September 27, 2019

Clerk, San Francisco Board of Supervisors
1 Dr. Carlton B. Goodlett Place
City Hall, Room 244
San Francisco, CA 94102

RE: Case No. 2014.0948ENX 344 14th Street
Appeal of the July 25, 2019 Planning Commission Decision

Dear President Yee and Members of the Board Supervisors:

Please accept this submission on behalf of Our Mission No Eviction in respect to its appeal of the proposed project at **344 14th Street.**

This project's "tiering" off of an outdated PEIR is highly problematic and may result in unintended harmful impacts if not given proper study with accompanying mitigations. The site's soil samples were inappropriately studied during an outlier period of extended drought. There are other significant concerns outlined below.

Summary of Concerns

In recent years San Francisco's Mission District has seen unprecedented and accelerated growth, placing unanticipated pressures on residents and the systems they rely on to live in an urban environment. These pressures have harmfully impacted the neighborhood's most vulnerable residents the most acutely.

When The Eastern Neighborhoods EIR (PEIR) was prepared in 2008, it had no way to predict the extraordinary changes coming to the Mission District. It had no way to predict this rapid rate of development, the creation of the TNC model, and the cultural shift to near absolute use of delivery services by high-income newcomers for shopping and services. And now, our systems and residents are paying the price of woefully low cumulative impact projections, inadequate impact fees, delayed infrastructure updates, and hyper-gentrification. As a result of concerns that development would stall during the 2008 recession, impact fees were set at only 1/3 of the actual needs, and adequate alternative funding sources have never been identified.

The PEIR assumed the construction of up to 2054 new units in the Mission between 2008-2025 and yet at least 3,923 units (including BMR units) are in the pipeline as of Q2

2019.[1] These PIER assumptions have fallen woefully short of actuals and did not come close to foreseeing the unprecedented rate of market-rate development in the Mission. With housing development assumptions this far from reality, the mitigations of the Mission Area Plan are no longer appropriate or acceptable for use. The number of pipeline units is more than twice the number of "preferred project" units recommended in the Mission Area Plan for the Mission District - 1,696. It's nearly double what was evaluated in Option C - 2,045. And we still have 6 years left on this 17-year timeframe.

Our transit systems are stressed to their limits, our aging sewer system - with some parts over 100 years old - is taking on unpredicted capacity. The city's traffic problem is now world renowned, pedestrian and bicycle injuries are increasing, and displacement continues to bring its trauma to the doorsteps of our most vulnerable residents and businesses.

I. 344 14th Street - Proposed Project

The project sponsor has proposed to construct a 60-unit seven story, 78 ft. tall building (The Project) with approximately 5,890 square feet of ground floor retail use utilizing the density bonus law. Eight of the 60 total housing units (13.3% overall) will be affordable as required by Section 415. It is in the Mission District adjacent to a recent project at 380 14th Street, and within 600 feet of projects at 1801 Mission Street and 1863 Mission Street, both currently under construction. The residential entrance to the Project is on 14th Street.

The only environmental review for the Project consisted of a Community Plan Evaluation (CPE)[2] that tiered off of the 2008 Eastern Neighborhoods Plan EIR (PEIR).[3] The fact that it tiered off of the PEIR without performing adequate supplemental analysis renders the findings of the CPE incomplete.

PEIR Tiering Practice

CEQA allows broader EIRS, such as area plan EIRS, to address cumulative impacts, leaving the CPE of an individual project to focus on project specific impacts. (CEQA Guidelines Section 15152). This process, called "tiering", relies on the effectiveness of the environmental analysis and integrity of the underlying EIR. However, if the underlying EIR is flawed, outdated, or missing valuable areas of environmental study, it is no longer a viable tool for evaluating cumulative CEQA impacts. Because the 2008 Eastern Neighborhoods Plan EIR is outdated and missing valuable areas of environmental study, it is no longer a viable tool for evaluating cumulative CEQA impacts.

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[1] See exhibit A page 7, Mission Projects, Units built, entitled or in the pipeline 2008 -Q2 2019

[2] CPE-IS FINAL_344 14th Street_2014.0948ENV_053019

[3] http://sf-planning.org/sites/default/files/FileCenter/Documents/3995-EN_Final-EIR_Part-3_Land-Use_Plans.pdf

Watershed Background

The project site lies in the Channel watershed, the second largest watershed in the Bayside Drainage System. Poor soils contribute to Channel's highly impervious land cover -- at 83% it is the most impervious of the five Bayside Drainage Basin urban watersheds. This location is well documented as a high liquefaction zone, part of the Maher Zone and Historical Infill Area with sandy infill soil and likely infill debris from Woodward's Gardens which was located at this site from 1866-1891. In fact, the location for this project site includes the location of the former Rotary Boat Pond of Woodward's Gardens, fed by the Old Arroyo Dolores.[4]

The Channel urban watershed contains the greatest quantity and density of property at risk for potentially significant flood damage, with the vast majority of risk areas located along historical creek channels. These areas along the Hayes, Old Arroyo Dolores, Arroyo Dolores and Mission Creek channels are likely to experience excess flow during large storms and occasionally during smaller storms as a result of the impact of urbanization and the increase of impervious surfaces and accompanying sewer systems. Higher peak flows are produced more quickly after rain hits the ground than what has historically been typical.

II. Missing Information Affecting Environmental Analysis

Inadequate study and lack of information affecting environmental analysis related to geotechnical study and hydrology was brought to the attention of San Francisco Planning Environmental Planners, the Project Sponsor, and the Planning Commission, yet it remains unaddressed, rendering the CPE incomplete.

1. **Geology, Hydrology and Soils.** Soil testing and geotechnical review was performed in early 2016 by Rockridge Geotechnical at the project site which lies in the Channel watershed, along the Arroyo Dolores Creek. SFPUC last reported on the urban watershed conditions of the Channel watershed as part of their Sewer System Improvement Program in 2013. Both of these reviews are insufficient due to recent changes which couldn't have been studied at the time they were written.

    a. **Soil samples were taken after outlier period of extended drought.** Groundwater-level measurements of borings and cone penetrometer tests (CPTs) were taken "after several years of severe drought," likely to "represent the lower end of the spectrum," and the "groundwater level may not have fully stabilized at the time of the measurements."[5] Heavy rainfall during the 2017-2018 and 2018-2019 seasons has made the current soil conditions different from what was tested in the spring of 2016, and possibly different from what has been historically tested. More up to date CPTs are required to understand the current soil conditions as a result of the heavy rainfall and to

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[4] See exhibit A page 9, SanbornOverlayWG_waterFeatures

[5] See exhibit A page 10, 14th Stevenson_GI_Report_Final_20160506

ensure all reasonable mitigation measures are taken to prevent harmful impacts.

   i. One of the CPTs adjacent to 82 Woodward Street, CPT-2, could not advance more than a foot due to obstruction, likely the remaining foundation of the College of Physicians and Surgeons building that was demolished in the 1970s. This condition meant that soil conditions adjacent to existing historic resources could not be analyzed. As a result of this failure to properly analyze, further soil samples should also be taken in this area to ensure all reasonable mitigation measures are taken for the preservation of adjacent historical resources.

b. **Limitations.** Recommendations made in the geotechnical report are "based on the assumptions that the subsurface conditions do not deviate appreciably from those disclosed in the initial borings and cone penetrometer tests."[6] ***However, we know that those tests were performed in an outlier year with exceptional conditions.*** We also know that relevant climate change predictions and its potential impacts were not addressed in either PEIR or CPE despite the San Francisco Public Utilities Commission expressing it as a major watershed concern in 2013.[7]

c. **Actual building settlement could be significantly greater than estimated.** The site is on loose to medium density sandy fill above a groundwater table that is susceptible to cyclic densification. Liquefaction analysis using the same borings and cone penetrometer tests revealed that the bearing capacity of the proposed building would be greatly reduced and potential for liquefaction increased during an earthquake should a shallow foundation system be used without significant soil improvement.[8] The resulting liquefaction would pose not only a danger to the foundation system of the existing building, but to those around it as well. Were this danger to occur, it also poses a significant life threatening danger to the residents of these buildings. Further reasonable testing that is more up to date should be required to ensure that sufficient mitigation measures are being taken prior to project approval.

d. **No cumulative impact on the existing sewer system was studied.** Groundwater that flows into the sub-basements of 6 recent buildings within 600 feet of the proposed project is continually pumped into the San Francisco storm/sewer system.[9] If the pumps are currently working, the SF Armory also adds to this load. This groundwater pumping in concert with increased,

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[6] See exhibit A page 10, 14th Stevenson_GI_Report_Final_20160506

[7] https://sfwater.org/modules/showdocument.aspx?documentid=4147

[8] See exhibit A page 10, 14th Stevenson_GI_Report_Final_20160506

[9] See exhibit A page 18, Dewatering Sites

unanticipated sewer loads, last studied for a population of 789,200 in 2013,[10] resulting from overbuilding as well as land use changing from industrial to residential/mixed use, has the potential to exceed the capacity of a system that was not designed to accommodate this volume.[11] Historically, neighbors adjacent to the SF Armory have experienced sewage problems which correlate with large events held at the Armory.

We cannot know the cumulative impacts of climate change, land use changes and overbuilding on groundwater flows, nor the increased loads on our aging storm/sewer system because they have not been studied.

2. **Impacts on Cultural Resources.** "A project that may cause a substantial adverse change in the significance of an historical resource is a project that may have a significant effect on the environment." (PRC div 13 § 21084.1) Therefore, CEQA Guidelines (CEQA Guidelines Section 15183) require analysis of the potential for substantial adverse change to Historic Resources, *yet no study of the potential impacts to the adjacent Woodward Street Romeo Flats Historic District or San Francisco Armory are present in the CPE.* In fact, the only mention of potential impacts, including potential damage, was an acknowledgement in the CPE that letters expressing concerns had been received.

    a. **Cumulative impacts on groundwater conditions were not studied.** At least 6, and potentially 7, existing buildings are currently diverting groundwater within 600 ft of the project site, with 2 of these projects adjacent to the project site.[12] No cumulative study has been done as to impacts groundwater patterns and potential for flooding in perimeter areas resulting from the foundations of these buildings and subsequent groundwater diversions, despite the knowledge that intense rainfall events are prone to resulting in property damage along historical creeks.[13]. As no study was done, there is no way to identify potential impacts to adjacent and nearby historic resources.

    b. **Current functionality and capacity of drainage and pumping system for the SF Armory was not assessed.** Geotechnical engineers acknowledged in a follow up memo in December of 2018 that it was not clear if the "underslab drainage system is still functioning" at the basement floor level of the SF Armory.[14] If the capacity and functionality of the SF Armory drainage system is in question, and the cumulative effects to drainage patterns and flooding conditions of this Project and recent adjacent developments have not been

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[10] https://sfwater.org/modules/showdocument.aspx?documentid=4147

[11] https://sfwater.org/modules/showdocument.aspx?documentid=4147

[12] See exhibit A page 69, Dewatering Sites

[13] https://sfwater.org/modules/showdocument.aspx?documentid=4147

[14] See exhibit A page 70, 14th Stevenson GW Memo_20181210 (002)

studied, it is not possible to know the potential for flooding and damage to the SF Armory.

   c. **Foundation work could require dewatering of the site.** Groundwater must remain at least three feet below the bottom of any excavation for removal of the old foundation, soil amendment and foundation work. Rockridge Geotechnical acknowledged that " the magnitude of shoring movements and resulting settlements will be difficult to estimate and rely on the contractor's skill in shoring installation."[15] They recommend a monitoring program be established to evaluate effects on existing buildings, roads and sidewalks, yet none of their recommendations appear in the Mitigation Monitoring and Reporting Program Report of mitigation measures agreed to by the Project Sponsor.

We cannot know the potential substantial adverse change that could affect the adjacent Historic Resources because the cumulative effects of changing groundwater conditions have not been studied.

