CENG238 Geotech Project

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May 2025

Abstract

This geotechnical investigation supports the design and construction of two proposed structures near the San Francisco Bay Area: a four-story office building and a single-story vehicle maintenance facility. Field exploration included Standard Penetration Tests (SPT) and Cone Penetration Tests (CPT) to characterize subsurface conditions and evaluate geologic risks such as liquefaction, settlement, and bearing capacity limitations. The site is underlain by fat clay, bay mud, and deeper alluvial and sand layers, with groundwater encountered at approximately 11 feet. For the vehicle maintenance facility, a shallow foundation system was selected, with bearing capacity and settlement analyses confirming the suitability of a 2-foot-wide strip footing. For the office building, deep foundations were evaluated using both the alpha and beta methods to estimate skin friction resistance of 72-foot-long precast concrete piles. The analysis recommends two piles per gravity column. Results from laboratory testing, CPT logs, and analytical models informed final design recommendations to ensure structural stability and serviceability in challenging Bay Area soil conditions.

1 Project Description

The proposed development consists of two primary structures located at approximately 37.478067° latitude and -122.133534° longitude, near the San Francisco Bay Area. The first structure is a four-story office building with plan dimensions of approximately 250 feet by 150 feet. This building is expected to support gravity column loads ranging from 300 to 400 kips. The second structure is a single-story vehicle maintenance facility, measuring 30 feet by 30 feet, with anticipated strip loads of 2 to 3 kips per foot.

Given the scale and structural demands of the project, a comprehensive geotechnical investigation is being conducted to evaluate subsurface conditions and identify potential geologic hazards. The investigation will assess factors such as soil composition, bearing capacity, groundwater levels, and seismic risks, including liquefaction potential. These findings will inform the design of foundations and other structural elements to ensure safety, performance, and long-term stability of the proposed buildings.

2 Geology

The geotechnical characteristics of the site suggest several important considerations for development, particularly in relation to soil stability, groundwater conditions, and historical land use. A thorough geotechnical investigation is recommended to assess potential risks and inform appropriate engineering solutions.

2.1 Regional Geology

The area is susceptible to liquefaction, primarily due to the presence of loose, saturated granular soils combined with seismic activity common to the region. This hazard can lead to ground failure during earthquakes, making it a critical factor in foundation design and site development. Regional soil and seismic conditions must be integrated into the overall risk assessment and engineering strategy.[4]

2.2 Local Geology

The site appears to be generally flat, which may indicate the presence of previous structures or grading activities. Despite this, a detailed geotechnical analysis is necessary to confirm subsurface conditions. The presence of bay mud, poses concerns due to its low bearing capacity and potential for significant settlement. Additionally, the nearest two wells indicate groundwater levels ranging from 220 to 224 feet and 164 to 184 feet, respectively. Given the site's proximity to the bay, groundwater conditions should be carefully evaluated during planning and construction.[3]

2.3 Geologic Risk Factors

The site presents several geologic risk factors that warrant careful consideration during planning and development. The potential for liquefaction, driven by the region's seismic activity and the presence of loose, saturated soils, poses a significant threat to structural stability during earthquakes. Additionally, the presence of bay mud introduces challenges related to settlement and foundation support, as this type of soil is known for its low shear strength and high compressibility. Groundwater levels, while varying between nearby wells, remain a critical factor due to the site's proximity to the bay, increasing the likelihood of soil saturation and further exacerbating liquefaction risks. These combined conditions underscore the importance of a comprehensive geotechnical investigation to evaluate subsurface variability, assess seismic response, and design appropriate mitigation strategies to ensure long-term structural integrity and safety.[4]

3 Site Exploration

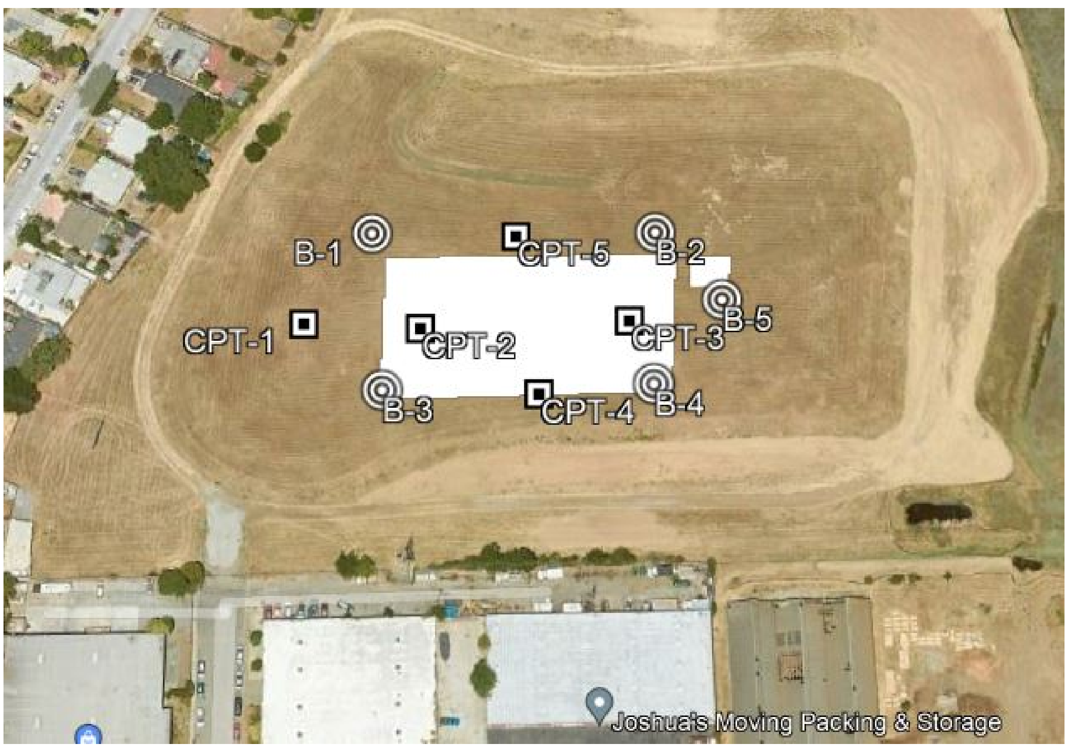

Figure 1: Boring Locations on Build Site

Site exploration was conducted using a combination of Standard Penetration Tests (SPT) and Cone Penetration Tests (CPT) to evaluate subsurface conditions across the project site. A total of five SPT borings and five CPT borings were performed, evenly distributed throughout the site, as shown in Figure 1.

