28 March 2014

**PRIVELEDGED AND CONFIDENTIAL – FOR DISCUSSION PURPOSES ONLY**

Mr. Marty Glick
Golden State Warriors
1011 Broadway
Oakland, California 94607

Subject: Preliminary Geotechnical Evaluation
Block 29-32 Mission Bay
San Francisco, California
Project No. 731617202

Dear Mr. Glick:

This letter provides our preliminary geotechnical evaluation for the proposed development at Blocks 29-32 in San Francisco. Our services were performed in accordance with our proposal dated 21 March 2014. Data from previous investigations by us and others at the site and adjacent properties were used for this study; no additional subsurface exploration was performed during this evaluation.

The project site is comprised of Blocks 29, 30, 31, and 32 in Mission Bay. The site is located within an area bound by Third Street on the west, South Street on the north, the future Terry Francois Boulevard on the east, and 16th Street on the south, as shown on Figure 1. The project area has approximate plan dimensions of 770 feet by 600 feet.

We understand the proposed development will include below-grade levels extending 6 to 10 feet below grade, occupying the majority of the site. We anticipate excavations will generally extend about 8 to 15 feet below grade, with isolated deeper excavations at building cores.

## SCOPE OF SERVICES

The objectives of our study were to evaluate subsurface conditions at the site using existing subsurface information at the site and in the site vicinity and develop preliminary conclusions for the geotechnical aspects of the proposed project. On the basis of our review of existing subsurface information, engineering analyses, and experience with development of other projects in the site vicinity, we developed conclusions and preliminary recommendations regarding:

- the soil, rock, and groundwater conditions at the site
- site seismicity and seismic hazards (specifically liquefaction)
- probable foundation type(s) for the new structure
- probable temporary shoring type(s)
- settlement behavior
- floor slab support

- 2013 San Francisco Building Code (SFBC) seismic design criteria
- construction considerations.

**SITE CONDITIONS**

Originally, the site was below water in a shallow bay known as Mission Bay. The tip of historic Point San Quentin was located just south of the site, along the 1852 San Francisco shoreline. Starting in the late 1860s, the bay was reclaimed by placing fill. A review of historic maps (Rumsey, 2003) and documents (ESA, 1990) indicates that the site was reclaimed starting around 1869 with soil and rock from nearby Irish Hill and the Second Street cut, and the filling completed between 1906 and 1910s with fill and building rubble from the 1906 San Francisco earthquake. In addition, a structure named the Long Bridge was constructed along what is now 3rd Street; this structure was a timber pile-supported bridge that crossed Mission Bay from north to south.

The site is mostly vacant, with paved parking lots in the western and northern portions of the site. With the exception of an area in the southern portion of the site, the site is relatively flat, with the ground surface elevations ranging from about 99 to 103 feet[1]. There is a depressed area in the southern portion of the site due to an excavation previously performed at the site for an environmental cleanup; the area has a plan dimension of approximately 320 feet by 280 feet and the ground surface elevation ranges from about 91 to 96 feet.

**SUBSURFACE CONDITIONS**

We previously explored subsurface conditions at the site by drilling borings and advancing cone penetration tests (CPTs) at the site. We also maintain a database that includes the results of drilling and CPT exploration by us and others at the site and in the site vicinity.

In general, subsurface conditions at the site consist of fill, Bay Mud, Colma Formation sand, clay and sand layers, and bedrock. Where explored, the site is blanketed by approximately 9 to 33-1/2 feet of fill. The thickness of fill varies significantly throughout the site; however, where explored, the fill in general is greater than 15 feet within Block 29, the northern half of Block 30 and the western half of Block 31. The fill consists of gravel, sand, and clay mixtures, with brick, rock (including serpentinite), and other rubble. The sands and gravel are loose to very dense, and the clay is soft to stiff. The fill likely also includes cobble- and boulder-sized pieces of serpentinite that were apparent from the drilling but could not be recovered within the 1.5- to 2.5-inch diameter samplers.

A weak and compressible marine clay deposit, referred to as Bay Mud, is present beneath the fill. Where explored within the project site, this layer ranges from about 2-1/2 to 46-1/2 feet thick, generally becoming thicker to the north.