III. New Information Affecting Environmental Analysis

Substantial new information affecting environmental analysis has become available. When new information becomes available, CEQA Guidelines require comprehensive analysis of these issues. (CEQA Guidelines Section 15183) Numerous changes have taken place on the ground since the adoption of the Eastern Neighborhoods EIR that require significant analysis of cumulative effects and can not be addressed on a project by project basis through a CPE. These new conditions include:

1. **An Unanticipated Rapid Pace of Development**. The PEIR was prepared in the midst of the "great recession" and did not project the steep increases in housing prices that has been especially exacerbated by the increase in high-paying jobs that have come to San Francisco. As a result, development has accelerated at a faster pace than anticipated by the PEIR. Original growth projections of the PEIR have already been exceeded and it's original growth projections have proven to be wholly inaccurate.

   a. The assumptions of population growth of the PEIR were based on a projection of 835,000 by the year 2025 requiring the construction of an additional 17,000 housing units citywide.[16] As of 2019-Q2, 55,915 housing units were entitled with 23,172 of those units either under construction or with approved building permits.[17] The SF Planning Citywide Quick Facts (July 2017) sets the

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[15] See exhibit A page 10, 14th Stevenson_GI_Report_Final_20160506

[16] http://sf-planning.org/sites/default/files/FileCenter/Documents/3995-EN_Final-EIR_Part-3_Land-Use_Plans.pdf, Page 30

[17] https://sfplanning.org/project/pipeline-report#housing-development-snapshot

population at 884,363 well above projections and likely even higher at the present point in time two years later.[18]

b. The PEIR evaluated potential CEQA impacts of forecasted housing unit growth for the Mission under a "no project" scenario, providing three different options - Option A /782 units, Option B/1,118 units, Option C/2,054 units - with the Prefered Project units of 1,696 units approved in 2008.[19] Option C anticipated the most growth and projected the largest housing production but did not evaluate environmental impacts where growth was greater than what was stated in Option C. The Mission is now well above its projected growth numbers.[20]

c. The CPE analysis of cumulative growth employs a faulty methodology by which it looks at neighborhood growth, ignoring projections from the Eastern Neighborhoods Plan, and then compares it to citywide Plan Bay Area projections. The comparison of population increase directly resulting from the Proposed Project to projected overall population throughout San Francisco is not a valid basis; the proper comparison is the Project's cumulative contribution within the area. The Association of Bay Area Governments (ABAG) projections and Plan Bay Area goals are for the whole region and cannot be the sole measure of growth at the neighborhood level. It's unreasonable to label impacts from the Project's population growth as "less than significant" by simply claiming the Project is consistent with Plan Bay Area's goals for the entire region.

We cannot know the exact issues related to cumulative impacts resulting from unanticipated rapid pace of development because they have not been studied.

2. **Gentrification Has Caused Physical Impacts due to Unanticipated Increases in Traffic and Automobile Ownership**. The unanticipated influx of high earners in the Mission has resulted and will continue to result in a substantial increase in the rate of automobile ownership and TNC use in the Mission. It is now well recognized that high earners are more likely to own an automobile than their low income counterparts even in transit rich areas such as the Mission, and drive significantly more miles, taking more "discretionary" trips.[21] The TNC "ride-share" usage, increased frequency of residential deliveries (amazon, online retail, meal, grocery), and private buses have resulted in significantly changed traffic patterns.

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[18] https://sfplanning.org/neighborhood/citywide

[19] See Exhibit A page 74, PEIR Forecast Growth and Rezoning Options

[20] See exhibit A page 7, Mission Projects, Units built, entitled or in the pipeline 2008 -Q2 2019

[21] https://docs.lib.purdue.edu/cgi/viewcontent.cgi?referer=&httpsredir=1&article=1685&context=jtrp#page=98

a. **Unanticipated traffic increases have made our streets more dangerous for pedestrians and cyclists.** Transit Network Company ride-hails (TNCs) were first defined in 2013, several years after the PEIR was published and the Eastern Neighborhoods Area Plan was adopted. Mode share analysis for the Project fails to consider TNC's, relying in part on outdated methodology from the 2000 census. This is a serious omission. According to a recent report from the San Francisco County Transportation Authority (SFCTA), half of the City's traffic congestion and traffic delays measured from 2010-2016 is attributable to the rise of ride-hails.[22] However, joint analysis released in September 2018 by Uber and Lyft indicates that TNCs actually accounted for nearly twice the VMT estimated by the SFCTA.[23]

b. **The proposed project entrance lies on a high injury corridor.** 14th Street between Valencia and Mission Street has been identified by Vision Zero and the San Francisco Department of Public Health as a high injury corridor.[24] Our streets continue to get more dangerous as we fall short on safety improvements, bicycle infrastructure and vehicular loading due to insufficient mitigations and inadequate funding because we continue to rely on outdated traffic studies. No mitigations were made for deliveries and vehicle loading on 14th Street, nor recommendations made for infrastructure improvements for bicycles and pedestrian safety. Without further study and recommendations, we are concerned that this project may add to pedestrian and bicyclist injuries on the corridor.[25] [26]

c. **Outdated Loading Analysis.** There is no Loading Demand analysis included in the CPE, and assumptions in the trip generation studies prepared for the environmental review vastly understate the number of delivery vehicles by apparent reliance on outdated guidelines, showing only .32 deliveries an hour, or 7.68 a day. Further study is required.[27]

We cannot know the exact issues related to cumulative impacts on transportation and circulation because the underlying studies assumed a level of growth that has been exceeded, and did not anticipate transit modes such as TNC's and increased reliance on delivery vehicles.

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[22] http://www.sfexaminer.com/study-half-sfs-increase-traffic-congestion-due-uber-lyft/

[23] https://www.citylab.com/transportation/2019/08/uber-lyft-traffic-congestion-ride-hailing-cities-drivers-vmt/595393/

[24] https://sfgov.maps.arcgis.com/apps/webappviewer/index.html?id=fa37f1274b4446f1bdddd7bdf9e708ff

[25] https://www.sfchronicle.com/bayarea/article/Surge-of-critical-injuries-on-SF-s-streets-14444554.php

[26] https://www.insurancejournal.com/news/national/2019/07/09/531584.htm

[27] See exhibit A page 76, 344 14th Street Trip Generation

3. **State of Advanced Gentrification in the Mission and Disproportionate Community Benefits.** Rapid speculative growth, increase in the cost of living and a rise in the cost of housing that has followed the glut of high income earners moving into the MIssion, has led to hyper-gentrification.

    a. Hyper-gentrification has led to the displacement of long-time residents, the loss of much of the industrial sector, loss of Latinx “mom and pop” businesses, nonprofits and artists. The San Francisco Analyst reported that the Mission lost 27% of its Latinos and 26% of its families with children since the 2000s.[28] The PEIR made no mention of this exodus, nor the changes to the physical environment that would accompany it, and had it observed this phenomenon of hyper-gentrification as it was occurring, one would hope that it would have advocated for more protective measures.

    b. The protective measures provided by community benefits to mitigate the direct and indirect harms of gentrification have not kept pace with actual need. Benefits such as infrastructure, pedestrian/bicycle safety, open space and affordable housing production have not met the pace of development. As part of the Eastern Neighborhood Plan’s environmental review, a Nexus Study was prepared to determine the cost of mitigating the impacts of growth with the idea that developers would pay impact fees to fund necessary infrastructure improvements. As a result of concerns that development would stall during the 2008 recession, impact fees were set at only 1/3 of the actual needs, and adequate alternative funding sources have never been identified. The ENCAC Response to the 2015EN Monitoring Report details numerous unmet needs resulting from rapid development including the inadequacy of impact fees in addressing increasing infrastructure requirements.[29]

The impact fees required to offset the cost of providing community benefits has not been projected because cumulative impacts of hyper-gentrification and the necessary level of community benefits to mitigate the direct and indirect harm has not been studied. Also, Impact fees set during the ENP process were reduced to 1/3 of the actual needs, and adequate alternative funding has never been realized.

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[28] https://www.urbandisplacement.org/sites/default/files/images/case_studies_on_gentrification_and_displacement-_full_report.pdf, page 24

[29] See Exhibit A page 80, 2016 ENCAC Response to the EN Monitoring Reports (2011-2015)

Conclusion

CEQA Guidelines require us to assess cumulative environmental impacts based on current and reasonably anticipated circumstances.

Because there have been numerous changes on the ground and substantial new information has become available whose impacts have yet to be studied, San Francisco is utilizing flawed, outdated and incomplete environmental data for CEQA review in the Eastern Neighborhoods. Each new project that is approved without examining these cumulative environmental effects leads to the assessment of insufficient mitigation measures and delayed and inadequate infrastructure updates, to the detriment of Mission residents, and particularly its most vulnerable -- already under dire stress.

The tiered EIR process was created to allow for efficient, thorough assessment and mitigation of environmental impacts, not be a tool to disenfranchise and endanger citizens for the sake of expediency. Eastern Neighborhood's communities have historically received marginalized environmental planning. These communities, including the North Mission, deserve parity and better analysis -- because their lives depend on it.

Sincerely,

Larisa Pedroncelli
Kelly Hill
Members, Our Mission No Eviction

EXHIBIT LIST

A-2	November 28, 2018 Letter to San Francisco Planning Commission, J. Scott Weaver
A-5	October 22, 2018 Letter to Mark Kelly, Esmeralda Jardines (Project Planner) and John Kevlin, Theresa Lazzari
A-7	Mission Projects, Units built, entitled or in the pipeline 2008 -Q2 2019
A-9	Sanborn Overlay of Woodward's Gardens Water Features
A-10	May 6, 2016 Geotechnical Investigation, Rockridge Geotechnical
A-69	Dewatering Sites within 600 feet of 344 14th Street
A-70	December 10,2018 Memorandum, Rockridge Geotechnical
A-70	January 8, 2019 Geotechnical Consultation, Rockridge Geotechnical
A-74	PEIR Forecast Growth and Rezoning Options
A-76	344 14th Street Trip Generation
A-80	2016 ENCAC Response to the EN Monitoring Reports (2011-2015)

EXHIBIT A

November 28, 2018

Commissioners,
San Francisco Planning Commission
1650 Mission Street, Room 400
San Francisco, CA 94103

Re:* *Case No.2014.0948ENX 344 14th Street

This letter is with respect to 344 14th Street, Item 19 on your November 29th Agenda. This project is not yet ready for your consideration because neither the Commission nor the public have had the opportunity to review the Community Plan Evaluation.

The developer proposes a 6 story 76 foot tall building with 56 units along with a 43 car parking garage. This project is situated on 14th and Stevenson Streets, between Mission and Valencia. This area is the "Gateway to the Mission", an already gentrifying area and one that is seeing numerous projects, proposed, entitled, and/or built in the immediate vicinity. The Department has not carefully evaluated the project from the standpoint of its cumulative impacts on an area that already faces challenges with respect to traffic and circulation, noise, air quality, recreation, and open space, and displacement – especially of its SRO tenants.

Context.

The proposed project (56units) is being built in conjunction with a number of other projects currently in the pipeline for the area. Projects either built after 2008 or currently entitled in the area between the intersection of South Van Ness and Mission, and 16th and Mission and one block either side of Mission (eight blocks) include: 1601 Mission Street (354 units), 1724-1730 Mission Street (39 units), 1801 Mission Street (54 units), 1863 Mission Street (37 units), 1880 Mission Street (202 units) 1900 Mission Street (9 units), 1924 Mission Street (13 units), , 198 Valencia (28 units), 235 Valencia (40 units), 411 Valencia (16), 80 Julian (9 units), 380 14th Street (29 units), 1501 15th Street (40 units), and 1587 15th (26 units). Additionally, there are two affordable housing projects, one at 1950 Mission Street (157 units), and one at 490 South Van Ness Avenue (81 Units). This is a total of 1,134 units built or entitled. In addition, there are at least two additional unentitled projects in the area, 1979 Mission Street (331 units), and 1500 15th Street, (184 units – density bonus), raising the total to 1,649 units in an eight square block area.

Further compounding the matter, the Armory, at 1800 Mission Street, proposes to convert 49,999 square feet of video production space to office use, and 25,385 square feet of video production to entertainment (dubbed "the Madison Square Garden of the West") That translates into three hundred or more office workers and thousands attending evening events. this is incorrect. it will be office space and PDR. get with peter on the exact percentages

The proposed Market/Van Ness "Hub", a four block walk from the project site, will consist of between 7.300 and 9,000 residential units.

This is extraordinary for such a small geographic area. The total number of units contemplated under the most ambitious scenario for the Mission in the Eastern Neighborhoods Plan was 2054 units, with a Preferred Project at 1696 units. To provide a sense of proportionality, the Mission Area Plan is approximately 72 blocks, whereas the number of blocks considered above is eight.