SPT Borings 1 through 4 were advanced to a depth of 60 feet, while Boring 5 extended to a depth of 100 feet. For the CPTs, Borings 2 and 3 reached depths of 70 feet, and Borings 1, 4, and 5 were extended to 100 feet. These borings provided critical data on soil stratigraphy, relative density, and strength parameters necessary for foundation design and geotechnical risk assessment.

3.1 Boring Log Analysis and Site Characterization

Analysis of the SPT boring logs indicates that the site is generally covered by a thin layer of fill, typically less than 1 foot thick. Beneath this, a layer of fat clay extends to a depth of approximately 5 feet. As anticipated during the initial site characterization, this is followed by a layer of bay mud, which generally extends to a depth of around 10 feet, though it is deeper in some areas. Below the bay mud lies a layer of alluvium, typically reaching depths of 15 to 20 feet. Underlying these deposits are various layers of clay, which continue beyond the explored depths.

The CPT data further reveals that groundwater is encountered at a depth of approximately 11 feet below the surface. This shallow groundwater table is consistent with the site's proximity to the San Francisco Bay and has important implications for liquefaction potential and foundation design.

Using data from both the SPT and CPT investigations, four cross-sectional profiles were developed to illustrate the variability of subsurface conditions across the site. These profiles provide a clearer understanding of the stratigraphy and are essential for evaluating foundation performance and geotechnical risks.

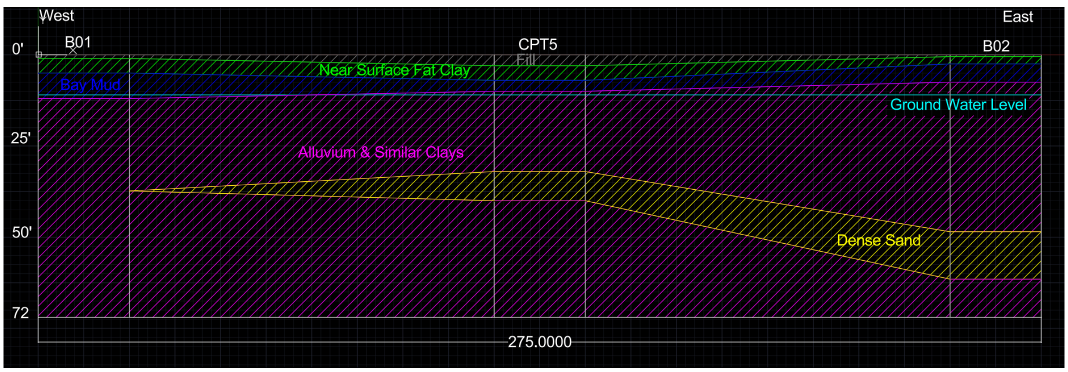

Figure 2: Cross Section 1 (West to East)

Figure 2 presents a cross section running west to east along the north end of the site. A thin 1–2 ft layer of fill is observed at the surface, underlain by a relatively uniform 3–4 ft layer of fat clay. Beneath the fat clay lies a 3–5 ft deposit of bay mud extending across the section. Below the bay mud, alluvial deposits consisting of clays or clayey silts are present. On the eastern side of the section, a dense sand layer approximately 15 ft thick appears at a depth of around 40 ft.

Figure 3 shows a cross section running south to north along the west side of the site. This section reveals thicker layers of both fat clay and bay mud, each approximately 5 ft in thickness. A dense sand deposit is visible at depth on the southern end of the cross section.

Figure 4 illustrates a cross section running south to north along the east side of the site. An even layer of bay mud is seen cutting through the underlying fat clay toward the center of the profile. A substantial deposit of dense sand appears below 60 ft across much of the section, with a smaller isolated deposit present near the southern end.

Figure 5 depicts a cross section running west to east along the south side of the site. Here, the bay mud layer appears thinner, ranging from 1 to 3 ft in thickness across the profile. Around 40 ft below the surface, a thick continuous layer of dense sand is present, with an additional deeper sand layer observed on the eastern end of the section.

3.2 Soil Characterization

Table 1 summarizes the soil properties derived from unconsolidated undrained (UU) lab test results. The final column, labeled S_u, represents undrained shear strength, which is equivalent to cohesion for clayey soils and bay mud. Unit weights were calculated using the equation:

$$\gamma = \mathrm{dd} \cdot (1 + \mathrm{WC})$$

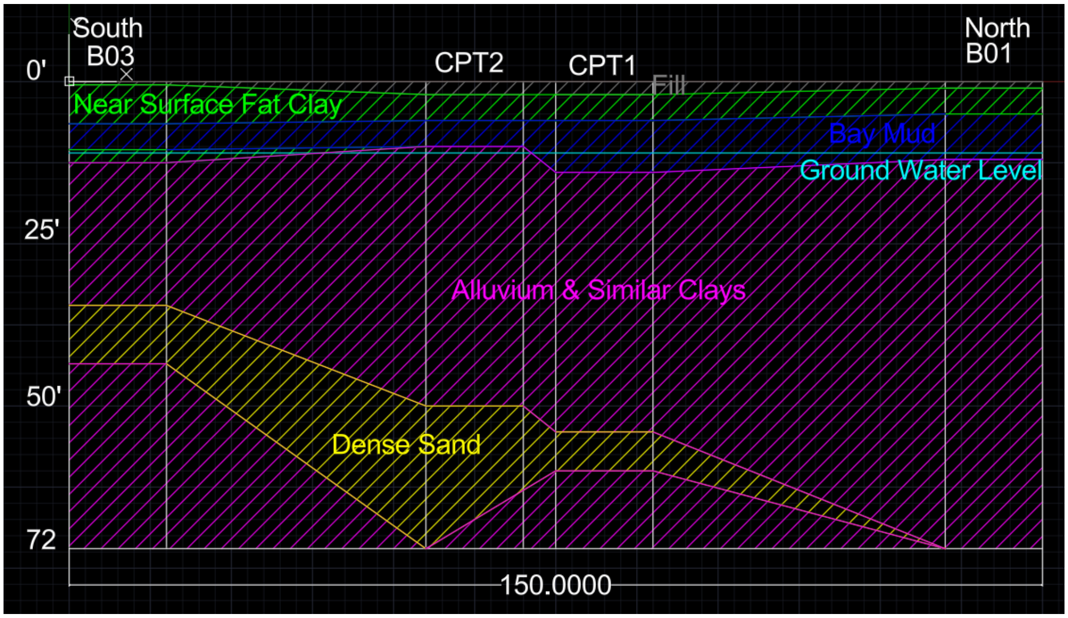

Figure 3: Cross Section 2 (South to North)

where dd is the dry density and WC is the water content (expressed as a decimal).

Table 1: Summary of Soil Properties by Site and Layer

Site	**Layer**	**Depth (ft)**	**Dry Density (pcf)**	**Water Content (%)**	**Unit Weight (pcf)**	S_u **(psf)**
B2	Bay Mud	5.0	43.2	106.6	89.3	217.0
B1	Bay Mud	7.0	68.3	54.2	105.3	292.8
B4	Bay Mud	7.5	46.8	93.0	90.3	253.2
B5	Bay Mud	8.0	51.5	80.7	93.1	304.7
B3	Bay Mud	10.5	62.4	57.8	98.5	443.0
B5	Bay Mud	12.5	57.0	71.0	97.5	267.2
B4	Deep Clay	15.0	104.2	22.9	128.1	1386.4
B1	Deep Clay	15.5	106.6	21.5	129.5	3415.0
B2	Deep Clay	16.0	112.2	18.0	132.4	1220.0
B1	Deep Clay	25.0	119.3	14.6	136.7	5004.6
B2	Deep Clay	36.0	100.3	25.2	125.6	715.5
B5	Deep Clay	39.2	94.0	30.5	122.7	904.0
B5	Deep Clay	84.2	110.1	20.5	132.7	1052.7

All of the samples in Table 1 were taken from either bay mud or deep clay layers. Unfortunately, no direct lab test data was available for the near-surface fat clays or deeper dense sands. To characterize these layers, Cone Penetration Test (CPT) logs were used.

The CPT data indicates that dense sand layers typically exhibit N_{160} values around 30, while the surface-level fat clays show values closer to 12. Figure 6, sourced from [1], shows expected unit weight ranges for various soils. For dense sands, this range is approximately 120–145 pcf.

To refine these estimates, Figure 7 correlates unit weight with effective friction angle. CPT logs (see Figure 8) show that poorly graded sands exhibit effective friction angles around 40°, consistent across different gradations. This corresponds to unit weights of 123, 128, and 137 pcf for poorly graded sand, well graded sands, and poorly graded gravel, respectively. An average of these values—approximately 129.3 pcf—is used for dense sand.

For the surface-level fat clay, which is of high plasticity (as noted in boring logs), the unit weight is expected to fall between 80 and 110 pcf. Since this layer is above the water table and exhibits relatively high N_{160} values compared to bay mud, we conservatively estimate the unit weight at 110 pcf.

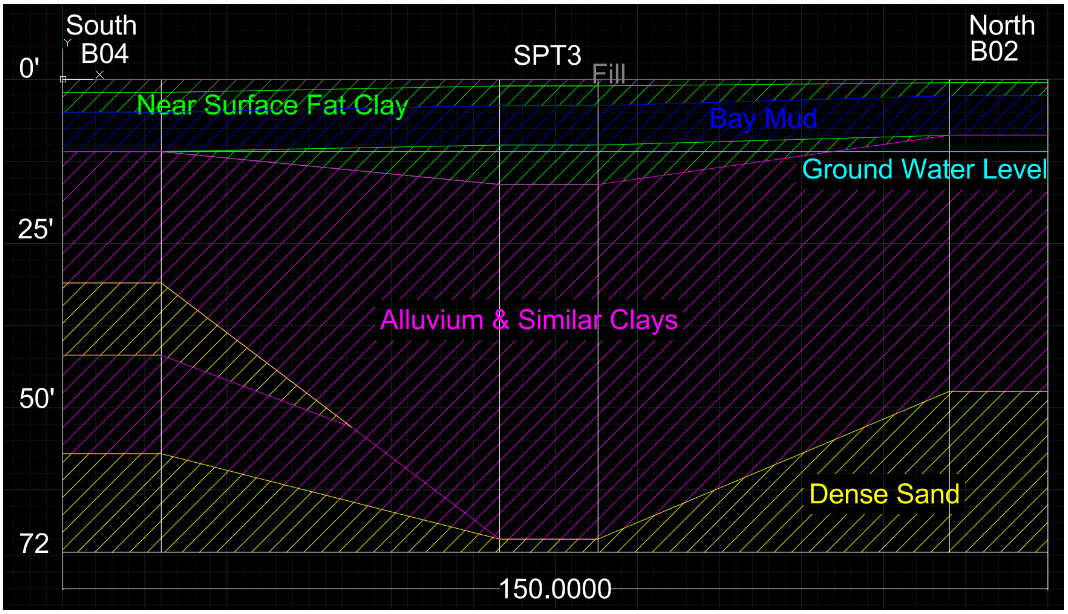

Figure 4: Cross Section 3 (South to North)

