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REPORT
GEOTECHNICAL INVESTIGATION
IRISH FAMINE MEMORIAL
LINCOLN PARK, LINCOLN HIGHWAY
SAN FRANCISCO, CALIFORNIA

Job No.: 1974.19

Prepared for:

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By

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Geotechnical Engineer – 390

June 14, 2019

INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Great Irish Famine Memorial in Lincoln Park on Lincoln Highway, San Francisco, California. We understand the project is to be constructed as shown on the plans Sheets A0.0-A2.2 by Andy Forrest, P.E. of ARCHIdigm dated January 30, 2019.

The purpose of the geotechnical investigation is to evaluate the soil conditions at the site and provide geotechnical conclusions and recommendations to aid the design and construction of the project.

The scope of our work included exploring and evaluating the subsurface conditions with test borings and laboratory tests, analyzing the results of the field and laboratory work, and presenting our findings in this report. Our report provides the following geotechnical information:

1. A description of the soil and geologic conditions observed.
2. An opinion of project feasibility from a geotechnical standpoint.
3. Design recommendations for new foundations and retaining walls.
4. Site grading and soil engineering drainage recommendations.

The scope of our work does not include an evaluation of soil or groundwater hazardous waste contamination, toxicity, or corrosion potential at the site. The scope of our work also does not include an evaluation of any existing foundations or the foundation support of any existing structures.

WORK PERFORMED

We performed a reconnaissance of the site and reviewed the following geological maps:

> Schlocker J., Bonilla M.G., and Radbruch D.H., 1958; Geology of the San Francisco North Quadrangle, U.S. Geological Survey Miscellaneous Geologic Investigation Map I-272, Scale 1:24,000.

On June 5, 2019 we explored the subsurface conditions at the site to the extent of 2 test borings in the proposed construction area. The test borings were drilled with portable power augers to depths of 15.5' and 14', terminating in dense friable sandstone bedrock. The approximate locations of the borings are indicated on the attached Boring Location Sketch, Plate 1. We observed the drilling, logged the conditions encountered, and obtained samples for visual examination, classification and laboratory testing.

Detailed descriptions of the materials encountered in our borings are presented on the logs of boring. The attached boring logs and related information depict subsurface conditions only at the approximate location shown on Plate 1 and on the date designated on the log; subsurface conditions at other locations and times could differ from the conditions occurring at our boring location. Details of the field and laboratory work are presented in the appendix at the end of the report.

SITE AND SOIL CONDITIONS

The site is located on the north and downslope side of Lincoln Highway above the 17th Tee of the Lincoln Park Golf Course in San Francisco. The site is relatively level at the shoulder of the highway and then slopes down to the north at approximate gradients on the order of 2:1 (horizontal: vertical) as shown on the topographic survey by Westover Surveying, dated 03/07/2019. The slope where the structure is proposed to extend is currently densely vegetated.

Geology and Soils

The area has been mapped as containing dune sands in close proximity to sandstone bedrock (Schlocker et al, 1958).

Our test borings encountered very loose to loose sand with clay and some rock fragments (partially fill) to about 8 to 10 feet depth, where medium dense clayey sand was encountered to about 12 feet. Dense to very dense cemented clayey sand (friable sandstone bedrock) was encountered below about 12 feet depth.

Detailed descriptions of the materials encountered are presented in logs of Borings, plates 2 and 3.

Groundwater

Free groundwater was not encountered in our borings at the time of drilling. However, fluctuations in the groundwater level could occur due to variation in rainfall and/or other factors and groundwater may be encountered during construction.

CONCLUSIONS AND DISCUSSION

Based on our field, laboratory and office studies we judge that the project is feasible from a geotechnical engineering standpoint provided that the recommendations presented in this report are incorporated in the design and construction.

We recommend that the structure be supported on drilled, cast-in-place reinforced concrete pier foundations. The piers should be connected with reinforced concrete grade beams and/or structural slabs spanning between the pier foundations.

Temporary excavations during construction should be properly sloped backed or shored in accordance with OSHA requirements. Construction safety and stability of temporary excavations during construction is solely the responsibility of the contractor.

Surface and subsurface drainage facilities should be constructed as discussed below in the

"Recommendations" section of the report.