This project, when looked at cumulatively results in significant impacts on the immediate area, including impacts on traffic, circulation, air quality, noise, and open space, as well as socio-economic impacts on this a working class neighborhood and an especially vulnerable SRO Hotel population.[1] Once these projects are in place, existing SRO tenants will be ousted and replaced by will be gone, replaced by tourists, and - need to finish this sentence

<u>Cumulative Impacts Require Examination</u>

Under Public Resources Code Section 21083 subdivision (b)(2).) "The possible effects of a project are individually limited but cumulatively considerable. As used in this paragraph 'cumulatively considerable' means that the incremental effects of an individual project are considerable when viewed in connection with the effects of past projects, the effects of other current projects, and the effects of probable future projects." Stated otherwise, a lead agency shall require an EIR be prepared for a project when the record contains substantial evidence that the "project has possible environmental effects that are individually limited but cumulatively considerable." (Guidelines section 15065 subdivision (a) (3).)

Therefore, the impact of the proposed project should be evaluated in conjunction with the cumulative impacts it <u>and</u> the additional 2,000 plus units would have on the eight block area immediately surrounding it. **No such evaluation has been done, and is necessary given the extraordinary number of units being proposed for such a small area.**

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[1] Tis oncoming wave of gentrification will result in a significant reduction in traditional SRO residents as Hotel owners "upgrade" their units. Currently there are hundreds of SRO units within the area between Duboce and 16th Street, Valencia and South Van Ness Avenue.

The environmental assessment of this project consisted largely of a yet to be reviewed CPE for the proposed project which was dependent solely on the 2008 Eastern Neighborhoods Plan EIR (PEIR). The PIER envisioned a scenario of up to 2054 units in an area nine times the size of the subject area. Further, this evaluation did not consider subsequent new information impacting the environment (discussed in greater detail below). Cumulative analysis in this area of heavily concentrated development is required in order to inform on substantial environmental impacts, and to adopt necessary and appropriate mitigation measures. Reliance almost exclusively on the PEIR in this instance does not provide the required information.

Cumulative impacts on traffic and circulation are especially appropriate in this particular circumstance. Mission Street, Valencia Street, 14th Street and Duboce Street are highly traveled areas that will be further impacted. The existence of bicycle lanes on both Valencia and 14th Streets raise serious issues bicycle safety. The addition of nearly 2,000 units will only make matters worse and will cause further congestion affecting automobile drivers, cyclists, and commuters traveling along the many bus lines that travel through the area. Red lanes, "ride sharing vehicles," and "Amazon deliveries by UPS and other carriers will further complicate the traffic patterns. Moreover, the intersection of Duboce Avenue and South Van Ness is already a traffic nightmare and a dangerous intersection for pedestrians. The addition of these units will greatly complicate that mess.

In addition to traffic and circulation, there are issues related to noise (the 101 Freeway crosses Mission Street very close to the proposed project). Open space is virtually non-existent, yet the thousands of people who would move to the area would require it. There is no recreation to be provided - other than the local bars which will undoubtedly increase exponentially as the Mission becomes more and more of a party zone.

Finally, the cumulative gentrification impacts would effectively wipe out small mom and pop businesses and SRO Hotels in the immediate eight block area and will radiate down Mission Street.

Simply put, neither the CPE nor the PEIR provide adequate information regarding potential cumulative impacts in this highly concentrated area. As a result, mitigation measures that would ease these impacts have not been put in place.

More Rigorous Evaluation is Requested.

More rigorous of this and the other related projects listed above is necessary, not only in light of the CEQA issues raised by the lack of cumulative impact study, but also in terms of the goals of the Eastern Neighborhoods Plan,……………………………..

Sincerely,

J. Scott Weaver

JSW:sme
cc Plaza 16 Coalition
bcc numerous

Theresa Lazzari <tlazzari2@yahoo.com>
Sent: Monday, October 22, 2018 9:37 PM
To: Mark Kelly; Jardines, Esmeralda (CPC); jkevlin@reubenlaw.com
Subject: 344 14th Street and 1463 Stevenson Street Project

Dear Ms. Jardines and Mr. Kevlin,

I am the property owner of the 82-84 Woodward St building directly adjacent to this project, and have seen the plans from 2016 when the initial showing of them occurred. I haven't seen the most recent changes to the plans. Regardless, I have a number of concerns:

1. One design complaint is that the proposed building is shored up against the light wells in my building which deliver considerable light to bedrooms, living rooms and water closets. I spoke to Chris, the architect that was present at the 2016 showing of the plans, about this and he indicated that he would "work to change the design," such as mirrored light wells in the proposed complex. I would like to see this change in writing and in blueprints before things get started.

2. Since my building is the first adjacent to the proposed complex, I have serious concerns about the excavation and construction’s impact on my 100+ year old Victorian. I have consulted with a San Francisco based structural engineer, Monte Stopp, and would like to request and discuss the following:

a. That M. M. Stevenson, LLC or current developer - pay for a structural engineer, of my choosing, to review every square inch of the interior, exterior and foundation of 82-84 Woodward St. PRIOR TO ANY GROUND BEING BROKEN, and that the report produced act as a guide to any structural change and damage that might occur throughout the construction of the proposed project;

b. That the same structural engineer inspect my building upon completion of the project, document any changes in structure, and that changes/damage is repaired at M.M. Stevenson, LLC's or current developer's cost;

c. I would also like my designated structural engineer to review the construction plan, prior to the project launch, to ensure it meets San Francisco guidelines;

d. I want to see the “Underpinning Agreement” for the project and hire an attorney to review the agreement, at M.M. Stevenson, LLC’s or current developer's cost. I was told at the 2016 meeting that the "rebar that is extended under my building to create structural support will create more seismic stability for my building.” The engineer I spoke with indicated that is "not necessarily true”. No question, there is a lot that can go wrong. So let’s collaborate and ensure things go right.

e. Assuming these requests are honored, and the project is expedited, I need to ensure the safety of my building’s tenants throughout the construction. If there is any aspect of the construction that creates any risk to their safety (such as the underpinning of the building), then M.M. Stevenson, LLC or current developer, needs to pay at minimum, the San Francisco Renter’s Board standard rate to temporarily relocate my tenants until the safety risk is resolved. I believe the rate is $350 per day, per tenant, at this time, although it may have gone up since I last checked this out. I have a total of 5 adults living in my two flats;

f. That a copy of the developer's current insurance be provided for my attorney to review prior the start of any excavation or construction.

I’m not trying to be difficult, but this building means a lot to me. It’s not just a rental property. My father grew up in this building and it holds much folklore and family heritage. My great grandfather actually paid to have it built in 1912, towards the end of the district's reconstruction after the 1906 earthquake and fire. I also lived there for several years while in grad school and would like my children or another family, if I decide to sell it, be safe in a structurally sound dwelling.

Please let me know how best to proceed. I'm happy to meet with you, or anyone else, to discuss my concerns and requests. I appreciate your time and consideration.

Theresa A. Razzano =8^)

There are only two ways to live your life. One is as though nothing is a miracle. The other is as though everything is a miracle. -- Albert Einstein

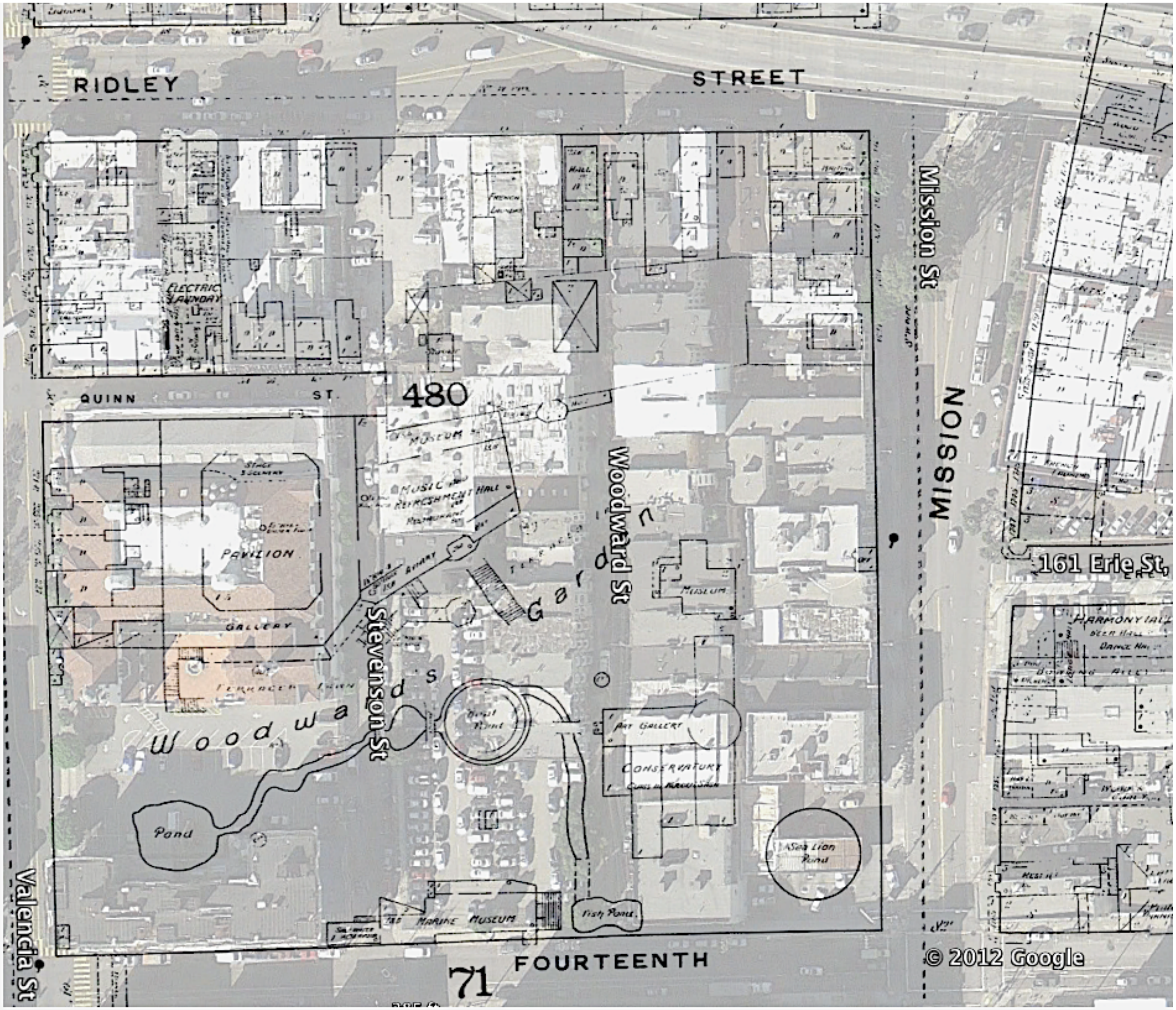
Mission Projects 2008-Q2 2019

address	street	units	units entitled	units built	
344	14th Street	60			
380	14th Street				
1450	15th Street			23	
1501	15th Street			40	
1785	15th Street			8	
1721	15th Street	23			
1500	15th Street	184			
2435	16th Street		53		
3420	18th Street			16	
2750	19th Street		60		
3500	19th Street			17	
2799	24th Street		4		
3230-36	24th street	21			
3418	26th Street			13	
3357-59	26th Street	7			
2000-2070	Bryant Street			194	
2000-2070	Bryant Street		130		
1798	Bryant Street	131			
792	Capp Street		4		
606	Capp Street			20	
3314	Cesar Chavez		52		
3620	Cesar Chavez		28		
750	Florida Street	92			
321	Florida Street	151			
2675	Folsom Street		117		
2070	Folsom Street		127		
1990	Folsom Street		158		
2600	Harrison Street		20		
80	Julian Street			8	
2550-58	Mission Street			114	
1875	Mission Street			39	
1801	Mission Street			17	
1863	Mission Street		37		under construction
1924	Mission Street		12		
1979	Mission Street	331			
1900	Mission Street		11		
1726-30	Mission Street		40		
2100	Mission Street		29		
2918-24	Mission Street		75		

address	street	units	units entitled	units built	
	Mission Street		157		
1880	Mission Street			202	also 1600 15th Street
2632	Mission Street	16			
3178	Mission Street	4			
480	Potrero			84	
346	Potrero			72	
1458	San Bruno Avenue		205		
1296	Shotwell		96		
490	South Van Ness	87			
600	South Van Ness			27	
1515	South Van Ness		157		
793	South Van Ness		73		
986	South Van Ness	15			
953	Treat Street		8		
1298	Valencia Street			35	
411	Valencia Street			16	
1021	Valencia Street	25			
1120	Valencia Street	18			
1198	Valencia Street			52	
1050	Valencia Street			16	
899	Valencia Street			18	
198	Valencia Street		24		
235	Valencia Street		50		

units in the pipeline	1,165	Number of units studied under EIR project options:	
units entitled	1,727	**Option A**	762
units built	1,031	**Option B**	1118
		Option C	2054
total units	**3,923**	**Preferred project approved in 2008 EIR**	**1.696 units**

This information was provided through Planning Department Data and SF Property Information Map. Most projects with fewer than 10 units have been excluded.