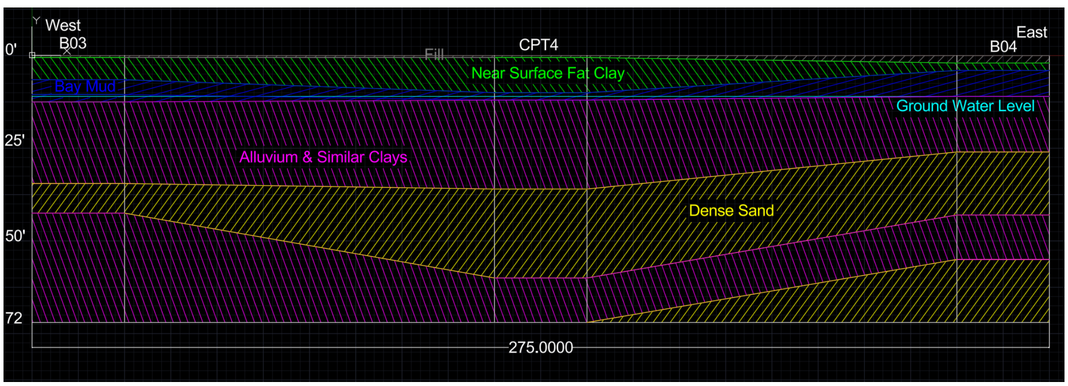

Figure 5: Cross Section 4 (West to East)

Shear strength (S_u) values for clays were derived from CPT logs, which suggest strengths around 2 tsf (4.4 kpsf) for surface clays. For dense sands, CPT-based shear strength estimates were unreliable due to their exceeding the test's measurable limit (greater than 6 tsf). However, since dense sand will be analyzed using the β-method for skin friction, exact shear strength values are not needed.

The final summary of average soil properties by layer is presented in Table 2. Values for bay mud and deep alluvial clays are based on averages from Table 1, while values for fat clay and dense sand are estimated as described above.

Table 2: Average Unit Weights and Shear Strengths by Material Type

Material	**Unit Weight (pcf)**	S_u **/ Cohesion (psf)**
Bay Mud	95.6	260.5
Fat Clay	110.0	4400.0
Alluvium and Deep Clay	129.7	1956.9
Dense Sand	129.3	–

Soil Type and Unified Soil Classification	Typical Unit Weight, γ — Above Groundwater Table (kN/m³)	(lb/ft³)	Below Groundwater Table (kN/m³)	(lb/ft³)
GP — Poorly-graded gravel	17.5–20.5	110–130	19.5–22.0	125–140
GW — Well-graded gravel	17.5–22.0	110–140	19.5–23.5	125–150
GM — Silty gravel	16.0–20.5	100–130	19.5–22.0	125–140
GC — Clayey gravel	16.0–20.5	100–130	19.5–22.0	125–140
SP — Poorly-graded sand	15.0–19.5	95–125	19.0–21.0	120–135
SW — Well-graded sand	15.0–21.0	95–135	19.0–23.0	120–145
SM — Silty sand	12.5–21.0	80–135	17.5–22.0	110–140
SC — Clayey sand	13.5–20.5	85–130	17.5–21.0	110–135
ML — Low plasticity silt	11.5–17.5	75–110	12.5–20.5	80–130
MH — High plasticity silt	11.5–17.5	75–110	11.5–20.5	75–130
CL — Low plasticity clay	12.5–17.5	80–110	11.5–20.5	75–130
CH — High plasticity clay	12.5–17.5	80–110	11.0–19.5	70–125

Figure 6: Unit weight ranges by soil type, from [1].

4 Shallow Foundation

The proposed structure is a single-story vehicle maintenance facility with plan dimensions of 30 ft by 30 ft. The building will be supported by continuous (strip) footings subjected to line loads of approximately 2–3 kips/ft. To be conservative, a design load of 3 kips/ft is used in this analysis.

Based on boring log B05, the soil profile at the facility location consists of approximately 5 feet of fat clay underlain by bay mud. To avoid interaction with the weaker bay mud layer, the footings are designed to remain as shallow as practicable. This also ensures that the footing stays above the groundwater table, which begins at approximately 11 feet below the ground surface. Staying above the water table eliminates the need to account for pore water pressure in the design.

The undrained shear strength (S_u) of the fat clay is estimated at 4.4 kips/ft^2, based on CPT data. To determine the required footing width, the bearing pressure equation from [1] is used:

$$q = \frac{P + W_f}{A} - u_D$$

Where: - P is the applied load (3 kips/ft), - W_f is the self-weight of the footing per foot length, - A is the area of the footing (per foot of length, i.e., width B), - u_D is the pore water pressure at depth (zero in this case since the footing is above the water table).