A medium dense to very dense clayey sand, silty sand and sand with clay and stiff to hard sandy clay, clay with sand and clay was encountered below the Bay Mud. Where encountered the sand and clay layers total 3 to 31 feet thick.

A medium dense to very dense sand, sand with clay, clayey sand, silty sand and sand with silt, known as the Colma Formation, was encountered below the sand and clay in portions of the site. The top of the Colma formation was encountered about 19 to 70 feet below the ground surface. Where encountered,

1 All elevations reference San Francisco City Datum plus 100 feet.

the sand is approximately 5 to 35 feet thick. The Colma Formation generally becomes thicker to the north and west.

A stiff to hard clay known as Old Bay Clay, very stiff to hard sandy clay, clay, gravelly clay with sand and clay with gravel and dense to very dense sand with silt and clayey sand were encountered below the Colma Formation to bedrock. Bedrock was encountered at depths ranging from 32 to 130 feet. Bedrock generally becomes deeper to the northwest. Bedrock consists of serpentinite, greenstone, shale, and claystone of the Franciscan Complex. The rock is crushed to intensely fractured, soft to moderate hardness, and friable to weak, with deep to moderate weathering.

Groundwater was encountered about 6-1/2 feet bgs to about 7 feet bgs. The groundwater level is influenced by rainfall and tides; therefore, the groundwater level measurements may not represent stabilized groundwater levels.

## REGIONAL SEISMICITY AND FAULTING

The major active faults in the area are the San Andreas, Hayward, San Gregorio, and Calaveras Faults. For each of the active faults within 50 kilometers (km) of the site, the distance from the site and estimated mean characteristic Moment magnitude[2] [2007 Working Group on California Earthquake Probabilities (WGCEP, 2008) and Cao et al. (2003)] are summarized in Table 1.

**TABLE 1**

**Regional Faults and Seismicity**

| Fault Segment | Approx. Distance from fault (km) | Direction from Site | Mean Characteristic Moment Magnitude |
|---|---|---|---|
| N. San Andreas - Peninsula | 12.7 | West | 7.23 |
| N. San Andreas (1906 event) | 12.7 | West | 8.05 |
| N. San Andreas - North Coast | 17 | West | 7.51 |
| Total Hayward | 16 | Northeast | 7.00 |
| Total Hayward-Rodgers Creek | 16 | Northeast | 7.33 |
| San Gregorio Connected | 19 | West | 7.50 |
| Mount Diablo Thrust | 33 | East | 6.70 |
| Total Calaveras | 34 | East | 7.03 |
| Rodgers Creek | 35 | North | 7.07 |
| Green Valley Connected | 38 | East | 6.80 |
| Monte Vista-Shannon | 39 | Southeast | 6.50 |
| Point Reyes | 44 | West | 6.90 |
| West Napa | 46 | Northeast | 6.70 |

Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated

[2] Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

Moment magnitude, $M_w$, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a $M_w$ of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length.

It had a maximum intensity of XI (MM), a $M_w$ of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989, in the Santa Cruz Mountains with a $M_w$ of 6.9, approximately 94 km from the site.

In 1868 an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated $M_w$ for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a $M_w$ of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w$ = 6.2).

The 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

**TABLE 2**
**WGCEP (2008) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake**

| Fault | Probability (percent) |
|---|---|
| Hayward-Rodgers Creek | 31 |
| N. San Andreas | 21 |
| Calaveras | 7 |
| San Gregorio | 6 |
| Concord-Green Valley | 3 |
| Mount Diablo Thrust | 1 |

## SEISMIC HAZARDS

Published data indicate neither known active faults nor extensions of active faults exist beneath the site. Therefore, we conclude the potential of surface rupture at the site is low.

During a major earthquake on a segment of one of the nearby faults, strong to violent ground shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[3], lateral spreading[4], and seismic densification[5]. We used

[3] 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, low-plasticity silt, and some low-plasticity clay deposits.

the available subsurface information to evaluate the potential of these phenomena occurring at the project site.