RIDLEY STREET
QUINN ST.
480
MUSEUM
MUSIC & REFRESHMENT HALL
PAVILION
GALLERY
ELECTRIC LAUNDRY
FRENCH LAUNDRY
Woodward St
Stevenson St
Woodward's Garden
Pond
MUSEUM
ART GALLERY
CONSERVATORY
Sea Lion Pond
Fish Pond
MARINE MUSEUM
Mission St
MISSION
161 Erie St,
HARMONY HALL
Valencia St
FOURTEENTH
71
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ROCKRIDGE
GEOTECHNICAL

Prepared for **Mx3 Ventures, LLC**

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
14TH & STEVENSON
San Francisco, California

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May 6, 2016
Project No. 15-1019

270 Grand Avenue
Oakland, CA 94610

www.rockridgegeo.com

510 420-5738 tel
510 652-3096 fax

ROCKRIDGE
GEOTECHNICAL

May 6, 2016
Project No. 15-1019

Mr. Manouch Moshayedi
Mx3 Ventures, LLC
2429 West Coast Highway, Ste.205
Newport Beach, California 92663

Proposed Mixed-Use Development
344 14th Street, 1463-1499 Stevenson Street, 86-98 Woodward Street
San Francisco, California

Dear Mr. Moshayedi,

The attached report, dated May 6, 2016, presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed mixed-use building to be constructed at 344 14th Street, 1463-1499 Stevenson Street, 86-98 Woodward Street in San Francisco. Our services were provided in accordance with our proposal dated December 1, 2015.

The project site is on the northeastern corner of the intersection of 14th and Stevenson streets and consists of two adjacent rectangular parcels that form an L-shaped project site with maximum plan dimensions of 130 by 237 feet. The site is currently used as a parking lot. Current plans are to construct a mixed-use building that will occupy most of the site. The building will have one level of below-grade parking. Above the garage will be a one-level concrete podium that will include retail spaces, as well as the lobby for the residences in the upper floors. Two to four stories of residential units will be constructed above the podium.

On the basis of the results of our geotechnical investigation, we conclude the proposed improvements can be constructed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and properly implemented during construction. We conclude a mat designed to resist hydrostatic uplift pressures supported on improved soil would be an appropriate foundation system for the proposed building. Alternatively, the proposed commercial building may be supported on a deep foundation system.

We appreciate the opportunity to provide our services to you on this project. If you have any questions, please call.

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.

C 84312
EXP. 9/30/17

GE2116
3/31/17

Tessa E. Williams, P.E.
Project Engineer

Craig S. Shields, P.E., G.E.
Principal Geotechnical Engineer

Enclosure

TABLE OF CONTENTS

LIST OF FIGURES

APPENDIX A

APPENDIX B

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
14TH & STEVENSON
San Francisco, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed mixed-use development to be constructed at 344 14th Street, 1463-1499 Stevenson Street, 86-98 Woodward Street in San Francisco, California. The site is located at the northeastern corner of the intersection of 14th and Stevenson streets, as shown on the attached Site Location Map (Figure 1).

The project consists of two adjacent rectangular parcels that form an L-shaped site with maximum plan dimensions of 130 by 237 feet. The site is currently used as a parking lot. Previous environmental borings by Rosso Environmental, Inc. indicate the site is blanketed by about 11 feet of sand fill with debris. Beneath the sand fill are native soils consisting of sand with layers of clayey silt. Groundwater was observed at depths between 11.2 and 12.5 feet in the environmental borings.

Current plans are to construct a mixed-use building that will occupy most of the site. The building will have one level of below-grade parking. Above the garage will be a one-level concrete podium that will include retail spaces, as well as the lobby for the residences in the upper floors. Two to four stories of residential units will be constructed above the podium.

2.0 SCOPE OF SERVICES

Our investigation was performed in accordance with our proposal dated December 1, 2015. Our geotechnical investigation included reviewing subsurface data from a previous geotechnical investigation within the site vicinity and exploring subsurface conditions at the site by drilling two borings and advancing five cone penetration tests (CPTs). We used the data collected during our field investigation to perform engineering analyses to develop conclusions and recommendations regarding:

- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- the most appropriate foundation type for the proposed structure
- design criteria for the recommended foundation type, including vertical and lateral capacities
- estimates of foundation settlement
- design groundwater elevation
- subgrade preparation for slab-on-grade floors and exterior flatwork
- site grading and excavation, including criteria for fill quality and compaction
- temporary slopes and shoring
- underpinning of adjacent structures, as appropriate
- 2013 San Francisco Building Code (SFBC) site class and design spectral response acceleration parameters
- soil corrosivity
- construction considerations, including dewatering.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

We investigated the subsurface conditions beneath the site by drilling two borings, designated as B-1 and B-2, and performing five CPTs, designated as CPT-1 through CPT-5. The approximate locations of the borings and CPTs are shown on the Site Plan (Figure 2). Prior to mobilizing to the site, we contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained a private utility locator to check for existing utilities at each boring and CPT location. We also obtained a drilling permit from San Francisco Department of Public Health (SFDPH).

3.1 Test Borings

Two borings were drilled on December 8, 2015, by Pitcher Drilling Company of East Palo Alto, California at the approximate locations shown on Figure 2. Borings B-1 and B-2 were drilled to depths of 61 and 51-1/2 feet bgs, respectively, using a truck-mounted drill rig equipped with rotary-wash drilling equipment. During drilling, our field geologist logged the soil encountered and obtained samples for visual classification and laboratory testing. Logs of the test borings are presented in Appendix A on Figures A-1a through A-2b. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure A-3.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter brass/stainless steel tubes.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners.
- Thin-walled Dames and Moore (D&M) tubes with a 2.5-inch outside and 2.43-inch inside diameter.

The S&H and SPT samplers were driven with a 140-pound, automatic hammer falling 30 inches per drop. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers were recorded every six inches and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The blow counts used for this conversion were: (1) the last two blow counts if the sampler was driven more than 12 inches, (2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and (3) the only blow count if the sampler was driven six inches or less. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.84 and 1.44, respectively, to account for sampler type and approximate hammer energy. The converted SPT N-values are presented on the boring logs.

Upon completion, the boreholes were backfilled with neat cement grout in accordance with SFDPH grouting guidelines. The soil cuttings generated by the borings were placed in 55-gallon drums and were disposed of offsite.

3.2 Cone Penetration Tests

Middle Earth Geo Testing, Inc. of Orange, California performed on December 18, 2015. The CPTs were advanced to refusal at depths ranging from 26 to 30-1/2 feet bgs. CPT-2 could not be advanced beyond a depth of one foot due to an obstruction. The CPTs were advanced by hydraulically pushing a 1.4-inch-diameter cone-tipped probe with a projected area of 10 square centimeters into the ground. The cone measured tip resistance, and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance, frictional resistance, and pore water pressure were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information such as the soil behavior types, approximate strength characteristics, and liquefaction potential of the soil encountered. Upon completion, the CPT holes were backfilled with cement grout in accordance with SFDPH requirements.

The CPT logs, showing tip resistance, friction ratio, and pore water pressure with depth, as well as interpreted soil behavior types, are presented in Appendix A on Figures A-4 through A-8.

4.0 SUBSURFACE CONDITIONS

The geologic map of the site and vicinity (Figure 3) indicates the project site is underlain by artificial fill (af). Our CPTs and borings indicate the project site is underlain by a relatively thick layer of undocumented fill generally consisting of loose to very dense sand and with varying gravel and fines content. The fill extends to a depth of approximately 11 to 12 feet bgs. The undocumented fill is underlain by medium dense to very dense sand with varying silt and clay content to a depth of approximately 47 feet bgs in boring B-1, located in the southern portion of the site, and to the maximum depth explored of 51-1/2 feet bgs in boring B-2 located in the northern portion of the site. In boring B-1, a soft to medium stiff clay layer was encountered beneath the sand layer and extends to the maximum depth explored of 61 feet bgs.

4.1 Groundwater Conditions

Groundwater was measured in our borings and CPTs at depths ranging from 12 to 21 feet bgs; however, the groundwater level may not have fully stabilized at the time the measurements were taken. The groundwater level at the site is expected to fluctuate several feet seasonally with potentially larger fluctuations annually, depending on the amount of rainfall. Considering the groundwater-level measurements in our borings and CPTs were taken after several years of severe drought, we judge that the readings likely represent the lower end of the spectrum.

During a previous investigation we performed within the site vicinity, the groundwater level was measured prior to grouting the CPTS at depths ranging from 11 to 17 feet in October 2009. Based on the existing groundwater level data discussed above in combination with historic groundwater data, we conclude a design high groundwater level of approximately 8 feet bgs should be used across the site.

5.0 SEISMIC CONSIDERATIONS

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards, including ground shaking, ground surface rupture, liquefaction,[1] lateral spreading,[2] and cyclic densification[3]. The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.1 Regional Seismicity and Faulting

The major active faults in the area are the San Andreas, San Gregorio, Hayward, and Calaveras faults. These and other faults of the region are shown on Figure 4. The fault systems in the Bay Area consist of several major right-lateral strike-slip faults that define the boundary zone between the Pacific and the North American tectonic plates. Numerous damaging earthquakes have occurred along these fault systems in recorded time. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated mean

---

[1] Liquefaction is a phenomenon where loose, saturated, cohesionless soil experiences temporary reduction in strength during cyclic loading such as that produced by earthquakes.

[2] 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.

[3] Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground-surface settlement.

characteristic moment magnitude[4] [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	**Approximate Distance from Site (km)**	**Direction from Site**	**Mean Characteristic Moment Magnitude**
N. San Andreas - Peninsula	10	West	7.23
N. San Andreas (1906 event)	10	West	8.05
N. San Andreas - North Coast	14	West	7.51
San Gregorio Connected	16	West	7.50
Total Hayward	19	Northeast	7.00
Total Hayward-Rodgers Creek	19	Northeast	7.33
Rodgers Creek	36	North	7.07
Mount Diablo Thrust	36	East	6.70
Total Calaveras	37	East	7.03
Monte Vista-Shannon	40	Southeast	6.50
Green Valley Connected	41	East	6.80
Point Reyes	41	West	6.90
West Napa	47	Northeast	6.70

In the past 200 years, four major earthquakes (i.e., Magnitude > 6) have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) Intensity Scale occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated moment magnitude, M_w, for this

---

[4] 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.

earthquake is about 6.25. In 1838, an earthquake occurred on the Peninsula segment of the San Andreas Fault. Severe shaking occurred with an MM of about VIII-IX, corresponding to an 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), an 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 October 17, 1989 with an M_w of 6.9. This earthquake occurred in the Santa Cruz Mountains about 94 kilometers southwest of 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 an 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 USGS's 2007 WGCEP has compiled the earthquake fault research for the San Francisco Bay area in order to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Region during the next 30 years is 63 percent. The highest probabilities are assigned to the Hayward/Rodgers Creek Fault and the northern segment of the San Andreas Fault. These probabilities are 31 and 21 percent, respectively (USGS, 2008).

5.2 Geologic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong 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, lateral spreading, and cyclic densification. We used the results of our borings and CPTs to evaluate the potential of these phenomena occurring at the project site.

5.2.1 Ground Shaking

The ground shaking intensity felt at the project site will depend on: 1) the size of the earthquake (magnitude), 2) the distance from the site to the fault source, 3) the directivity (focusing of earthquake energy along the fault in the direction of the rupture), and 4) site-specific soil conditions. The site is about 10 kilometers from the San Andreas Fault. Therefore, the potential exists for a large earthquake to induce strong to very strong ground shaking at the site during the life of the project.

5.2.2 Liquefaction and Liquefaction-Induced Settlement

When a saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by strong ground motion. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction. As shown on Figure 5, the site is within a liquefaction hazard zone, defined by the map titled *State of California, Seismic Hazard Zones, City and County of San Francisco, Official Map,* prepared by the California Geological Survey (CGS), dated November 17, 2000.