Assuming a footing embedment depth of 1 foot and a concrete unit weight of 150 pcf, the self-weight per foot of length becomes:

$$W_f = 150 \cdot B$$

A factor of safety (F) of 2.5 is applied due to the availability of comprehensive site data, including both SPT and CPT results. Rearranging the equation to solve for footing width B:

$$\frac{S_u}{F} = \frac{P + 150B}{B}$$

Solving algebraically:

$$\left(\frac{S_u}{F} - 150\right) B = P \quad \Rightarrow \quad B = \frac{P}{\frac{S_u}{F} - 150}$$

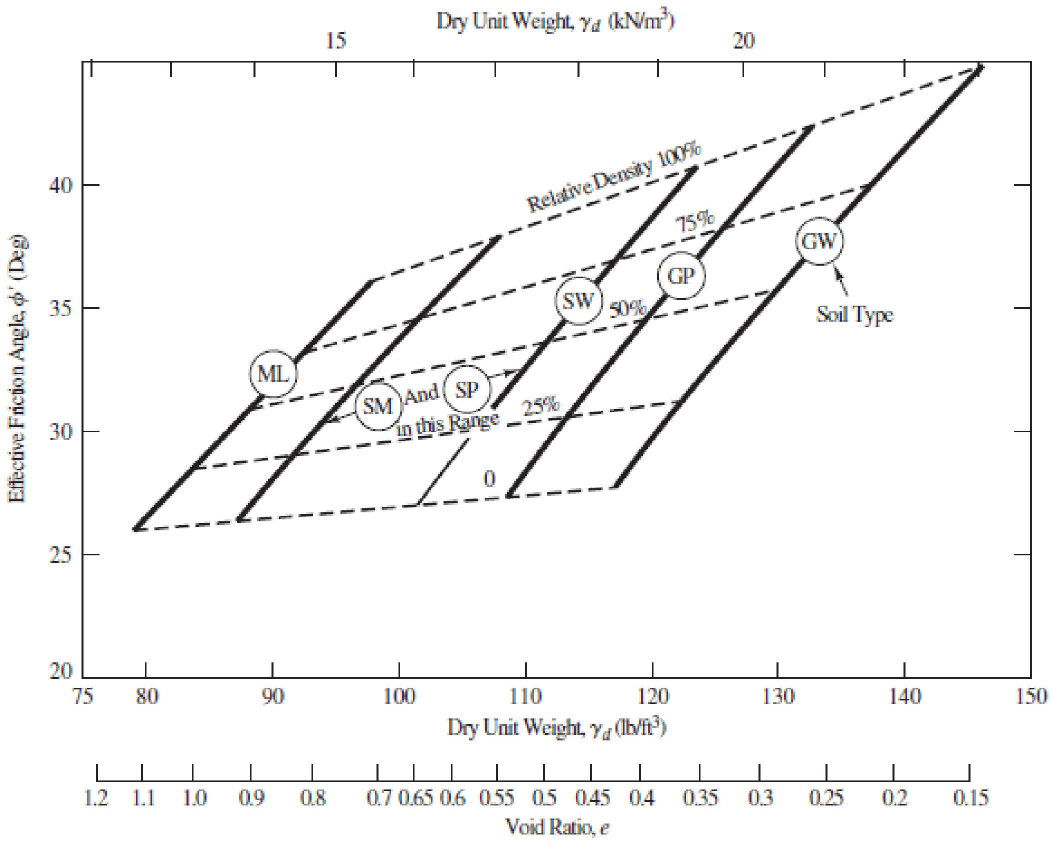

Figure 7: Relationship between effective friction angle and unit weight for sands and gravels, from [1].

Substituting values:

$$B = \frac{3,000}{\frac{4,400}{2.5} - 150} = \frac{3,000}{1,760 - 150} = \frac{3,000}{1,610} \approx 1.86 \text{ ft}$$

For constructability and conservatism, the footing width is rounded up to 2 feet.
This results in the following:
Allowable bearing pressure:

$$q_{allow} = \frac{P + 150B}{B} = \frac{3,000 + 300}{2} = 1,650 \text{ psf}$$

Ultimate bearing capacity:

$$q_{ult} = q_{allow} \cdot F = 1,650 \cdot 2.5 = 4,125 \text{ psf}$$

This design of a 2 foot wide strip footing safely supports the 3 kips/ft line load with sufficient bearing capacity and minimal excavation, avoiding both bay mud and groundwater interaction.

Settlement Analysis

A settlement analysis was performed using elastic theory. The constrained modulus M was calculated using the following equation:

$$M = E \cdot \frac{(1 - \nu)}{(1 + \nu)(1 - 2\nu)}$$

Assuming a Young's modulus E = 20,000 psf and Poisson's ratio ν = 0.45 (typical for semi-stiff clay), the resulting constrained modulus is:

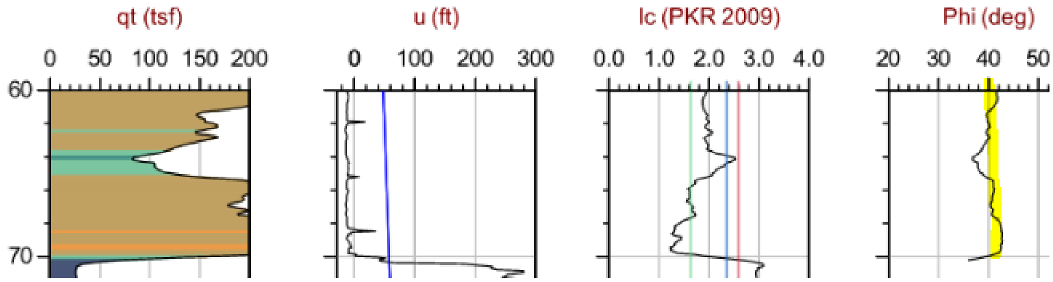

Figure 8: CPT log showing effective friction angle for a poorly graded sand.

$$M \approx 75,862 \text{ psf}$$

The total settlement was estimated by dividing the soil profile into 0.5 ft layers down to 3 feet and calculating the compression for each layer using:

$$\varepsilon_v = \frac{\Delta\sigma_z}{M}, \quad \text{Compression} = \varepsilon_v \cdot H$$

The results are shown in Table 3. The total estimated settlement is approximately 0.0198 ft, or about 0.24 inches.