### Liquefaction and Associated Hazards

When a saturated soil with little to no cohesion liquefies during a major earthquake, it experiences a temporary loss of strength as a result of a transient rise in pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction. The site is within a liquefaction hazard zone as designated by the California Geological Survey (CGS) seismic hazard zone map for the area titled *State of California Seismic Hazard Zones, City and County of San Francisco, Official Map,* dated 17 November 2001. However, there was no documented observation of liquefaction at this site during the 1906 Earthquake or the 1989 Loma Prieta Earthquake. During the 1906 earthquake, the site and vicinity was still under development with minimal improvements and liquefaction may not have been documented. During the 1989 Loma Prieta Earthquake, no observed liquefaction occurred; the earthquake occurred 93 kilometers away and the ground motions at the site were relatively low.

Many of the borings encountered loose to medium dense sand and gravel layers with varying silt and clay content just above or below the water table. These layers could liquefy during a major earthquake. We estimate settlement ranging from 1 to 7 inches may occur, depending upon the layer thickness and relative density. This settlement is expected to be erratic and vary significantly across the site. Because of the shallow groundwater table and the relatively shallow liquefiable deposits, we conclude ground failure, such as lurch cracking and/or the development of sand boils, could occur. The ground-surface settlement will likely be larger than estimated in areas where sand boils and associated ground failure occur.

### Lateral Spreading

Lateral spreading is a phenomenon in which a surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a channel, by earthquake and gravitational forces. Lateral spreading is generally the most pervasive and damaging type of liquefaction-induced ground failure generated by earthquakes.

We judge the potential for lateral spreading of the saturated fill layer during a strong event on a nearby fault is high.

### Seismic Densification

Seismic densification (also referred to as cyclic densification and differential compaction) can also occur during strong ground shaking in loose, clean granular deposits above the water table, resulting in ground surface settlement. Up to 4-1/2 feet of loose to medium dense sand was encountered above the groundwater table. This soil may densify in a major earthquake. We estimate seismic densification settlement will generally be less than 1/2 inch; however, our analysis indicates up to about 2-1/2 inches may occur based on the conditions encountered in one boring. The excavation for the below-grade space will remove the soil above the groundwater table; therefore, the settlement will be limited to the area surrounding the site.

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[4] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[5] Seismic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing differential settlement.

## PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

The primary geotechnical concerns at the site are the presence of liquefiable fill, the potential for lateral spreading, consolidation settlement in the Bay Mud, and the variable depth to supporting soil and rock.

### Settlement

We conclude consolidation of the Bay Mud under the weight of the existing fill is likely complete, with the exception of the secondary compression. If new fill or building loads are placed at the site, however, additional consolidation settlement of the Bay Mud will occur. The amount of settlement will depend on the magnitude of the new loads; for the loads from the planned development, settlement would be excessive. Therefore, we conclude the proposed building should be pile supported.

We anticipate that less than about one inch of settlement will occur for a properly installed pile foundation. However, settlement will likely occur adjacent to the building due to liquefaction and seismic densification, and differential settlement will occur between utilities and the building and at building entrances. In addition, if grades are raised around the building, settlement will occur due to consolidation of the Bay Mud. Flexible connections, utility hangers, and hinged slabs should be incorporated into the design to accommodate the differential settlement.

Ground settlement could have adverse effects on utilities and transitions between on-grade and pile-supported structures. Seismic densification and liquefaction settlement will also create a downward frictional load on piles, as discussed in the Foundations section.

### Soil Improvement

As previously discussed, liquefaction and lateral spreading is expected to occur at the site during a large earthquake. Based on our experience, it may not be practical to design the foundation system to accommodate the loss of support resulting from liquefaction and the lateral movements resulting from lateral spreading. Therefore, the most practical and economical solution may be to improve the fill at the site to resist lateral spreading, at a minimum, or to improve the site to resist liquefaction and lateral spreading. If the fill is improved to mitigate both liquefaction and lateral spreading, there should be minimal lateral spreading or liquefaction, and seismically-induced settlement would also be significantly reduced. Accordingly, piles within an improved area could be designed for a non-liquefied soil condition. To mitigate for lateral spreading only, ground improvement to a depth of about 15 feet below the existing ground surface would be needed. To mitigate liquefaction potential and lateral spreading, ground improvement would need to extend to the bottom of the fill, which is up to about 33-1/2 feet below existing grade.