Liquefaction susceptibility was assessed using the software CLiq v1.7 (GeoLogismiki, 2014). CLiq uses measured field CPT data and assesses liquefaction potential, including post-earthquake vertical settlement, given a user-defined earthquake magnitude and peak ground acceleration (PGA). We performed a liquefaction triggering analysis using our CPT data in accordance with the methodology by Boulanger and Idriss (2014) and post-earthquake settlements by Zhang et al. (2002).

Our analysis was performed using a high groundwater depth of 8 feet bgs. In accordance with the 2013 SFBC, we used a peak ground acceleration of 0.58 times gravity (g) in our liquefaction evaluation; this peak ground acceleration is consistent with the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M).

We also used a Moment magnitude 8.05 earthquake, which is consistent with the mean characteristic Moment magnitude for the San Andreas Fault 1906 event, as presented in Table 1.

Our liquefaction analyses indicate there are soil layers between depths of approximately 8 and 28 feet bgs that are susceptible to liquefaction during a major earthquake. Based on the results of our analyses, we estimate total settlement associated with liquefaction after an MCE event generating a PGA_M of 0. g will be on the order of 3 inches and liquefaction-induced differential settlement will be approximately 2 inches over horizontal distance of 30 feet, respectively. Because the uppermost potentially liquefiable layers are at or near the proposed finished floor elevation, there is potential for significant reductions in bearing capacity if the proposed building is supported on a shallow foundation system founded on unimproved soil. Consequently, the actual building settlement could be significantly greater than that estimated above for the free-field ground surface during an earthquake. As discussed in later sections of this report, the potential for liquefaction within these relatively shallow layers should be mitigated if the building is to be supported on a shallow foundation system.

If the soil beneath the proposed foundation elevation is improved to mitigate liquefaction within these layers, we estimate that total building settlement associated with liquefaction of the remaining layers will be less than one inch and liquefaction-induced differential settlement will be about 1/2 inch over a horizontal distance of 30 feet.

We evaluated the potential for lateral spreading to occur at the site using an empirical relationship developed by Youd, Hansen, and Bartlett (1999). The method incorporates the thickness of the liquefiable layer, the fines content and mean grain-size diameter of the liquefiable soil, the relative density of the liquefiable soil, the magnitude and distance of the earthquake from the site, the slope of the ground, and boundary conditions (i.e. proximity to a free face), to estimate the horizontal ground movement due to lateral spreading. The results of our analysis indicate the liquefiable layers have sufficient relative density such that the potential for lateral spreading to occur at the site to be very low. Our review of published data also revealed no documented occurrence of lateral spreading in the area during the 1989 Loma Prieta Earthquake. Therefore, we conclude the potential for lateral spreading to occur at the site is low.

5.2.3 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The site is underlain by loose to medium dense sandy fill above the groundwater table that is susceptible to cyclic densification.

The proposed building will have one level of below-grade parking. The loose to medium dense sandy fill will be removed when constructing the below-grade parking level. Therefore, the effects of cyclic densification of the loose sand should only occur within the surrounding improvements. Following a Maximum Considered Earthquake (MCE) event with a peak ground acceleration (PGA) of 0.58 times gravity (g), we estimate ground-surface settlements on the order of 1/2 inch could occur due to cyclic densification of the loose sand outside of the basement footprint.

5.2.4 Ground Surface Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. We therefore conclude the risk of fault offset at the site from a known active fault is very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is also very low.

6.0 DISCUSSION AND CONCLUSIONS

Based on the results of our engineering analyses using the data from the test borings and CPTs within the site vicinity, we conclude the site may be developed as proposed provided the geotechnical issues discussed below are properly addressed. The primary geotechnical issues affecting design and construction of the proposed building include: 1) shallow groundwater relative to the proposed building foundation and excavation depth, 2) the presence of potentially liquefiable soil layers that extend about 16 to 18 feet below the proposed top of basement slab elevation, which could result in reduced bearing capacity and excessive settlement under seismic conditions if not mitigated, and 3) providing suitable lateral support and dewatering for the proposed excavation, while minimizing impacts to the surrounding improvements. These issues are discussed in more detail below.

6.1 Groundwater

Based on the available groundwater data discussed in Section 4.1, we recommend using a design high groundwater level of 8 feet below existing sidewalk grade for the proposed project. As discussed in Section 1.0, we understand the proposed development will include one level of below-grade parking. Current drawings indicate the lower garage top-of-slab elevation is at approximately 12 feet bgs. We estimate the construction of the proposed building will require an excavation bottomed up to about 14 feet bgs, assuming a preliminary mat foundation thickness of about 24 inches. Therefore, the bottom- -foundation may be up to about 6 feet below the design high groundwater level. As a result, the proposed building foundation and below-grade walls will need to be designed to resist hydrostatic pressures and include waterproofing.

Considering the proposed excavation will extend below the groundwater, the excavation will need to be temporarily dewatered and the excavation shoring system will need to be designed for the effects of groundwater. A more detailed discussion regarding temporary excavation shoring and dewatering is presented in Section 6.3.

6.2 Foundations Support

The proposed building will have one level of below-grade parking that will require an excavation of about 14 feet bgs. The basement will be underlain by interbedded layers of medium dense to dense sand that extends to depths ranging from approximately 23 to 28 feet bgs. Shallow foundations, such as spread footings or a mat, bearing on these soil deposits will experience: (1) erratic and excessive settlement caused by post-liquefaction settlement of the underlying soils; and (2) reduction of bearing due to liquefaction of the supporting soil. Therefore, we conclude the liquefaction potential of the soil immediately below the foundation level will need to be mitigated for shallow foundations to be feasible.

We conclude a mat designed to resist hydrostatic uplift pressures supported on improved soil would be an appropriate foundation system for the proposed building, provided: (1) the soil improvement is implemented to mitigate the potential for bearing capacity failure under seismic conditions, and (2) the soil improvement extends to a depth that would reduce differential settlement of the structure under seismic conditions to a tolerable amount. Based on our recent experience, we believe either compaction grouting or drilled displacement sand-cement (DDSC) columns would be the most economical ground improvement method; however, other soil improvement methods, such as soil-cement (SMX) columns, are also feasible. If soil anchors are required to resist hydrostatic uplift pressures, the DDSC columns may be designed to accommodate reinforcing steel in lieu of tiedowns or micropiles.

Compaction grouting consists of driving a small-diameter pipe into the soil to be improved and injecting a low-slump, mortar-like grout under pressure. The grout displaces the soil forces it into a denser mass. The grout does not penetrate the voids but expands under pressure to form a bulb up to two feet in diameter. Compaction grouting is generally performed on a grid pattern with injection points spaced approximately 4 to 8 feet on center.

DDSC columns are installed by advancing a continuous flight, hollow-stem auger that mostly displaces the soil and then pumping a sand-cement mixture into the hole under pressure as the auger is withdrawn. This system results in low vibrations during installation and generates little to no drilling spoils for off-haul. DDSC columns are installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of columns should be determined by the contractor, based on the desired level of improvement. The replacement ratio for ground improvement should be selected to mitigate liquefaction. We anticipate the DDSC spacing would be on the order of seven feet on center. The lengths of the DDSCs would range from about 30 to 35 feet. We recommend a preliminary design, including calculations of static and seismic settlement, be prepared by the ground improvement contractor and submitted for our review.

Our settlement analyses indicate total settlement of a mat foundation bearing on improved ground designed using the allowable bearing pressures presented in Section 7.2 of this report will be on the order of one inch and differential settlement will be on the order of 3/4 inch over a 30-foot horizontal distance. We anticipate approximately two-thirds of this settlement will occur during construction, with the remainder occurring within a few years after construction is complete.

6.3 Construction Considerations

6.3.1 Excavation Support

Temporary shoring will be required to laterally restrain the sides of the excavation for the proposed basement. All excavations that will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). The shoring engineer should be responsible for shoring design. The contractor should be responsible for the construction and safety of temporary slopes.

We anticipate an excavation extending up to about 14 feet bgs will be needed to construct the below-grade parking garage. We judge that a cantilevered soldier pile and timber lagging shoring system is appropriate for support of excavations up to about 12 feet in depth. One row of tiebacks may be used to reduce the soldier pile size and embedment depth for deeper excavations. A soldier pile-and-lagging system usually consists of steel H-beams and concrete placed in predrilled holes extending below the bottom of the excavation. If it is not feasible to install the cantilevered soldier piles on the adjacent properties, the basement wall should be offset from the property line by about 12 to 18 inches to provide space for the shoring. Wood lagging is placed between the piles as the excavation proceeds from the top down.

Where granular soil layers are encountered below the groundwater, installing the soldier piles will likely require casing or use of drilling slurry to reduce caving of the holes. If drilling slurry is used, or groundwat
Installation of soldier piles by vibration would be feasible where the soldier piles are at least 25 feet from existing buildings.

Relatively loose, fine-grained, and/or saturated sandy soil is present within the proposed excavation; this soil is highly susceptible to caving and piping through lagging boards. Therefore, we conclude that excavations should extend no more than 12 inches below the last row of lagging. Furthermore, dewatering prior to excavating will be critical for this project. Where voids are developed behind wood laggings, the voids should be promptly filled by hand-packing dry material and/or filling the voids with flowable sand-cement slurry mix.

A structural/civil engineer knowledgeable in this type of construction should be retained to design the shoring. The shoring designer should design the shoring system for lateral deformation of less than 1/2 inch at any location on the shoring where there is a structure within a horizontal distance equal to twice the retained soil height and one inch where there are no structures within that horizontal distance. We should review the final shoring plans and calculations to check that they are consistent with the recommendations presented in this report.

6.3.2 Foundation Underpinning

Underpinning of the existing buildings along the northern and eastern property lines will be required to construct the proposed building. To design an underpinning system, it will be necessary to determine the configuration and depth of the existing foundations. If as-built plans cannot be obtained, test pits should be excavated prior to construction to determine the foundation type and depth to complete the design of an appropriate underpinning system.

We judge conventional hand-excavated end-bearing piers would be an appropriate underpinning system for this project. Hand-excavated, end bearing piers are generally installed by excavating three-foot by five-foot rectangular shafts down to a bearing layer. The shafts are constructed with reinforcing steel and backfilled with structural concrete. The shafts are constructed in phases, in order to maintain support for the existing foundations. Each shaft is shored with timber as it is excavated. Due to the presence of loose and/or saturated sand, we judge that hand-excavated piers should be thoroughly dewatered and shored with every foot of excavation.

Where underpinning will extend relatively deep, it may be more economical to use slant drilled cast-in-place soldier piles (referred to as “slant piles”).

As previously discussed, about 1/2 inch of ground surface settlement as a result of cyclic densification and about 3 inches of liquefaction-induced ground surface settlement is expected to occur surrounding the project site during a major earthquake. Existing buildings being underpinned may experience differential settlement between the existing foundation and the underpinned foundation, depending on the type and depth of the existing foundations. The magnitude of differential settlement will depend on the existing foundation configuration and the depth of underpinning piers. For underpinning piers bottomed about 14 to 16 bgs, seismically-induced differential settlement between existing and underpinned shallow foundations would be on the order of 1-1/2 to 2 inches. The project structural engineer should assess if this would be problematic.

Underpinning piers will extend beneath the neighboring properties, which will require an encroachment agreement with neighboring property owners. If it is not feasible to install the underpinning piers beneath the adjacent property, the basement wall should be offset from the property line by 12 to 18 inches to provide space for the shoring and the shoring should be designed to resist surcharge loads from neighboring foundations. Special precautions will be needed to prevent undermining of the neighboring foundations during installation of the shoring. These precautions may include soil mixing or permeation grouting.

6.3.3 Excavation Dewatering

The design groundwater level is above the bottom of the proposed excavation. During excavation of the basement, groundwater will flow into the excavation unless collected and removed prior to reaching the work area. Therefore, a temporary dewatering system should be installed to provide a firm, relatively dry base from which to construct the foundation system. We anticipate an active dewatering system will need to be installed prior to the start of excavation, including the excavation for underpinning piers. Localized passive dewatering, in which water is collected from trench drains around the perimeter and across the base of the excavation, may also be required. The method used to dewater the excavation should be the responsibility of the contractor. The dewatering system should be designed to drawdown the groundwater at least three feet below the bottom of the planned excavation and maintain that depth until a sufficient amount of the concrete structure is in place, as determined by the project structural engineer.

