away from the tops of slopes. Erosion protection and drainage control measures should be implemented during periods of inclement weather. During rainfall events, backfill operations may need to be restricted to allow for proper moisture control during fill placement.

6.4 Utility Support

Utilities can be supported on grade, bearing on native soils, or bearing on fill behind below-grade walls. Utilities should be backfilled with Caltrans Class 2 base, or equivalent, unless otherwise required by the utility owner. Bedding material should be separated from the underlying subgrade material with a geo-synthetic fabric (Mirafi 140N, or equivalent). The bedding material should extend at least 6-inches over the top of the utility, unless otherwise required by the utility owner. Utility trenches above pipe bedding should be backfilled using previously excavated soil (if suitable) or approved imported material.

7.0 FUTURE GEOTECHNICAL SERVICES AND INTERACTION

The conclusions and recommendations included in this report were developed in support of the analysis for the entitlement phase of the project. As the design progress, supplemental subsurface information should be obtained as part of design level evaluations. Additional services to be included are the following:

- Review of structural loading, preliminary foundation types, and refinement of settlement estimates;
- Design phase investigations and analysis.

8.0 LIMITATIONS

The conclusions and recommendations provided in this report result from our interpretation of the subsurface conditions encountered in a limited number of boring and CPTs, as well as architectural and structural information provided by MGP XII Buena Park Center. . Any proposed changes in structures or their locations should be brought to LANGAN's attention as soon as possible so that we can determine whether such changes affect our recommendations.

This report has been prepared to assist the Owner, Owner's representative, architect, and structural engineer in the design process during the entitlement phase of the project and is only applicable to the design of the specific project identified. The information in this report cannot be utilized or depended on by engineers or contractors who are involved in evaluations or designs of facilities (including underpinning, grouting, stabilization, etc.) on adjacent properties which are beyond the limits of that which is the specific subject of this report.

9.0 REFERENCES

9.1 Publications

American Society of Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE/SEI 7-16), dated 2016.

California Building Standards Commission, (2019), California Building Code, California Code of Regulations, Title 24.

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FIGURES

APPENDIX A
SEISMIC RESEARCH RESULTS

APPENDIX B
BORING LOGS

APPENDIX C
CPT REPORT

APPENDIX D
PERCOLATION TEST RESULT

APPENDIX E
LABORATORY RESULTS

APPENDIX F
LIQUEFACTION ANALYSIS

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August 7, 2021

Dudek

Attn: Michael Williams

re: Paleontological resources for the Downtown Buena Park Project (PN: 13618)

Dear Michael:

I have conducted a thorough search of our paleontology collection records for the locality and specimen data for proposed development at the Downtown Buena Park project area as outlined on the portion of the Anaheim USGS topographic quadrangle map that you sent to me via e-mail on August 5, 2021. We do not have any fossil localities that lie directly within the proposed project area, but we do have fossil localities nearby from the same sedimentary deposits that occur in the proposed project area, either at the surface or at depth.

The following table shows the closest known localities in the collection of the Natural History Museum of Los Angeles County.

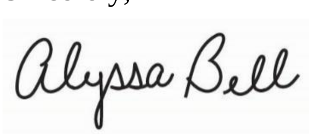
Locality Number	Location	Formation	Taxa	Depth
LACM VP 4185 - 4201, 7971	Coyote Creek, adjacent to Ralph B Clark Regional Park in West Coyote Hills	La Habra Formation (Pleistocene; sandy silt shot through with caliche)	Bison (*Bison*), camel (*Camelops*), horse (*Equus*), mammoth (*Mammuthus*), mastodon (*Mamut*), dire wolf (*Canis dirus*), Coyote (*C. latrans*), deer (*Odocoileus*), dwarf pronghorn (*Capromeryx*), unidentified artiodactyl; sea duck (*Chendytes*)	Surface, in creek bed
LACM VP 3660	Cover St & Pixie Ave; Lakewood	undetermined (Pleistocene)	Mammoth (*Mammuthus*)	19 feet bgs
LACM VP 3319	Intersection of Carson St. & Alameda St, Carson	Unnamed formation (Pleistocene)	Mammoth (*Mammuthus*)	30 feet bgs
LACM VP 3291	Sunset Beach at low tide, 50 yards north of Anderson Street & west of Pacific Coast Highway	Unknown Formation (Pleistocene)	Camel (*Camelops hesternus*)	Surface
LACM VP 7657	Ellis Avenue &	Unknown Formation	School shark (*Galeorhinus*),	150 - 350