Table 3: Compression Summary by Depth

Depth (ft)	σ_z (psf)	$\Delta\sigma_z$ (psf)	M (psf)	**Compression (ft)**
0.5	2,673	627	75,862	0.00413
1.0	1,815	858	75,862	0.00566
1.5	1,320	495	75,862	0.00326
2.0	1,020	300	75,862	0.00198
2.5	825	195	75,862	0.00129
3.0	293	532	75,862	0.00351
Total Compression (ft)				**0.01982**

5 Deep Foundation

The proposed structure is a four-story office building with approximate plan dimensions of 250 ft by 150 ft. Gravity column loads are expected to range from 300 to 400 kips, with column spacings of approximately 30 feet.

Given the project's location in the Bay Area, it is appropriate to use precast concrete piles similar to those used in BART construction. These are typically 18-inch square piles with maximum lengths of up to 72 feet, as referenced in [2]. Since 72 feet is the longest length that can be reliably produced by local precast facilities, pile capacity calculations are limited to this depth.

Pile capacity was evaluated using both the α- and β-methods, implemented in an Excel-based analysis tool. Skin friction capacity was calculated incrementally, foot-by-foot, up to 72 feet of depth. Based on CPT data, the groundwater table is assumed to begin at 11 feet below ground surface, and pore water pressure is considered from this depth downward.

For cohesive soils such as bay mud and clays, the α-method is used to estimate unit skin friction. For granular soils such as dense sands, the β-method is employed. α values were taken from the chart in Figure 9, based on guidance from [1]. Line A, recommended by the American Petroleum Institute, was selected. Bay mud, with an undrained shear strength of 260 psf, is assigned an α value of 1.0. Fat clay and deeper alluvial soils, which exhibit shear strengths exceeding 1500 psf, are assigned an α value of 0.5.

For sand layers, the β-method is used, with β estimated by the empirical equation:

$$\beta = 0.18 + 0.65 D_r$$

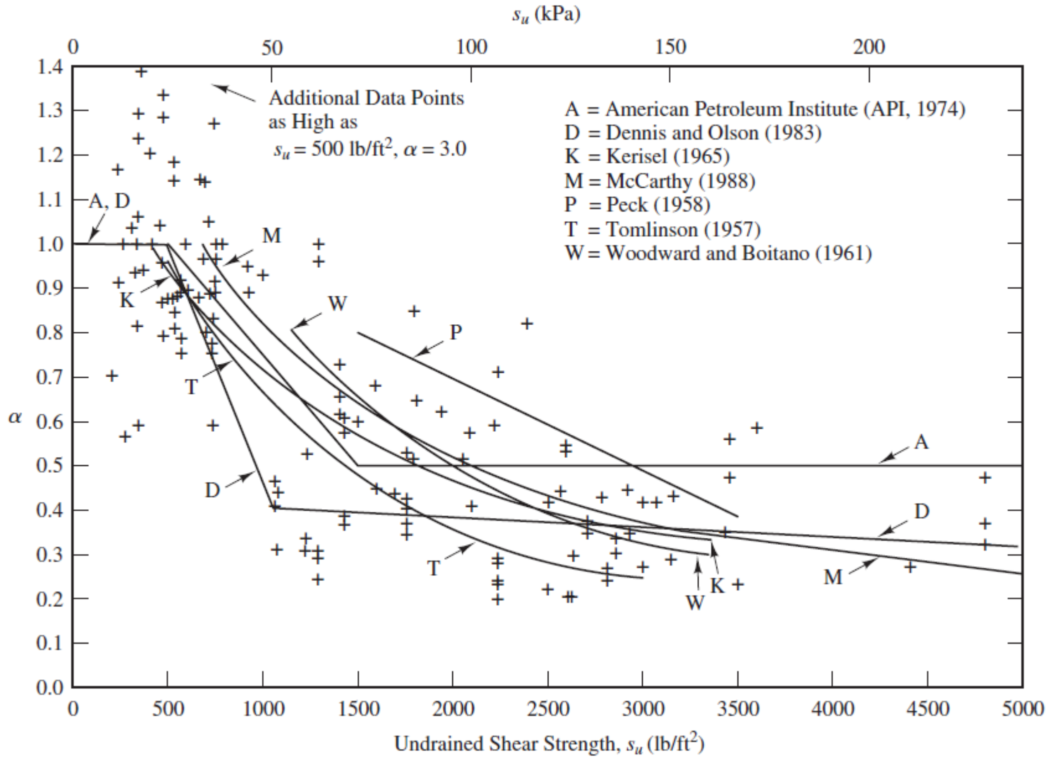

Figure 9: Chart of α values from [1].

The relative density D_r is obtained from Figure 10. As the sand is classified as dense, a relative density of 75% is assumed, resulting in a β value of approximately 0.67.

The Excel analysis begins by setting up the pile geometry and groundwater depth conditions, as shown in Figure 11.

The first location analyzed is the northwest corner of the site. This area contains no sand layers, so only the α-method is used. Layer properties including unit weight and undrained shear strength for fat clay, bay mud, and alluvium are taken from Table 2. Skin friction for each foot of depth is calculated using:

$$f_s = \alpha S_u$$

These values are then summed to compute the total skin friction capacity of the pile down to 72 feet. The stratigraphy at this location consists of 5 feet of fat clay, 6 feet of bay mud, and the remaining depth composed of alluvium and similar clays. The total skin friction capacity at this location is 435.6 kips, shown graphically in Figure 12.