Like the entire San Francisco Bay Area, the site is subject to strong ground shaking during earthquakes. It will be necessary to design and construct the project in strict conformance with current standards for earthquake resistant construction. The U.S. Geological Survey (USGS, 2015) predicts a 72% chance of a large earthquake (Richter Magnitude 6.7 or greater) occurring in the Bay Area in the next 30 years.

All conclusions and recommendations presented in this report are contingent upon Nersi Hemati being retained to: 1) Review the geotechnical engineering aspects of the final grading and foundation plans prior to construction; and 2) Observe construction of the project as outlined below in the "Supplemental Services" section of this report.

RECOMMENDATIONS

Faulting and Seismicity

The site is not within a current Alquist-Priolo Special Studies Zone, and the geologic maps reviewed indicate that active faults are not considered to exist within the site. The nearest known active faults are the San Andreas Fault, located about 3 kilometers to the southwest, and the Hayward fault, about 21 kilometers to the northeast. Maximum credible earthquake magnitudes of 7.9 and 7.1 (Richter scale) have been postulated for these faults, respectively.

Site Grading

Areas to be developed should be cleared of vegetation, debris, the existing structures, slabs, and foundations. The site should be stripped of the upper few inches of soil containing organic matter. The strippings should be removed, or if suitable, stockpiled for re-use as topsoil in landscaping.

Following initial site preparation, excavation should be performed as necessary. We anticipate that, with the exception of organic matter and rocks larger than 6 inches in diameter, the excavated material will be suitable for re-use as compacted fill.

In sloping ground areas steeper than about 5:1 a keyway should be excavated at the toe of the fill extending at least 2 feet into bedrock or dense strong soil. A subdrain consisting of minimum 4-inch diameter perforated pipe (SDR 35 or better) encased in Class 2 permeable materials (Caltrans Specification) should be installed in the keyway. The exposed subgrade to receive fill should be prepared by scarifying to a depth of 6 inches, moisture-conditioning as necessary, and compacting to at least 90% of the maximum dry density of the materials as determined by the ASTM D-1557 laboratory compaction test procedure. Fill material should then be spread in 8-inch thick loose lifts, moisture-conditioned as necessary, and compacted to at least 90% relative compaction. As successive layers of fill are placed they should be continually keyed into rock or strong soil. The final lift below slabs and pavements should be compacted to at least 95% Relative Compaction.

Imported fill should be non-expansive; that is, it should have a plasticity index of 15 or less. The imported fill material should be free of organic matter and of rocks or lumps larger than 6 inches in diameter. Not more than 15% of the rocks or lumps should exceed 2.5 inches.

Generally, grading is most economically performed during the summer months when on-site soils are usually dry of optimum moisture content. Delays should be anticipated in site grading performed during the rainy season due to excessive moisture in on-site soils. Special and relatively expensive construction procedures should be anticipated if grading must be completed during the winter.

Cut and fill slopes should be no steeper than 2:1. Where steeper banks are required, retaining walls should be used. Slopes should be planted with fast growing, deep-rooted groundcover to reduce sloughing or erosion.

Drilled Piers

Drilled piers should be at least 18 inches in diameter and should extend at least 12 feet into dense cemented sand or sandstone rock as determined by the Geotechnical Engineer during drilling. The final depth of the piers is estimated to be on the order of 24 feet or more, but the depths should be based on the design by the project structural engineer. The piers should be designed and reinforced to resist lateral creep forces equivalent to an average 12-foot thick zone exerting an equivalent fluid pressure of 60 pounds per cubic foot (pcf) acting on 2 pier diameters. The thickness of the creep zone may be reduced where excavation removes existing soil, such as for cut slope retaining walls, and should equal depth to bedrock or dense soil. The piers should be designed by the project structural engineer. However, we recommend that all piers be reinforced with at least four No. 5 bars. The piers should be connected with grade beams and tie beams as well as structural slabs as needed. The grade beams and slabs should span between the piers in accordance with structural requirements. The steel from the piers should extend sufficient distance into the grade beams to develop its full bond strength.