A grid of deep soil mixed (DSM) columns or panels,[6] rapid impact compaction (RIC),[7] and/or stone columns[8] are measures that can be used to improve the fill and mitigate liquefaction and lateral spreading. The depth of improvement using RIC is limited to about 9 to 15 feet below the impact; if RIC

[6] Deep soil mixing is performed by advancing augers or a cutting tool and pumping cement slurry through the tips of the augers or tool during drilling. The soil is mixed with cement slurry in situ, forming a solidified column or panel of soil and cement.

[7] The rapid impact compaction method uses a Rapid Impact Compactor (RIC) to impart energy by dropping a 7.5 ton weight from a controlled height of about 1 m onto a patented foot. Applications include compaction of loose soils to improve bearing capacity and mitigation of liquefaction potential.

[8] Stone columns are a liquefaction mitigation measure in which crushed rock columns are installed, usually using vibratory equipment, which displaces the potentially liquefiable soil with compacted crushed rock. This technique densifies the in-place granular soil and provides a path for rapid dissipation of excess pore water pressures.

is chosen for site improvement and is able to be performed at least 5 feet below existing grade, the lateral spread hazard will likely be mitigated (partial improvement), but liquefaction potential will remain in the deeper liquefiable layers. Both DSM and stone columns should be able to mitigation liquefaction and lateral spread to their full depth (full improvement). With DSM, however, the presence of excessive debris, cobbles, or boulders in the fill would preclude a well-mixed column; further investigation is needed, likely in the form of deep test pits, to evaluate the potential for using DSM.

The soil improvement program should include pre- and post-improvement testing if RIC or stone columns are used to evaluate the improvement of the soil. Post-improvement testing should be performed a sufficient time after improvement to allow for the pore pressures to dissipate and the soil fabric to "heal". At a minimum, four weeks should be allowed.

### Foundations

The major factors influencing foundation design at the site are the presence of non-engineered, heterogeneous, liquefiable fill and soft, compressible Bay Mud, neither of which will be adequate to support the building loads. The proposed development should be supported by deep foundations that extend below the fill and Bay Mud and gain support primarily from end bearing in dense sand or bedrock.

If grades will be raised around the development, consolidation settlement will cause a downdrag[9] load on the piles. Seismic densification and liquefaction settlement resulting from a major earthquake will also subject the foundations to downdrag loads. Piles will need to be designed for the additional load of downdrag.

Deep foundations, depending on their installation method, can encounter refusal in very dense, relatively clean sand layers that are at least 10 feet thick. If significant fines are present, the pile will generally continue through the layer. Although a layer of dense to very dense sand is present in some of the borings, this layer does not appear to be continuous across the site. Where the sand is not sufficiently clean, dense, or thick, the foundations will likely need to extend to bedrock to carry the structural loads and anticipated downdrag loads. The hardness of the bedrock is variable and the embedment of piles into the rock will vary. Consequently pile lengths will vary dramatically across the site. Further investigation and an indicator pile program should help evaluate if a bearing sand layer is present and should provide more information on the depth to the top of bedrock and the likely penetration of the piles into bedrock. Even with an extensive indicator pile program, variations in driving conditions will be encountered during production pile driving, and pile length adjustments will be needed. Based on existing data, we judge that the top of the bearing layer (very dense sand or, where it is too thin or is not present, bedrock) is about 35 to 85 feet below the existing ground surface. Piles will likely extend 5 to 15 feet into the bearing layer. Therefore, piles will likely range from about 40 to 100 feet long, as measures from adjacent street grade.

The most appropriate foundation type depends on the bearing condition, the variability in the depth to and hardness or density of the bearing layer, and the pile length. In addition to these conditions, the most appropriate pile type can depend on other site constraints, such as limitations on noise and vibrations. If there are no limitations, a driven pile can be used for support of the structure; however, if noise and vibrations are limited, then a drilled pile will be the most appropriate.