The construction dewatering system must be capable of maintaining the groundwater level below the foundation subgrade until sufficient building weight is available to resist the hydrostatic uplift pressure, at which time the groundwater may be allowed to rise to its normal elevation. The project structural engineer should determine when the temporary dewatering system can be turned off.

6.3.4 Construction Monitoring

Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. During excavation, the shoring system is expected to yield and deform laterally, which could cause the ground surface adjacent to the shoring wall to settle. The magnitudes of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Ground movements due to a properly designed and constructed shoring system should be within ordinary accepted limits of about one inch. A monitoring program should be established to evaluate the effects of the construction on the adjacent properties.

The conditions of existing buildings within 25 feet of the site should be photographed and surveyed prior to the start of construction and monitored periodically during construction. In addition, prior to the start of excavation, the contractor should establish survey points on the shoring system, on the ground surface at critical locations behind the shoring, and on adjacent buildings. These survey points should be used to monitor the vertical and horizontal movements of the shoring and the ground behind the shoring throughout construction.

The survey points should be monitored regularly and the results should be submitted to us and the shoring engineer in a timely manner for review. For estimating purposes, assume that the instrumentation will be read as follows:

- Prior to any excavation or shoring work at the site
- After installing soldier piles
- After excavation of each lift
- After the excavation reaches its lowest elevation
- Every two weeks until the street-level floor slab is constructed

6.4 Soil Corrosivity

Corrosivity testing was performed by Sunland Analytical of Rancho Cordova, California on a sample of soil obtained during our field investigation from Boring B-1 at a depth of three feet bgs. The results of the test are presented in Appendix B of this report. Based on the resistivity test results, the sample would be classified as “corrosive” to buried steel. Accordingly, buried iron, steel, cast iron, galvanized steel, and dielectric-coated steel or iron should be properly protected against corrosion. The chloride, and sulfate ion concentrati
not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures; however, the soil tested positive for sulfides.

7.0 RECOMMENDATIONS

Recommendations for site grading, temporary shoring, basement wall and foundation design, ground improvement, and seismic design are presented in this section of the report.

7.1 Site Preparation and Grading

Site demolition should include the removal of existing pavements and all existing underground utilities and foundations, if any. In general, abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Voids resulting from demolition activities should be properly backfilled with compacted fill following the recommendations provided later in this section. Demolished asphalt concrete should be taken to an asphalt recycling facility.

Excavations should be backfilled with properly compacted fill. Fill should consist of on-site soil or imported soil (select fill) that is free of organic matter and debris, contains no rocks or lumps larger than four inches in greatest dimension, has a liquid limit of less than 40 and a plasticity index lower than 12, and is approved by the Geotechnical Engineer. Samples of proposed imported fill material should be submitted to the Geotechnical Engineer at least three business days prior to use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed imported material.

Fill should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction. Fill consisting of clean sand or gravel (defined as soil with less than 10 percent fines by weight) should be compacted to at least 95 percent relative compaction. Fill greater than five feet in thickness or placed within the upper foot of vehicular pavement soil subgrade should also be compacted to at least 95 percent relative compaction, and be non-yielding.

During excavation for the basement level, the excavation will likely extend below groundwater. The foundation excavation subgrade will consist of saturated sand with varying fines content or clay with varying sand content, which will be sensitive to disturbance, especially under construction equipment wheel loads. Therefore, the subgrade should be compacted with a smooth-drum roller to densify disturbed soil after reaching the design excavation depth. If soft silty or clayey soil (marsh deposit) is encountered at subgrade elevation, it should removed and replaced with excavated on-site sandy soil. A mud slab is generally required beneath most waterproofing products and, in some cases, is required both above and below the waterproofing membrane.

7.1.1 Exterior Flatwork Subgrade Preparation

Exterior concrete flatwork that will not receive vehicular traffic (i.e. sidewalk) should be underlain by at least four inches of Class 2 aggregate base compacted to at least 90 percent relative compaction. Prior to placement of the aggregate base, the upper eight inches of the subgrade soil should be scarified, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction.

7.1.2 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of clean sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped. Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted in accordance with the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

7.2 Foundations

The proposed building should be supported on a reinforced-concrete mat foundation underlain by improved soil. The mat should be underlain by waterproofing and designed to resist hydrostatic uplift pressures. If the building weight is not sufficient to resist the hydrostatic uplift pressures imposed by the groundwater, soil anchors (i.e., tiedowns) may be required to provide the mat foundation with additional uplift resistance. The following sections present our recommendations for the design and construction of a mat foundation bearing on improved soil. If it is determined that the building weight is not sufficient to resist the hydrostatic pressures, we can provide recommendations for tiedowns upon request. We can also provide recommendations for other ground improvement methods upon request.

7.2.1 Mat Foundation on Ground Improved with DDSC Columns

For preliminary design of a mat foundation bearing on improved ground, we recommend assuming ground improvement elements will extend about five feet into the dense to very dense sand beneath the potentially liquefiable material. The top of the dense sand generally slopes down to the south, ranging from about 23 to 28 feet grades. Based on discussions with contractors with experience installing DDSC columns in the Bay Area, we anticipate the ground improvement systems described in later in this section, if properly designed, should be capable of increasing the allowable bearing pressure to approximately 3,000 to 4,000 pounds per square foot (psf) for dead-plus-live-load conditions, while limiting combined static and seismic differential settlement to less than about one inch over a horizontal distance of 30 feet. The actual design allowable bearing pressures and estimated settlement should be evaluated by the design-build ground improvement contractor, as they will be based on the diameter, depth, and spacing of the ground improvement elements.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the mat and friction between the bottoms of the mat and the supporting soil. To compute lateral resistance for sustained loading conditions, we recommend using equivalent fluid weights (triangular distribution) of 250 and 120 pcf above and below the design groundwater level, respectively. The upper foot of soil should be ignored unless confined by a slab or pavement. The recommended passive pressure includes a factor of safety of at least 1.5. Allowable frictional resistance along the base of the mat should be calculated based on parameters provided by the design-build ground improvement contractor.

The mat subgrade will be sensitive to disturbance due to its proximity to the groundwater table. The final two feet of excavation and fine grading of the mat subgrade should be performed with tracked equipment to minimize heavy concentrated loads that may disturb the wet soil. Rubber-tired equipment and dump trucks should not be operated on the final mat subgrade. The subgrade should be free of standing water, debris, and disturbed materials and be approved by the geotechnical engineer prior to placing the waterproofing and steel

If an internal excavation dewatering system is needed to continuously maintain the water level below the bottom of the mat until the building has sufficient weight to resist hydrostatic uplift pressures associated with the design water level, the mat will need to be constructed with temporary block-outs to accommodate the extraction wells or sump pits used to extract the water from the drainage layer. Once it has been determined by the structural engineer that the dewatering system may be shutoff, the pumps will need to be removed and the block-outs promptly waterproofed and plugged. The detailing of the waterproofing and plugging system at these locations will be critical and should be evaluated by a waterproofing consultant and structural engineer experienced with such operations.

7.2.2 Mat Foundation on Ground Improved with Compaction Grouting

As an alternative to ground improved by DDSC columns, the proposed building may be supported on a mat foundation bearing on soil improved by compaction grouting. The top of the mat foundation may be used as the basement floor or a thin layer of concrete (topping slab) may be placed above the mat to provide a smooth wearing surface.

For design of the mat, we recommend using a modulus of vertical subgrade reaction of 20 pounds per cubic inch (pci); this value has been reduced to account for the size of the mat. To check the behavior of the mat under total load conditions, a modulus of vertical subgrade reaction of 25 pci should be used. Once the structural engineer estimates the distribution of bearing stress on the bottom of the mat, we should review the distribution and revise the modulus of subgrade reaction, if appropriate. We recommend the mat be designed for allowable bearing pressures of 3,000 psf for dead-plus-live loads and 4,000 psf for total loads (including seismic and wind loads); we anticipate the average bearing pressure will be significantly lower. Localized higher bearing pressures may be acceptable; however, this should be reviewed on a case-by-case basis.

The mat should be designed to resist hydrostatic uplift using the design groundwater elevation discussed previously in this report. Lateral forces can be resisted by friction along the base of the mat and passive pressure against the sides of the mat foundation and the basement walls. To compute lateral resistance for sustained loading conditions, we recommend using equivalent fluid weights (triangular distribution) of 250 and 120 pcf above and below the design groundwater level, respectively. The upper foot of soil should be ignored unless confined by a slab or pavement. The allowable friction factor will depend on the type of waterproofing used at the base of the mat. For bentonite-based waterproofing membranes, such as Paraseal or Voltex, a friction factor of 0.12 should be used (assumes a bentonite friction angle of 10 degrees). If Preprufe is used, a base friction factor of 0.20 should be used. Friction factors for other types of waterproofing membranes can be provided upon request.

Ground Improvement with Compaction Grouting

We recommend the sand and silty sand between the bottom of the proposed mat foundation and the top of the dense sand at depths of 23 to 28 feet bgs be improved to mitigate its liquefaction potential. Based on our experience with similar soil conditions, we recommend a grout point spacing (rectangular) of six feet be used. The entire footprint of the proposed building should be treated. From a practical standpoint, however, the outermost row of the grout points should be located four feet from the property line. The grout points closest to the site property line should be grouted first and the grouting should proceed inward toward the middle of the site to reduce the potential for heave of adjacent structures. The compaction grouting should be performed prior to any excavation to maximize the overburden pressure at the grouting depths.

Based on our experience using compaction grouting to improve granular soil, we believe the grout pumping rate, grout slump, and the characteristics of the fine-grained material (passing the No. 200 sieve) in the grout are the most important factors influencing the effectiveness of the procedure. We recommend the pumping rate not exceed two cubic feet per minute (cfm) during grout injection. We recommend a maximum grout slump of two inches be allowed; the slump should be measured at the point of injection rather than at the mixer. In addition, the fine-grained material in the grout mix should consist primarily of silt. The clay content (percent passing No. 200 sieve equal to or smaller than 0.002 millimeters) should be no greater than three percent. The grouting subcontractor should verify the soil source used for compaction grouting meets the clay content requirement. If the subcontractor does not have this information, we should be provided with a sample of the source soil at least one week prior to use in the test section to run a hydrometer analysis.

Prior to the start of production grouting, we should perform two CPTs to check the effectiveness of the contractor's grouting procedure on a grout test section. The post-grout $(qc_{1N})_{CS}$ for the soil to be improved should average at least 150 tons per square foot (tsf) and the computed liquefaction-induced settlement using the CPT data should be less than one inch using the CLiq program and the methodologies by Boulanger and Idriss (2014) and Zhang et al. (2002). We should also verify the grout pumping rate and slump are acceptable during test grouting by pumping grout into a box with known dimensions for a given amount of time to measure the rate and measuring the grout slump immediately prior to injection. If the improvement observed after completion of the test section is satisfactory, additional verification testing (CPTs) should be performed during and at the completion of grouting to verify the desired improvement has been obtained. Pumping rate and slump measurements should be taken regularly during production grouting to verify the consistency of the grout throughout the project.

In our experience, special care must be taken when compaction grouting is performed near existing improvements. We recommend the adjacent buildings and the street and sidewalk adjacent to the site be surveyed daily to check for upward and lateral movement. If vertical or lateral movement greater than 1/4 inch is measured, we should be consulted to review the grout injection plan and volume and make modifications to protect the adjacent improvements, if necessary.

7.3 Permanent Below-Grade Walls

Below-grade walls should be designed to resist static lateral earth pressures, lateral pressures caused by earthquakes, vehicular surcharge pressures, and surcharges from adjacent foundations, where appropriate. We recommend below-grade walls at the site be designed for the more critical of the following criteria:

- At-rest equivalent fluid weight of 55 pcf above the design groundwater table and 86 pcf below the design groundwater table.
- Active pressure of 35 pcf plus a seismic increment of 25 pcf (triangular distribution) above the design groundwater level, and 77 pcf plus a seismic increment of 11 pcf (triangular distribution) below the groundwater level for seismic conditions.

The recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads. Where the below-grade walls are subject to traffic loading within 10 feet of the wall, an additional uniform lateral pressure of 100 psf, applied to the upper 10 feet of the wall, should be used.