The portion of the piers extending into dense soil or rock may be designed using an allowable skin friction of 800 pounds per square foot (psf). End bearing should be neglected because of the difficulty of cleaning out small diameter pier holes, and the uncertainty of mobilizing end bearing and skin friction simultaneously. Lateral loads on piers will be resisted by passive pressure on the rock or dense soil. An equivalent fluid pressure of 400 pcf acting on 2 pier diameters should be used and the stability of the system should be calculated using a minimum factor of safety of 1.5.

If ground water is encountered, it may be necessary to dewater the holes and/or place the concrete by the tremie method. If caving soils are encountered, it may be necessary to case the holes. Hard drilling may be required to achieve the required penetration.

Retaining Walls

New retaining walls should be designed to resist lateral earth pressures plus additional lateral pressures that may be caused by surcharge loads at the ground surface behind the walls such as for surcharge from nearby foundations and walls. Retaining walls supporting a relatively level backfill should be designed to resist an active equivalent fluid pressure of 40-pcf acting in a triangular pressure distribution. Where the backfill slopes up at a 2:1 gradient, the walls should be designed for an equivalent fluid pressure of 60 pcf. Values can be interpolated for flatter gradients. Retaining walls restrained from movement at the top should be designed for pressures of 60 and 80 pcf for level and sloping backfills respectively. Non-restrained retaining walls may deflect about 1% of the height of the wall at the top of the wall.

We recommend a uniform pressure (in psf) equal to 12 times the height of the retaining walls (measured in feet) be used as seismic surcharge.

Where an imaginary 1-1/2:1 line projected down from foundations intersects retaining walls, the portions of the retaining walls below the intersection should be designed for an additional horizontal surcharge load. Where an 1-1/2:1 line projected down from the toe of a retaining wall intersects a lower retaining wall, the portion of the lower wall below the intersection should be designed for an additional surcharge load. The upper wall surcharge load should be assumed to be a uniform lateral pressure equal to the height of the upper retaining wall times the equivalent fluid pressure acting on that wall. Where retaining wall backfill is subject to vehicular traffic, the walls should be designed to resist an additional surcharge pressure equivalent to 2 feet of additional backfill.

Retaining walls should be fully backdrained. The backdrains should consist of 4-inch diameter, rigid perforated pipe embedded in drain rock. The pipe should be PVC Schedule 40, SDR 35, or equivalent, and the pipe should be sloped to drain to outlets by gravity. Drain rock should consist of clean, free-draining crushed rock or gravel. The rock should be wrapped in filter fabric such as Mirafi 140N or equivalent. Alternatively, Class 2 permeable rock (Caltrans Specification) should be used in lieu of drain rock and filter fabric. The top of the pipe should be at least 8 inches below the lowest adjacent grade. The crushed rock or gravel should extend to within 1 foot of the surface. The upper one-foot should be backfilled with compacted soil to exclude surface water. The ground surface behind retaining walls should be sloped to drain.

Where migration of moisture through retaining walls would be detrimental, retaining walls should be waterproofed. Retaining walls will yield slightly during backfilling. Therefore, walls should be backfilled prior to building on or adjacent to the walls.

Slab-On-Grade

Slabs-on-grade should consist of structural slabs spanning between adjacent foundations designed by the project structural engineer. Slab-on-grade subgrade should be rolled to produce a dense, uniform surface. Loose fill and areas of soft soil should be over-excavated and re-compacted as engineered fill.

The slabs should be underlain with a capillary moisture break consisting of at least 4 inches of clean, free draining crushed rock or gravel at least 1/4 inch and no larger than 3/4 inch in size. Where migration of moisture vapor through slabs would be detrimental, an impermeable membrane moisture vapor barrier, 15 mils or thicker, should be provided between the drain rock and the slabs. The membrane should be covered with 2 inches of sand to protect it during construction. However, we defer to flooring and waterproofing specialists who should be consulted for these items especially the use of sand over the membrane. Outlets should be provided from the slab drain rock.

The future expansion potential of the sub-grade soils should be reduced by thoroughly presoaking the slab sub-grade prior to concrete placement. Slabs should be at least 6 inches thick, and should be reinforced with at least #4 bars on 12-inch centers each way. These are minimum requirements as the slabs should be designed by the project structural engineer. Slabs should be grooved at regular intervals to induce and control cracking.