- 7659	Patterson Lane, Huntington Beach	(Pleistocene; gray siltstone)	eagle ray *(Myliobatus*), skate (*Raja*), flatfish (*Citharichthys*), goby (*Lepidogobius, Leptocottus*), midshipmen (*Porichthys*), croaker (*Seriphus*), flatfish (*Citharichthys*), cusk-eel (*Otophidium*), skate (*Raja*), angelshark (*Squatina*), sculpin (Cottidae)	ft bgs
LACM VP 7366, 7422 – 7425, 7679; LACM IP 17427	The Huntington Beach Urban Center Sand Borrow Area, N of Pacific Coast Hwy and W of Huntington Dr	Unknown formation (Pleistocene, sands)	Legless lizard (*Anniella*), tree frog (Hyla), gopher snake (*Pituophis*), kingsnake (*Lampropeltis*), ring-necked snake (*Diadophis*), garter snake (*Thamnophis*), long-nosed snake (*Rhinocheilus*), coachwhip (*Masticophis*), salamander (*Enatina*), slender salamander (*Batrachoseps*), skinks (*Plestiodon*), alligator lizard (*Gerrhonotus*), toad (*Bufo*), side-blotched lizard (*Uta*), spiny lizard (*Sceloporus*), climbing salamander (*Aneides*), turtle (*Clemmys*); rail (*Rallus*), quail (*Callipepla*), vole (*Microtus*), shrew (*Sorex*), kangaroo rat (*Dipodomys*), cottontail rabbit (*Sylvilagus*), mole (*Scapanus*), harvest mouse (*Reithrodontomys*), deer mouse (*Peromyscus*), pack rat (*Neotoma*), chipmunk (*Eutamias*), pocket gopher *(*Thomomys*), bat (Chiroptera);* stickleback (*Gasterosteus*); land snails (gastropods)	Unknown

VP, Vertebrate Paleontology; IP, Invertebrate Paleontology; bgs, below ground surface

This records search covers only the records of the Natural History Museum of Los Angeles County ("NHMLA"). It is not intended as a paleontological assessment of the project area for the purposes of CEQA or NEPA. Potentially fossil-bearing units are present in the project area, either at the surface or in the subsurface. As such, NHMLA recommends that a full paleontological assessment of the project area be conducted by a paleontologist meeting Bureau of Land Management or Society of Vertebrate Paleontology standards.

Sincerely,



Alyssa Bell, Ph.D.
Natural History Museum of Los Angeles County

enclosure: invoice

1 Introduction

Dudek was retained by Merlone Geier to prepare a Historical Resources Technical Report for the proposed Buena Park Downtown project (Project). This section provides a description of the Project, including information about the location, setting, and proposed Project activities. This section also presents the regulatory setting for the Project, and a description of the Built Environment Study Area.

1.1 Project Location and Description

Project Location

The Buena Park Downtown Mall is located near the junction of Interstate 5 and California State Route 91 in Buena Park, Orange County, California (Figure 1, Project Location Map). The surrounding area is a combination of residential and commercial uses, with the Knott's Berry Farm theme park directly to the west of the Buena Park Downtown Mall. The approximately 28-acre Project site is bound by La Palma Avenue to the north, Stanton Avenue to the west, the Buena Park Downtown Mall property to the east, and single-family residential development to the south. The Project site is zoned CR (Regional Commercial).

The property boundary of the Buena Park Downtown Mall encompasses eight parcels developed with a large commercial building (mall complex) flanked by several stand-alone buildings and landscaped spaces. Specifically, the Project site comprises one parcel: 8150 La Palma Avenue (APN 070-511-01), which contains the Sears building, parking lot, and Sears Automobile Service Station. The Sears building anchors the western end of the mall complex. Detached from the mall complex is the Sears Automobile Service Station further to the west.

The other seven parcels comprising the Buena Park Downtown Mall are immediately adjacent to the Project site. These include: 8150 La Palma Avenue (APN 070-511-01), 8376 La Palma Avenue (APN 070-511-14), 8460 La Palma Avenue (APN 070-511-07), 8201 On the Mall (APN 070-511-15), 8290 On the Mall (APN 070-511-16), 8374 On the Mall (APN 070-511-08), 8374 On the Mall (APN 070-511-05), and APN 070-511-18 (Table 1).