This process is repeated for other key locations on the site. In areas containing sand, the Excel model uses the β-method for all segments containing granular material. In this case, the skin friction is calculated as:

$$f_s = \beta \sigma_z' \quad \text{where} \quad \sigma_z' = \sigma_z - u$$

At the northeast corner, for example, a 14-foot thick sand layer significantly increases skin friction capacity, resulting in a total capacity of 602.1 kips (Figure 13).

Analyses were completed for four corners of the site, as well as at midpoints of each cross section, resulting in eight total cases. The highest pile capacity was found in the southeast corner (701.3 kips, Figure 14), while the lowest was in the northwest (435.6 kips, Figure 12).

Results from all locations were compiled into a summary table. A safety factor of 2.5 was selected due to the availability of both CPT and SPT data across the site. Expected column loads were estimated at 300,000 lbs for corner locations, 350,000 lbs for edge center points, and 400,000 lbs at the site center.

Soil Description and Relative Density	Maximum Side Friction Capacity $f_{n,\max}$ (kPa)	(lb/ft^2)
Very loose to loose sand ($D_r = 0 - 35\%$)		
Loose sand-silt ($D_r = 15 - 35\%$)	Use CPT-based methods	
Medium dense to dense silt ($D_r = 35 - 85\%$)		
Medium dense sand-silt ($D_r = 35 - 65\%$)	67	1,400
Medium dense sand ($D_r = 35 - 65\%$)	81	1,700
Dense sand-silt ($D_r = 65 - 85\%$)		
Dense sand ($D_r = 65 - 85\%$)	96	2,000
Very dense sand-silt ($D_r = 85 - 100\%$)		
Very dense sand ($D_r = 85 - 100\%$)	115	2,400

Figure 10: Chart of β values and relative densities from [1].

CE238 Final Project Pile Analysis			
ground water depth	11	ft	
pile size	1.5	ft	square
end area	2.25	sqft/ft	
surface area per foot depth	6	sqft/ft	

Figure 11: Excel model setup for friction capacity calculations.

The number of piles required per column was calculated using:

$$p = \frac{C_a \cdot F}{C_u}$$

where C_a is the allowable capacity, F is the safety factor, and C_u is the ultimate capacity.

Table 4 presents the results of this analysis. At all locations, the number of piles required per column ranged between 1 and 2. Based on this, the recommendation is to install two 18-inch square, 72-foot-long precast concrete piles beneath each gravity column supporting the primary structure.

Table 4: Pile Capacity Summary by Location (Safety Factor = 2.5)

Location	**Ultimate Cap. (lbs)**	**Expected Load (lbs)**	**Allowable Cap. (lbs)**	**Piles Needed**
NW	435,600	300,000	174,240	1.7
NE	605,113	300,000	242,045	1.2
SW	530,462	300,000	212,185	1.4
SE	701,303	300,000	280,521	1.1
NC	502,595	350,000	201,038	1.7
EC	547,172	350,000	218,869	1.6
SC	685,046	350,000	274,018	1.3
WC	556,639	350,000	222,656	1.6
Total Average	570,491	350,000	228,197	1.5
N Average	514,436	350,000	205,774	1.7
E Average	617,863	350,000	247,145	1.4
S Average	638,937	350,000	255,575	1.4
W Average	507,567	350,000	203,027	1.7
C Average	572,863	400,000	229,145	1.7

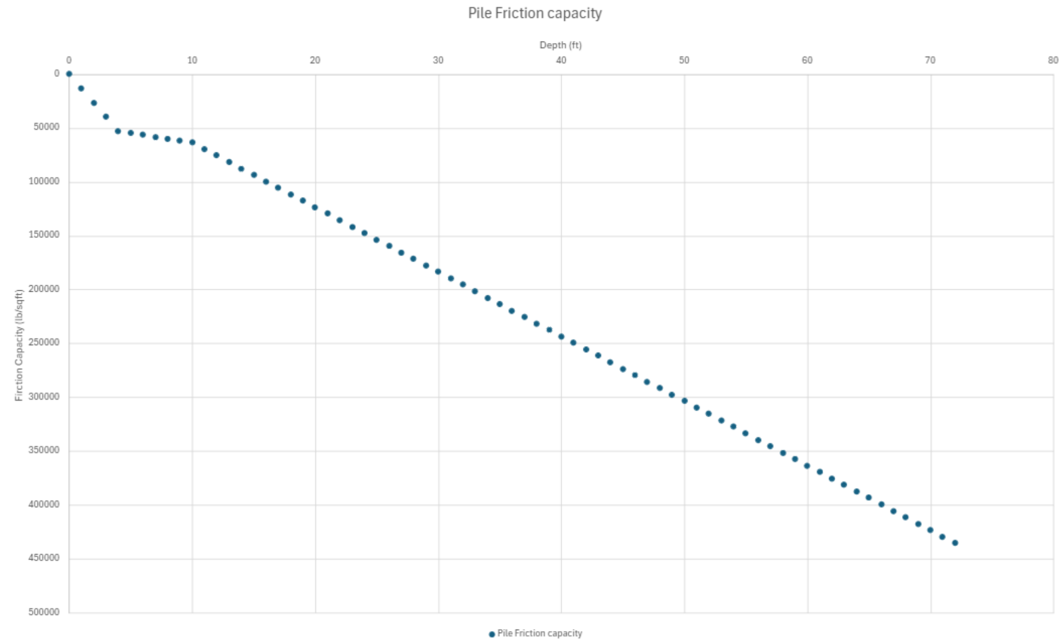

Figure 12: Skin friction capacity vs. depth for a pile in the northwest corner.