Soil Engineering Drainage

Surface water should be diverted away from slopes and foundations. Roofs should be provided with gutters, and the downspouts should be connected to closed conduits discharging well away from foundations and slopes or into the street storm drain / sewer system in accordance with the requirements of the City and County of San Francisco. Drainage outlets on the slopes should be avoided. Roof downspouts and surface drains must be maintained entirely separate from foundation drains and retaining wall back drains.

Flatwork / Exterior Slabs

We recommend that the subgrade be scarified at least 6 inches and compacted to 90% Relative Compaction. At least 6 inches of Class 2 aggregate base rock should be placed and compacted to 90% Relative Compaction below the concrete surface. The slabs should be reinforced with steel bars designed by the project structural engineer and grooved at regular intervals to induce and control cracking.

LIMITATIONS

Our services consist of professional opinions, conclusions and recommendations that are made in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

Geotechnical engineering is characterized by uncertainty. Therefore, we are unable to eliminate all risks or provide guarantees.

We judge that construction in accordance with these recommendations will be stable, and that the risk of future instability is within the range generally associated with construction in San Francisco. Subsurface conditions are complex, and may differ from those indicated by surface features and those encountered at the test hole locations.

Due to limitations inherent in geotechnical investigations, it is not uncommon to encounter variations in the soil conditions during construction nor is it practical to determine all such variations during an acceptable program of subsurface exploration for a project of this scope.

Soil conditions and standard of practice change. Therefore, we should be consulted to update this report if construction is not performed within 18 months.

SUPPLEMENTAL SERVICES

We should review the final plans for conformance with the intent of our recommendations. During construction, we should observe the conditions encountered in construction excavations and modify our recommendations, if warranted. We should observe and test fill placement and compaction.

We should observe footing excavations or pier drilling operations to determine the actual depths required. Our services during foundation construction are limited to observation of soil and bedrock conditions, depth of excavation or drilling, and the condition of excavations or pier holes prior to concrete placement. Our services do not include observation or approval of steel, concrete, or asphalt nor do they include establishing or verifying construction lines and grades. This should be performed by the appropriate party. Upon completion of the project, we should perform a final observation. We should summarize the results of this work in a final report.

These supplemental services are performed on an as-requested basis, and we cannot accept responsibility for items that we are not notified to observe. These supplemental services are in addition to this soil investigation, and are charged for on an hourly basis in accordance with our Schedule of Charges.

MAINTENANCE

Periodic land maintenance will be required. Surface and subsurface drainage facilities should be checked frequently, and cleaned and maintained as necessary. A dense growth of deep-rooted ground cover must be maintained on all slopes to reduce sloughing and erosion. Sloughing and erosion that occurs must be repaired promptly before it can enlarge into sliding.

APPENDIX - FIELD EXPLORATION AND LABORATORY TESTING

Field Exploration

Our field investigation consisted of a site reconnaissance and subsurface exploration. Due to site inaccessibility, we drilled 4-inch diameter exploratory borings with portable power auger equipment at the approximate locations shown on the Boring Location Sketch, Plate 1.

The materials encountered in the test borings were continuously logged in the field. Logs of our borings are included as Plates 2-3. The soils encountered in our exploratory borings are classified in accordance with the Unified Soils Classification System presented on Plate 4.

Relatively undisturbed soil samples were obtained from the exploratory borings at selected depths appropriate to the subsurface investigation. The samples were obtained with the 2.4" inside diameter Modified California Sampler as well as the Standard Penetration Test (SPT) sampler.

The blow counts were obtained by dropping a 70- pound hammer through a 30-inch free fall. The sampler was driven 18 inches, or a shorter distance where hard resistance was encountered, and the number of blows were recorded for each 6 inches of penetration. The blow per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the sampler the last 12 inches or the number of inches indicated where the sampler did not penetrate the full 18 inches.

The blows per foot recorded on the boring log have been adjusted to represent the standard penetration test. The approximate location of the exploratory borings was established in the field by pacing and tape methods. Boring locations were not established by surveying methods and the approximate locations indicated on the Boring Location Sketch should be assumed accurate only to the degree implied by the method used.