[9] Downdrag is the load transferred to the pile by the settlement of the soil relative to the pile. The downdrag movement imposes a "negative" skin friction force on the pile.

The most appropriate driven pile types would consist of precast, prestressed, concrete piles, steel H-piles or steel pipe piles. Because concrete piles are cast in advance, this pile type is less able to accommodate the expected variations in length; they would need to be cast with extra length in case the bearing layer is deeper than expected and with additional reinforcing steel in the case where refusal is encountered shallower than expected. Steel piles can be cutoff or lengthened as needed during driving to accommodate the variations in length. To prevent damage to concrete piles due to cobbles, boulders, or debris in the fill, predrilling would be needed, which would produce spoils. Steel piles can be vibrated in through the fill, so no spoils would be produced.

Drilled piles would likely consist of auger cast piles, auger cast displacement piles, or steel and concrete pipe composite piles that are screwed into the ground under very high torque and down-pressure. Auger cast displacement piles are limited to about 100 feet, although not all rigs are capable of this length. All of these pile types are displacement piles, with the exception of auger cast piles, where drilling spoils will be produced during installation.

In consideration of the expected variable bearing conditions and assuming production of drilling spoils is undesirable, we judge that driven steel piles or auger cast displacement piles would be the most appropriate pile type, depending on whether noise and vibrations are of concern. Both pile types can be designed to accommodate the variation in pile length and neither installation method produces significant spoils. With regard to driven steel piles, H-piles are the most commonly used in the area.

Regardless of the pile type chosen, an extensive indicator and test pile program should be performed. Indicator piles should be driven throughout the site to provide information regarding the bearing layers and their variability. Dynamic testing should be performed throughout indicator pile driving and a number of indicator piles should be restruck a number of days after the initial drive to evaluate soil setup. In addition, after the indicator program and restrikes are performed, static compression and uplift load testing should be performed on selected piles.

The lateral capacity of the piles will depend not only on the type of pile, but also on the extent of ground improvement performed. Pile lateral capacity will be greater with full improvement than with partial improvement.

### Floor Slabs

Because of the weak condition of the existing fill and Bay Mud and the potential for settlement, we judge floor slabs should be structurally supported. Although the ground surface may settle away from the slabs, the slabs will initially be in contact with the ground. Where moisture is not acceptable, slabs for any at-grade portions of the development should be underlain by a vapor retarder membrane and capillary break. Basement slabs that extend near or below the water table should be waterproofed and be designed to resist hydrostatic uplift.

### Seismic Design

If the proposed buildings are designed in accordance with the 2013 San Francisco Building Code (SFBC), the requirements for Zone 4 should be used as a minimum. Because of the presence of potentially liquefiable soil, the site will be characterized as site class F and site-specific response spectra in accordance with the requirements of the SFBC will be required in the final geotechnical report.

However, if the fill is improved and liquefaction is mitigated throughout the entire site (full improvement), the structure can be designed in accordance with 2013 SFBC using Site Class E. For seismic design in accordance with the provisions of 2013 SFBC we recommend the following:

- risk targeted Maximum Considered Earthquake ($MCE_R$) $S_S$ and $S_1$ of 1.50g and 0.60g, respectively
- site class E
- mapped $MCE_R$ spectral response acceleration parameters, $F_a$ and $F_v$ of 0.9 and 2.4, respectively
- Risk Targeted $MCE_R$ spectral acceleration parameters at short period, $S_{MS}$, and at one-second period, $S_{M1}$, of 1.35g and 1.44g, respectively
- Design Earthquake (DE) spectral acceleration parameters at short period, $S_{DS}$, and at one-second period, $S_{D1}$, of 0.90g and 0.96g, respectively.

**Excavation and Temporary Shoring**

We anticipate an excavation about 8 to 15 feet deep will be needed to construct the basement level, with some localized deeper excavations likely needed in core areas. Excavation of a basement will require the use of temporary shoring to laterally restrain the sides of the excavation and limit the movement of adjacent improvements.