6 Conclusion

The geotechnical evaluation of the proposed development site near the San Francisco Bay Area revealed complex subsurface conditions. These conditions necessitated the use of a shallow and deep foundation systems tailored to the loading demands and soil profiles of each proposed structure. For the single-story vehicle maintenance facility, a 2-foot-wide strip footing was determined to be structurally adequate , providing sufficient bearing capacity with negligible settlement. For the four-story office building, a deep foundation system consisting of 18-inch square, 72-foot-long precast concrete piles was selected. Load analysis using alpha and beta methods confirmed that two piles per column would provide adequate capacity. The study underscores the importance of site-specific testing and stratigraphic profiling when designing foundations in geologically sensitive regions. The final design recommendations reflect a conservative and robust approach to ensure long-term structural performance.

References

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[2] C. Layman and R. D. Neve. Ebrc - celr noise and vibration assessment. Technical Report Attachment E, Volume 2, ATS Consulting, Pasadena, CA, February 2019. Prepared for BKF, Chris Adams, PE.

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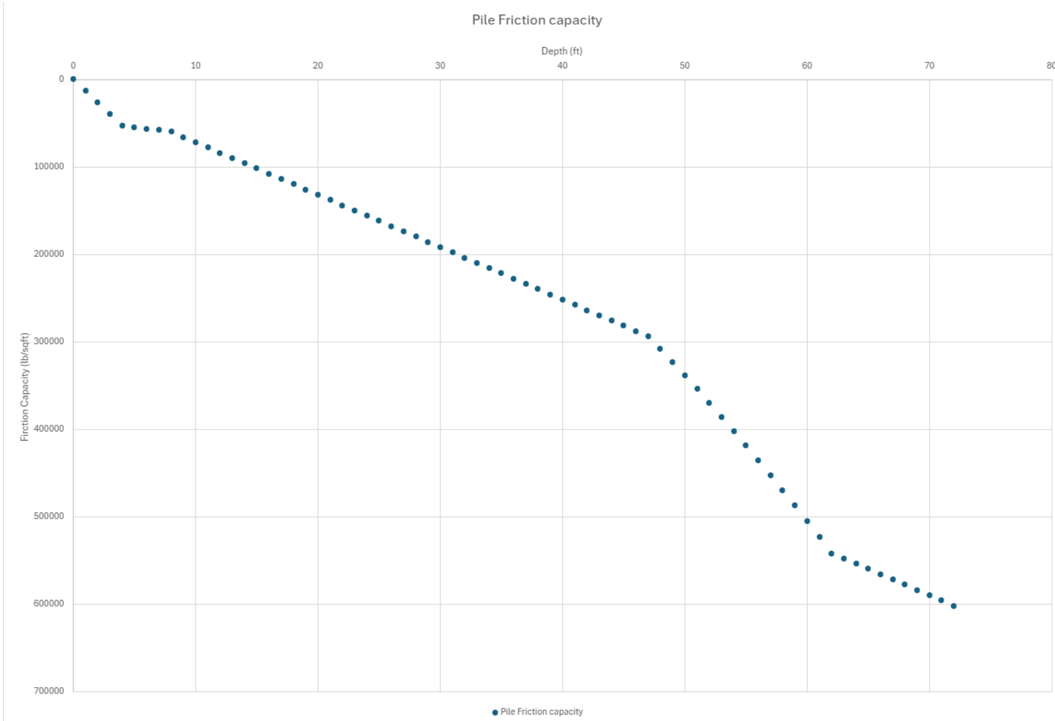

Figure 13: Skin friction capacity vs. depth for a pile in the northeast corner.

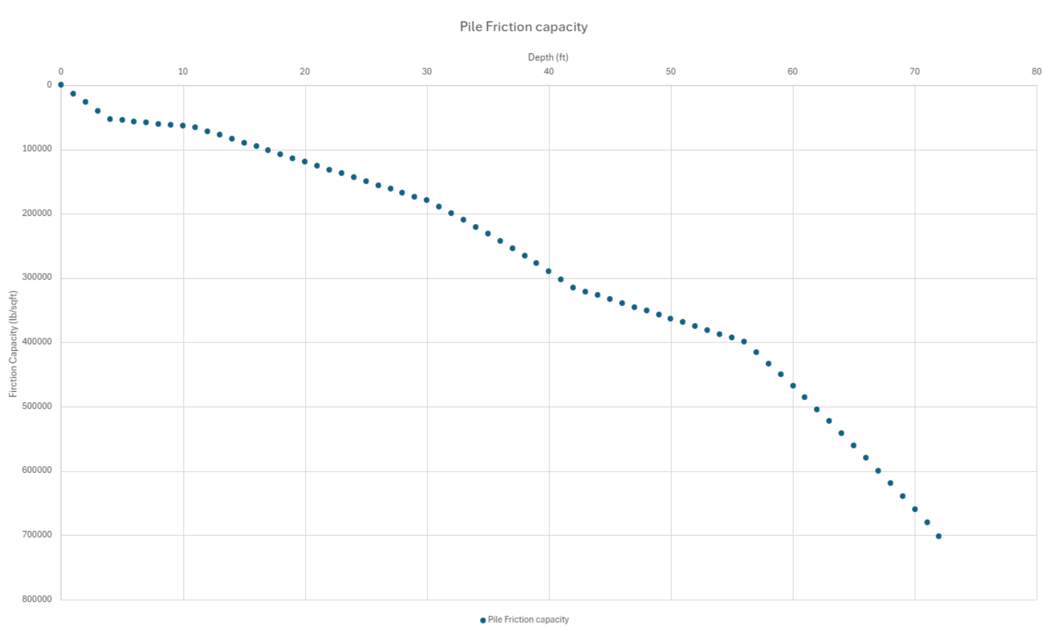

Figure 14: Skin friction capacity vs. depth for a pile in the southeast corner.