The boring logs show our interpretation of the subsurface conditions on the dates and at the locations indicated and it is not warranted that they are representative of the subsurface conditions at other locations and times. The stratification lines on the borings represent the approximate boundaries between the material types; actual transitions may be gradual.

Laboratory Testing

Water Content and Dry Density

The natural water content and dry density were determined on several samples of the materials recovered from the borings respectively; these are recorded on the boring logs at the appropriate sample depths.

Minus #200

The percentage of particles passing the No. 200 sieve was determined on a sample of the subsurface materials to assist in the classification of the soils. The results of the tests are shown on the logs of borings and denoted (-200).

PROJECT	Irish Famine Memorial, Licoln Hwy, San Francisco			BORING NO: 1
DATE OF BORING	6/5/19			
TYPE OF BORING	4" Augers			
HAMMER WEIGHT	#70			

DESCRIPTION OF MATERIALS	SAMPLES				
	DEPTH IN FEET	*BLOWS PER FOOT	DRY DENSITY (PCF)	WATER CONTENT (%)	OTHER TESTS
BROWN TO DARK BROWN SAND WITH SOME CLAY (SP)		4			
very loose to loose, with rock fragments and roots					
(FILL)			103	7.5	
grades with more rock fragments					
	5-	9			
			99	2.8	
	10-				
MOTTLED ORANGE BROWN SAND TO CLAYEY		17			
SAND (SP/SC)				6.1	-200=5%
medium dense					
grades clayey, cemneted and denser					
YELLOW BROWN CEMENTED CLAYEY SAND		30			
AND SANDSTONE BEDROCK	15-				
dense, friable rock					
Bottom of Boring 15.5'					
* Blow counts have been converted to SPT					
No Groundwater Encountered					

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PROJECT	Irish Famine Memorial, Lincoln Hwy, San Francisco	BORING NO: 2
DATE OF BORING	6/5/19	
TYPE OF BORING	4" Augers	
HAMMER WEIGHT	#70	

DESCRIPTION OF MATERIALS	DEPTH IN FEET	*BLOWS PER FOOT	DRY DENSITY (PCF)	WATER CONTENT (%)	OTHER TESTS
LIGHT BROWN SAND WITH GRAVEL (SP)					
BROWN SAND & RED BROWN CLAYEY SAND WITH SILTY CLAY (SP/SC)		8			
loose, with roots and rock fragments				10.1	
(FILL)					
	5				
DARK GRAYISH TO BLACKISH BROWN SAND WITH ROCK FRAGMENTS (SP)		12			
medium dense (FILL)			104	4.6	
grades very clayey					
	10				
YELLOW BROWN CEMENTED CLAYEY SAND AND SANDSTONE BEDROCK		50			
dense to very dense, friable rock				11.6	
	15				
Bottom of Boring 14'					
* Blow counts have been converted to SPT					
No Groundwater Encountered					

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PLATE 3

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS (MORE THAN HALF IS LARGER THAN #200 SIEVE)	GRAVELS (MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE)	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL - SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL - SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL - SAND - CLAY MIXTURES
	SANDS (MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE)	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND - SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND - CLAY MIXTURES
FINE GRAINED SOILS (MORE THAN HALF IS SMALLER THAN #200 SIEVE)	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

Shear Strength, psf
Confining Pressure, psf

Consol	- Consolidation	Tx	320	(2600)	Unconsolidated Undrained Triaxial
LL	- Liquid Limit (in %)	Tx CU	320	(2600)	Consolidated Undrained Triaxial
PL	- Plastic Limit (in %)	DS	2750	(2000)	Consolidated Drained Direct Shear
PI	- Plasticity Index	FVS	470		Field Vane Shear
G_s	- Specific Gravity	UC	2000		Unconfined Compression
SA	- Sieve Analysis	LVS	700		Laboratory Vane Shear
	"Undisturbed" Sample	SS	- Shrink Swell		
	Bulk or Disturbed Sample	EXP	- Expansion		
	Standard Penetration Test	P	- Permeability		
	Sample Attempt with No Recovery				

Note: All strength tests on 2.8" or 2.4" diameter sample unless otherwise indicated.

KEY TO TEST DATA

SOIL CLASSIFICATION AND KEY TO TEST DATA Irish Famine Memorial, Lincoln Highway San Francisco, CA		
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