To protect against moisture migration, below-grade walls should be waterproofed and water stops should be placed at all construction joints. The design pressures recommended for above the design water level are based on fully drained walls. Although part of the basement walls will be above the groundwater level, water can accumulate behind the walls from other sources, such as rainfall, irrigation, and broken water lines, etc. One acceptable method for backdraining a basement wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to the design groundwater level. Since the soil below the design groundwater level has a relatively high permeability, any water collected in the drainage panels should dissipate into the soil. Therefore, it is not necessary to install a collection pipe at the base of the drainage panels.

If backfill is required behind basement walls prior to pouring the floor slabs, the walls should be braced, or hand compaction equipment used, to prevent unacceptable surcharges on walls (as determined by the structural engineer).

7.4 Underpinning

Provided the seismically induced differential settlement between existing and underpinned shallow foundations presented in Section 6.3.2 is acceptable, hand-excavated piers may be used to underpin adjacent foundations. Where hand-excavated underpinning piers are used to underpin adjacent foundations, the piers should be designed to gain support through end bearing on medium dense to dense native sand. An allowable bearing pressure of 2,000 psf for dead-plus-live loads may be used for design of underpinning piers. The underpinning piers should extend at least 24 inches below the planned excavations for the project or 24 inches below an imaginary line that lies at 45 degrees from horizontal, projected upward from the bottom edge of the proposed excavation. The width of the underpinning piers should be determined by the structural engineer or underpinning designer based on the ability of the existing foundation to span an area of non-support. Underpinning should be designed for unbalanced horizontal loads resulting from the soil retained by the piers. The unbalanced load should be computed using an at-rest equivalent fluid weight of 55 pcf.

7.5 Temporary Shoring

As discussed previously, we judge the most economical shoring methods for the proposed excavation consist of cantilevered soldier piles with lagging where the excavation is less than approximately 12 feet deep and soldier pile and lagging with one row of tiebacks where the excavation is more than 12 feet deep. Recommendations for design of other types of shoring systems can be provided upon request.

7.5.1 Cantilevered Soldier Piles and Lagging

For design of a cantilevered soldier pile and lagging system, we recommend using an active equivalent fluid weight of 35 pcf where the excavation will be adjacent to public sidewalks and where there will be no structures within a horizontal distance equal to twice the proposed excavation depth. Where the adjacent structures are within a horizontal distance equal to twice the proposed excavation depth, the shoring should be designed using an at-rest equivalent fluid weight of 55 pcf plus the building surcharge load.

The above pressures should be assumed to act over the entire width of the lagging installed above the base of the excavation. The active pressure need only be assumed to act over one pile width below the bottom of the excavation. This value assumes perched groundwater, if present, seeps through the lagging and does not impose a lateral pressure on the shoring. Passive resistance at the toe of the soldier pile should be computed using equivalent fluid weights of 250 and 125 pcf above and below the drawn-down groundwater table, respectively. For design of shoring, it should be assumed the groundwater table has been lowered by dewatering to three feet below the mat subgrade. Passive pressure can be assumed to act over an area of three soldier pile widths assuming the toe of the soldier pile is filled with structural concrete. If lean concrete is placed in the soldier pile shaft, the passive pressure can be assumed to act over two pile diameters. These passive pressure values include a factor of safety of at least 1.5.

7.5.2 Soldier Piles and Lagging with Tiebacks

Recommended lateral pressures for the design of soldier beam and lagging shoring with tiebacks are presented on Figure 6. In calculating these design pressures, we assume drained conditions with no hydrostatic pressure acting on the shoring.

The penetration of the soldier piles must be sufficient to ensure stability and resist the downward loading of tiebacks. For computing lateral resistance below the bottom of the excavation, we recommend using equivalent fluid weights of 250 and 125 pcf above and below the drawn-down groundwater table, respectively. Passive pressure can be assumed to act over an area of three soldier pile widths assuming the toe of the soldier pile is filled with structural concrete. If lean concrete is placed in the soldier pile shaft, the passive pressure can be assumed to act over two pile diameters. These passive pressure values include a factor of safety of at least 1.5. The factor of safety applied to the allowable passive pressure value may be adjusted by the shoring designer, depending upon the design requirements.

Vertical loads can be resisted by skin friction along the portion of the soldier piles below the excavation. An allowable skin friction of 600 psf may be used to compute the vertical capacities of soldier piles.

7.5.3 Tieback Design and Testing

Design criteria for tiebacks are also presented on Figure 6. As shown, tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation at angle 60 degrees from horizontal, where H is the wall height in feet. The minimum stressing and bond lengths should both be 15 feet.

Tiebacks will generally be installed in loose to medium dense sand. Allowable capacities of the tiebacks will depend upon the drilling method, hole diameter, grout pressure, and workmanship. Because of the tendency of sand to cave, solid- or hollow-stem augers should not be used in these materials. We recommend a smooth-cased method (such as a Klemm rig) be used to install tiebacks in the sand layers. For estimating purposes, we recommend using the skin friction values for pressure-grouted tiebacks given on Figure 6.

The shoring designer should be responsible for determining the actual length of tieback required. The determination should be based on the designer’s familiarity with the installation method to be used. The computed bond length should be confirmed by a performance- and proof-testing program under the observation of an engineer experienced in this type of work. The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to 1.5 times the design load. The remaining tiebacks should be confirmed by a proof-test to 1.25 times the design load. The bottom of the excavation should not extend more then two feet below a row of unsecured tiebacks.

The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during proof and performance testing. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 1/2, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is more than 0.04 inches, the load should be held for an additional 50 minutes. If the deflection is more than 0.08 inches between the 6- and 60-minute readings, the tieback design loading should be re-evaluated. If any tieback fails to meet the performance- and proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as directed by the shoring designer. After testing, the tiebacks should be loaded to the design load (less if specified by the shoring designer) and locked off.

The shoring should be designed by a qualified engineer experienced in shoring design. We should review the shoring design prior to construction.

7.6 Seismic Design

As discussed in Section 5.2.2, the site is underlain by potentially liquefiable soil layers; however, if the potential settlement due to liquefaction is mitigated using ground improvement as described in Section 7.1.2, we do not expect significant non-linear soil behavior to occur. Consequently, we conclude a Site Class D can be used for the building design. The latitude and longitude of the site are 37.7681° and -122.4214°, respectively. Hence, in accordance with the 2013 SFBC, we preliminarily recommend the following:

- S_S = 1.501 g, S_1 = 0.657 g
- S_{MS} = 1.501 g, S_{M1} = 0.985 g
- S_{DS} = 1.000 g, S_{D1} = 0.657 g
- PGA_M = 0.581g
- Seismic Design Category D for Risk Categories I, II, and III.

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Prior to construction, Rockridge Geotechnical should review the project plans and specifications to verify that they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during shoring and underpinning installation, excavation, placement and compaction of fill, ground improvement, and installation of foundations. These observations will allow us to compare actual with anticipated soil conditions and to verify that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

9.0 LIMITATIONS

This geotechnical investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the exploratory borings and CPTs performed for this investigation. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

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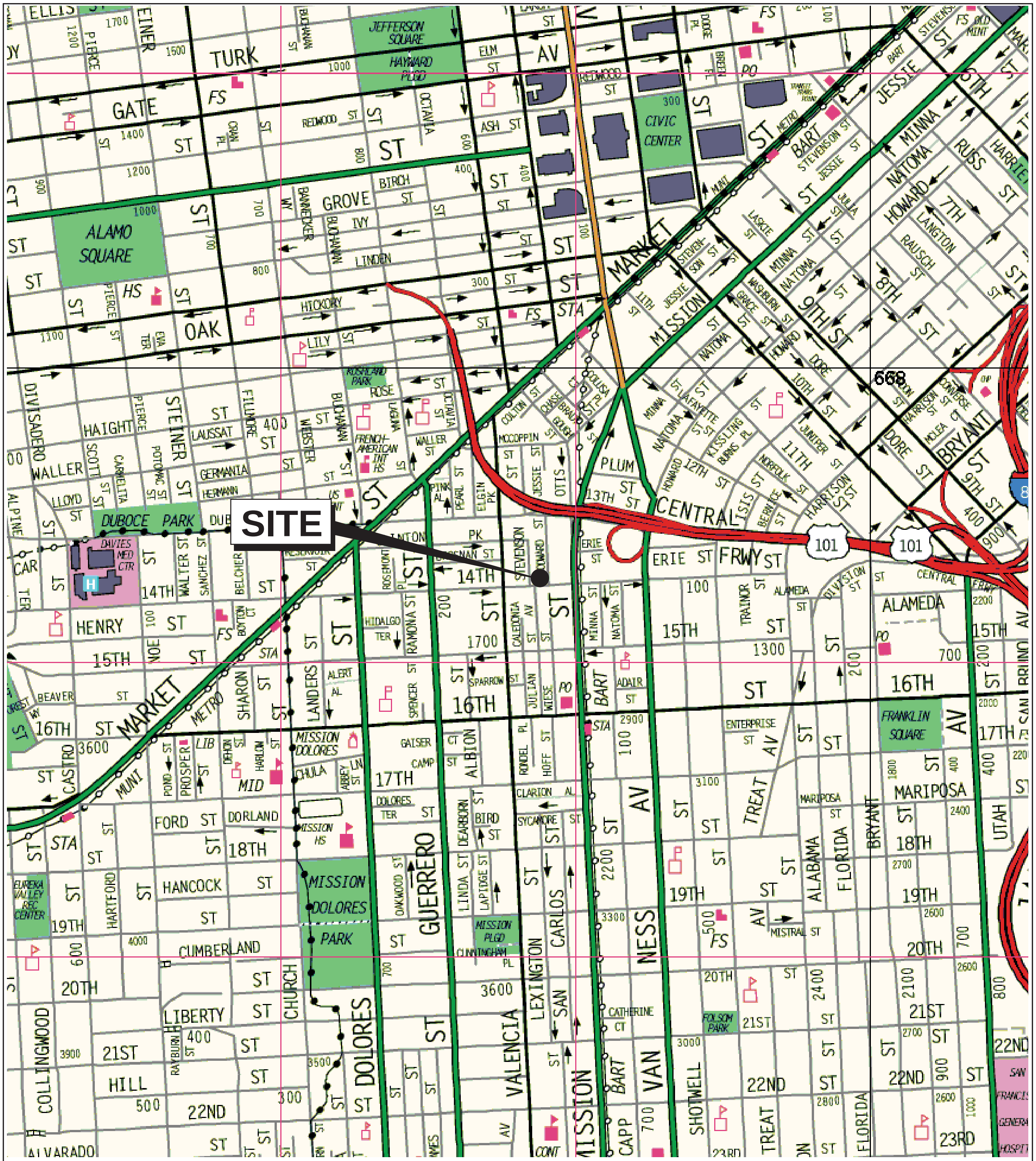
FIGURES



Base map: The Thomas Guide
San Francisco County
2002

0 1/4 1/2 Mile

Approximate scale

N

14TH & STEVENSON
San Francisco, California

ROCKRIDGE
GEOTECHNICAL

SITE LOCATION MAP

Date 12/18/15	Project No. 15-1019	Figure 1

EXPLANATION

CPT-1 ▲	Approximate location of cone penetration test by Rockridge Geotechnical, Inc., December 18, 2015
B-1 ⊕	Approximate location of boring by Rockridge Geotechnical, Inc., December 8, 2015