The soil to be excavated consists of gravel, sand, and clay mixtures, with brick, rock (including serpentinite), and other rubble, which can generally be excavated using conventional earth-moving equipment such as loaders and backhoes; however, some boulders and large debris may be encountered, including old foundations. The soil at the bottom of the excavation will be saturated and Bay Mud will likely be encountered in some areas. We anticipate that installation of a gravel working pad, one to two feet thick, will be required to stabilize the bottom of the excavation. Where the excavation extends below the groundwater table, dewatering will be needed.

Several types of shoring can be used at the project site. The most appropriate shoring system should take into account the requirements for protecting adjacent improvements against vertical and lateral movements, the ability to reduce groundwater flow into the excavation, and cost. We have qualitatively evaluated the following systems:

- conventional soldier pile and lagging
- DSM cutoff wall
- secant pile wall

A soldier pile and lagging system can be used to laterally restrain the sides of the excavation. However, this system will not prevent groundwater from flowing into the excavation. Therefore, this system should be used in conjunction with an active dewatering system where the groundwater level is drawn down below the proposed excavation level. Dewatering below the top of the Bay Mud level will be difficult because of extremely low permeability of the mud. Dewatering may cause settlement of nearby improvements; the effects of dewatering will need to be evaluated.

Deep soil mixed (DSM) cutoff walls are installed by jet-grout methods or by advancing hollow-stem augers or a cutter tool and pumping cement slurry through the tips of the augers or tool during auger withdrawal or by. The soil is mixed with the cement slurry in situ, forming continuous, overlapping, soil-cement columns. Steel beams are placed in some of the soil-cement columns to provide rigidity. In addition, DSM walls are generally more rigid than soldier-piles and lagging and usually result in less shoring deformations. The presence of extensive debris, cobbles, and boulders in the fill could preclude the use of DSM; as discussed above, further evaluation of subsurface conditions should be performed. If DSM is used, predrilling could be performed as an additional measure.

Secant piles are drilled shafts that overlap to form a continuous wall. The wall is constructed by drilling alternate shafts and then "back stepping" to drill the intervening shafts in order to interlock the two adjacent shafts. Every second shaft is reinforced usually with a wide flanged steel section or reinforcing steel cage. The reinforced shafts are called "primaries". The alternate shafts, which are not reinforced, are called "intermediates" or "secondaries". The concrete used for the secondary piles is usually lean concrete that remains soft enough for the drilling and interlocking of the primary shafts. The primaries are usually poured with structural concrete.

Both DSM and secant pile walls are continuous and will act to temporarily cut-off groundwater infiltration, resulting in less dewatering.

Depending on the depth of the excavation and the stiffness of the shoring, tiebacks may be needed for lateral restraint. Note that the fill is extremely variable and the Bay Mud is weak. These materials will need to be considered in the shoring design.

**Construction Considerations**

Brick, rock, concrete, and other building rubble may be encountered in the fill. Boulders and cobbles are likely present. In addition, old foundations may be present. Therefore, installation of shoring and foundations may be difficult in some areas of the site. The fill also contains serpentinite, which may require special handling and disposal during excavation.

To accommodate variability in pile length, piles may be shorted or lengthened during driving. If piles are installed from existing grade, driven piles will need to be driven with a follower, and auger cast displacement piles will be installed with sacrificial length. If steel piles are used with a follower and refusal is not encountered, it will be extremely difficult, if not impossible, to weld extra length and continue driving. Piles driven using a follower will not be able to be restruck. In addition, pile stickup will be encountered during excavation of the basement. Therefore, in order to allow for the most flexibility in pile installation, we recommend installing piles from the basement level. A suitable working pad should be constructed to allow for the equipment needed for foundation installation within the excavation.

We trust this letter provides the information you need. The conclusions and recommendations presented herein are preliminary and should not be relied upon for design. Detailed geotechnical analysis should be performed to provide final conclusions and recommendations regarding the geotechnical aspects of the project. If you have any questions, please call.