Table 1. Parcels Located in the Buena Park Downtown Mall Property Boundary

Address	APN	Property Description (Construction Date)	Project Site
8150 La Palma Avenue	070-511-01	Sears Building and Sears Automobile Service Station (1959)	Yes
8376 La Palma Avenue	070-511-14	Northern Detached Buildings (2003, 2004, and 2009)	Adjacent
8460 La Palma Avenue	070-511-07	Northern Detached Building (1980)	Adjacent
8201 On the Mall	070-511-15	Central Mall Addition (1961)	Adjacent
8290 On the Mall	070-511-16	Southern Addition (1979)	Adjacent
8450 On the Mall	070-511-08	Eastern Addition (2003)	Adjacent
8374 On the Mall	070-511-05	Parking lot	Adjacent
No Address	070-511-18	Parking lot	Adjacent

Project Description

Merlone Geier is proposing to redevelop the Sears building that was vacated in February 2020 at the Buena Park Downtown Mall located at 8150 La Palma Avenue in the City of Buena Park. The redevelopment plans include 1,381 residential units, 41,500 square feet of amenity and lobby space, and 2,551 residential parking spaces. The approximately 28-acre Project site is bound by La Palma Avenue to the north, Stanton Avenue to the west, the Buena Park Downtown Mall to the east, and single-family residential neighborhoods to the south. The Project site includes the Sears building, auto center, and surrounding parking lots. The property is designated “Entertainment Mixed-Use” in the City of Buena Park General Plan; it is zoned CR (Regional Commercial). Because residential uses are not permitted in the CR zoning district, a zone change would be required. A rezone to “General Mixed Use,” which would allow a mix of land uses, including residential, is being contemplated.

Built Environment Study Area

The Built Environment Study Area encompasses all areas that may be affected by the proposed Project (Figure 2, Built Environment Study Area Map). This includes those areas where demolition or construction activities related to the Project may result in impacts to historical resources that are not part of the Project site. The Buena Park Downtown Mall property comprises the Project site as well as adjacent parcels that are not proposed for demolition or alteration as part of the Project. The boundaries of the Buena Park Downtown Mall property form the Built Environment Study Area for the purposes of this study. The Built Environment Study Area includes eight (8) parcels: (APNs 070-511-01, 070-511-14, 070-511-07, 070-511-15, 070-511-16, 070-511-08, 070-511-05, and APN 070-511-18). The Buena Park Downtown Mall property encompasses the proposed project footprints, areas of demolition, new construction, building renovation, and areas used for staging, if known. Defining the Built Environment Study Area as the limits of the Buena Park Downtown Mall property boundary also takes into consideration the maximum extent of potential visual and vibration-related impacts that the near-term projects could have on historic built environment resources.

which brought people from the surrounding areas to eat in the newly expanded tearoom. The restaurant's popularity resulted in long wait times, led the Knotts to develop Ghost Town, Knott's Berry Farm's first attraction, in 1940. Knott's Berry Farm, which is located to the direct west of the Buena Park Downtown Mall, continued to grow and develop, eventually becoming a 57-acre theme park. The Knott family owned the property until 1997, when it was sold to Cedar Fair, L.P., which owns and operates multiple amusement parks in the United States (Brigandi 2008; VBP 2021; Dixon 2004).

The tourism generated by Knott's Berry Farm remains the single largest source of revenue for the City. By the early 1950s, agricultural land surrounding the core of Buena Park began to be redeveloped with single-family tracts. Development was focused around I-5 and Beach Boulevard. Between 1950 and 1960, Buena Park's population increased from 5,483 to 46,601. Locals feared that Buena Park would be absorbed into neighboring cities, such as Anaheim or Fullerton.

To meet the need for with residential services and address safety issues caused by the population boom, Buena Park was incorporated as a City on January 27, 1953. That same year, the City's first city council and mayor were elected. In 1954, the Santa Ana Freeway was expanded through Downtown Buena Park, essentially eliminating the City's original commercial corridor. The freeways continued to bring suburban expansion and multiple residential tracts were developed after incorporation. In 1960, the Buena Park Downtown Mall was first developed into the Buena Park Regional Shopping Center, an open-air regional shopping mall intended to serve the growing Buena Park population. Tourism and entertainment continued to be focal points of Buena Park, with the construction of the Movieland Wax Museum in the late 1970s and Medieval Times in the early 1980s, both along Beach Boulevard. By 2000, the City's population had risen to 78,282, with the majority of the open agricultural lots seen in the 1960s infilled with single-family residences, apartment complexes, and commercial properties. As of 2019, the population of Buena Park is relatively dense, with 81,788 people residing in about ten square miles (NETR 2021; Dixon 2004; Oftelie 2020).

History of the Project Site

Following the construction of the initial Sears building in 1959, the Buena Park Downtown Mall was expanded in multiple stages over the course of a 50-year development period. In addition to physical redevelopment projects, the property changed names four times within its history including the following: Buena Park Regional Shopping Center (1954-1960); Buena Park Center (1961-1975); Buena Park Mall (1976-2003); and Buena Park Downtown (2004-2021). For the purposes of this report, the property will be called the Buena Park Downtown Mall, which includes the eight parcels are located both within and immediately adjacent to the Project site.

Buena Park Regional Shopping Center (1954-1960)

Between the 1930s and 1950s, the Buena Park Downtown Mall (property) was primarily farmland with several small residences spread out on multiple lots. In 1954, the newly