Sincerely yours,
LANGAN TREADWELL ROLLO

Lori A. Simpson, GE
Principal/Vice President

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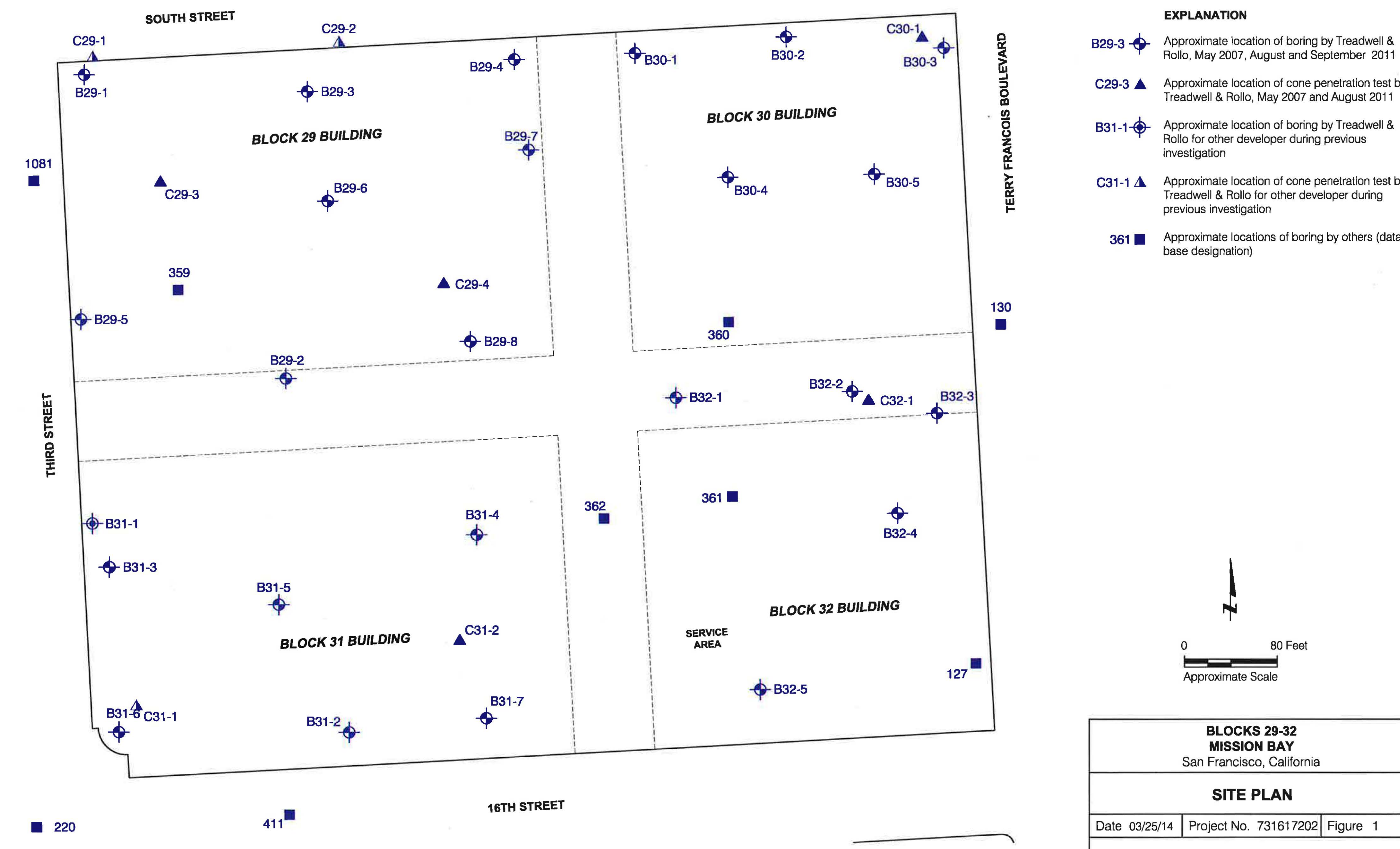
Attachments: Figure 1 – Site Plan

cc: Mr. Clarke Miller – Strada Investments




**BLOCKS 29-32**
**MISSION BAY**
San Francisco, California

**SITE PLAN**

| Date 03/25/14 | Project No. 731617202 | Figure 1 |
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**LANGAN TREADWELL ROLLO**

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