0 60 Feet

Approximate scale

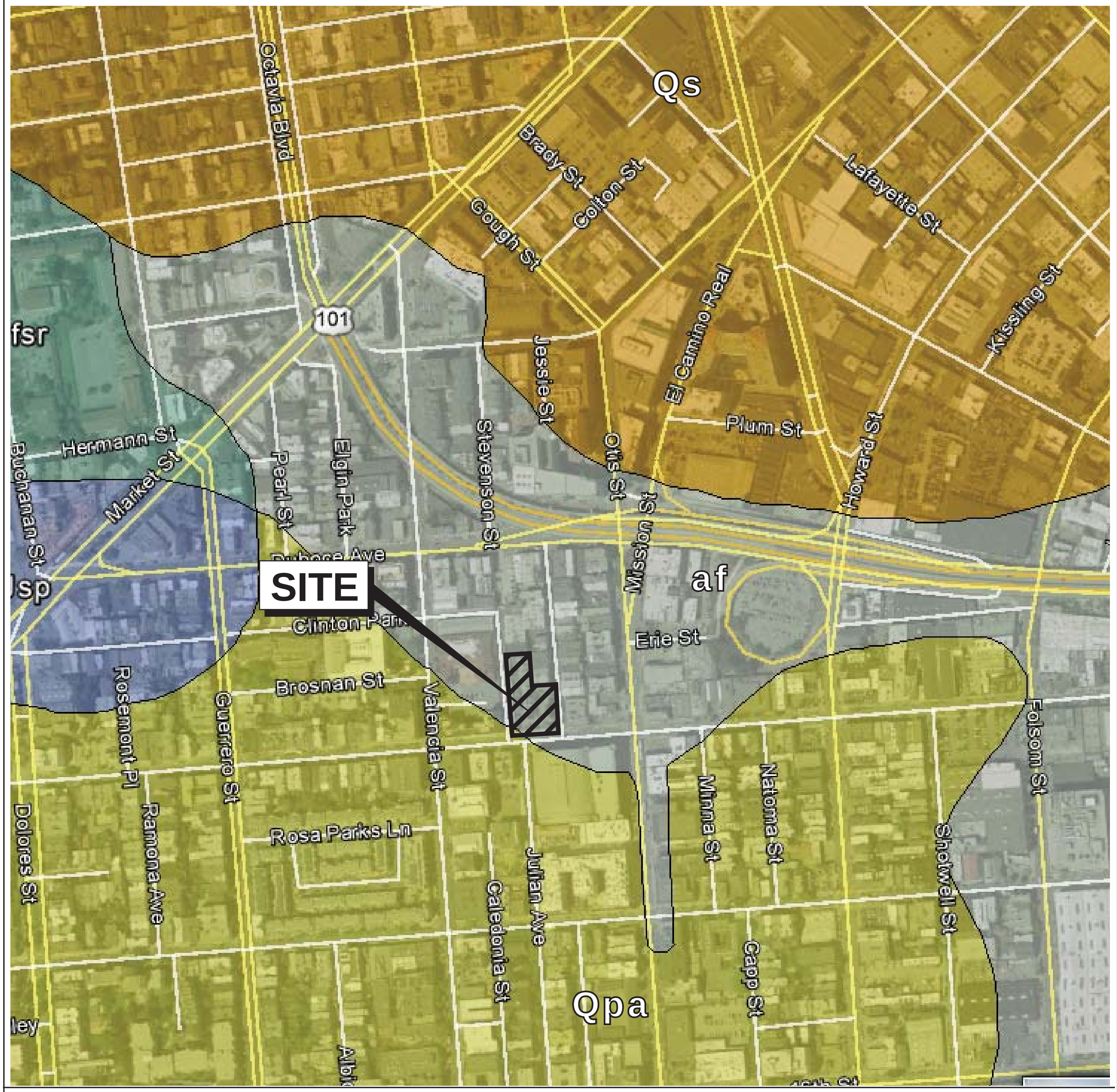
14TH & STEVENSON
San Francisco, California

SITE PLAN

Date 01/13/16 | Project No. 15-1019 | Figure 2

ROCKRIDGE GEOTECHNICAL

Reference: Base map from drawing titled "Existing Site Context - 150 Foot Radius", by BAR Architects, dated May 11, 2015.

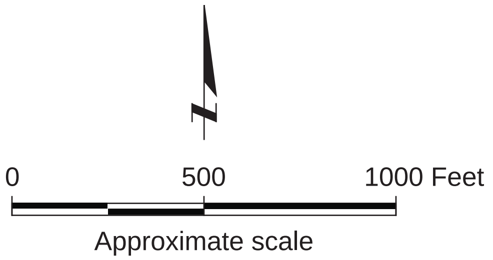


Base map: Goolge Earth with U.S. Geological Survey (USGS), San Francisco County, 2015.

EXPLANATION

af	Artificial Fill
Qs	Beach and dune sand (Quaternary)
Qpa	Alluvium (Pleistocene)
fsr	Franciscan Complex melange (Eocene, Paleocene, and (or) Late Cretaceous)
Jsp	Great Valley Complex Serpentinite (Jurassic)

Geologic contact: dashed where approximate and dotted where concealed, queried where uncertin

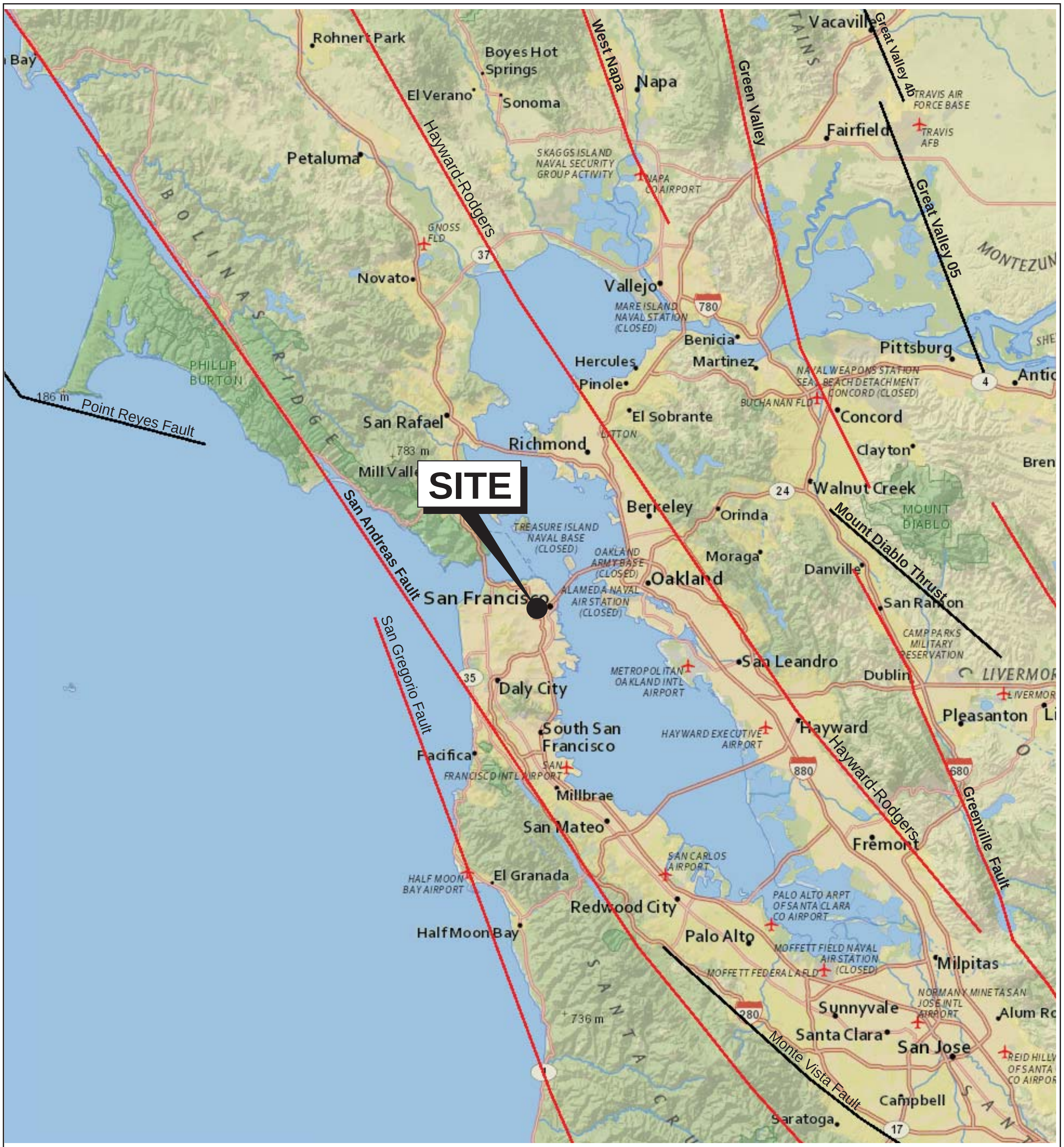


14TH & STEVENSON
San Francisco, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 01/13/16	Project No. 15-1019	Figure 3



Base Map: U.S. Geological Survey (USGS), National Seismic Hazards Maps - Fault Sources, 2014.

EXPLANATION

Strike slip

Thrust (Reverse)

Normal

0 5 10 Miles

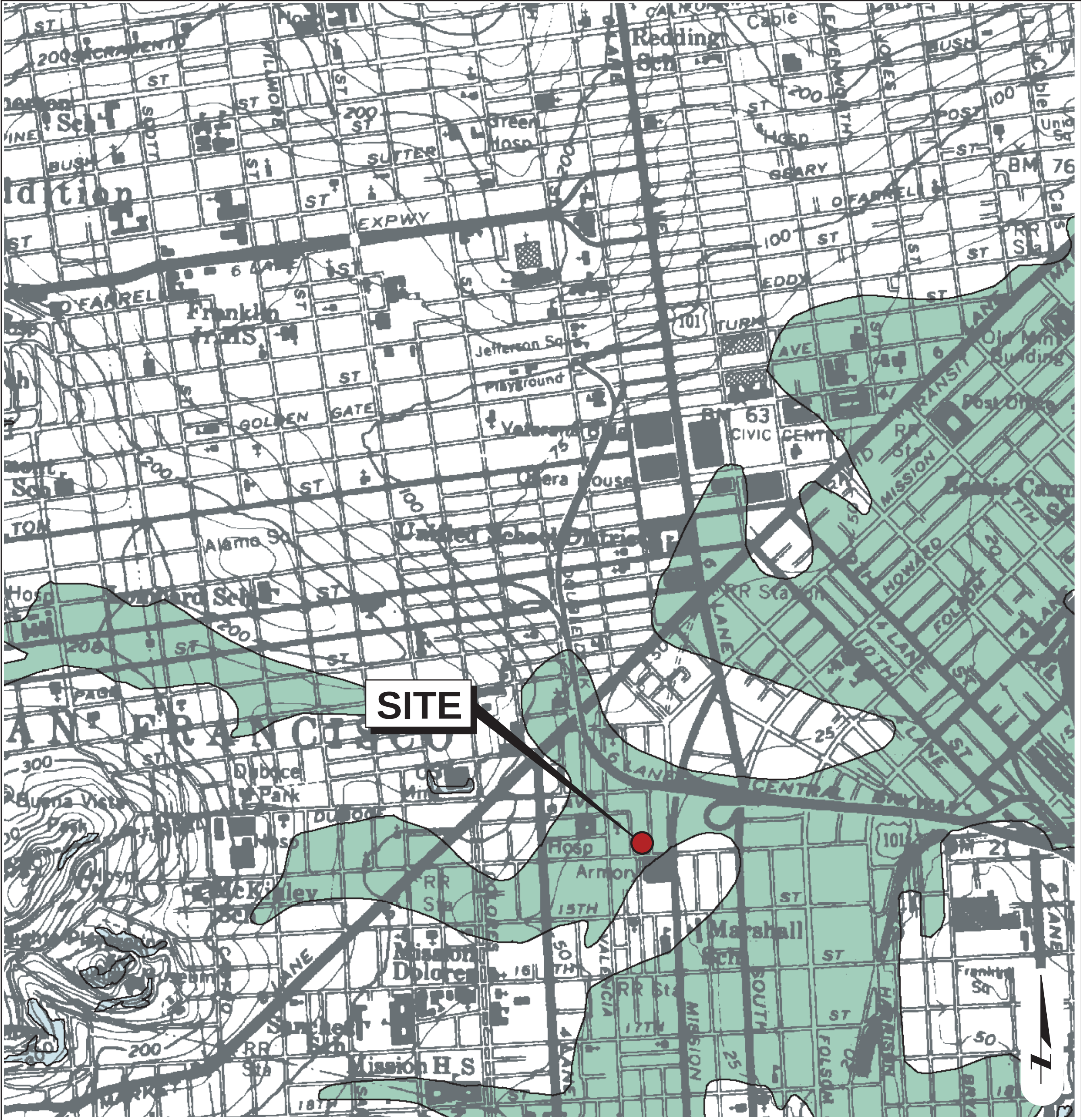
Approximate scale

14TH & STEVENSON
San Francisco, California

REGIONAL FAULT MAP

ROCKRIDGE GEOTECHNICAL

Date 12/18/15 | Project No. 15-1019 | Figure 4



EXPLANATION

🟩 **Liquefaction;** Areas where historic occurence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

🟦 **Earthquake-Induced Landslides;** Areas where previous occurence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

0 1,500 3,000 Feet

Approximate scale

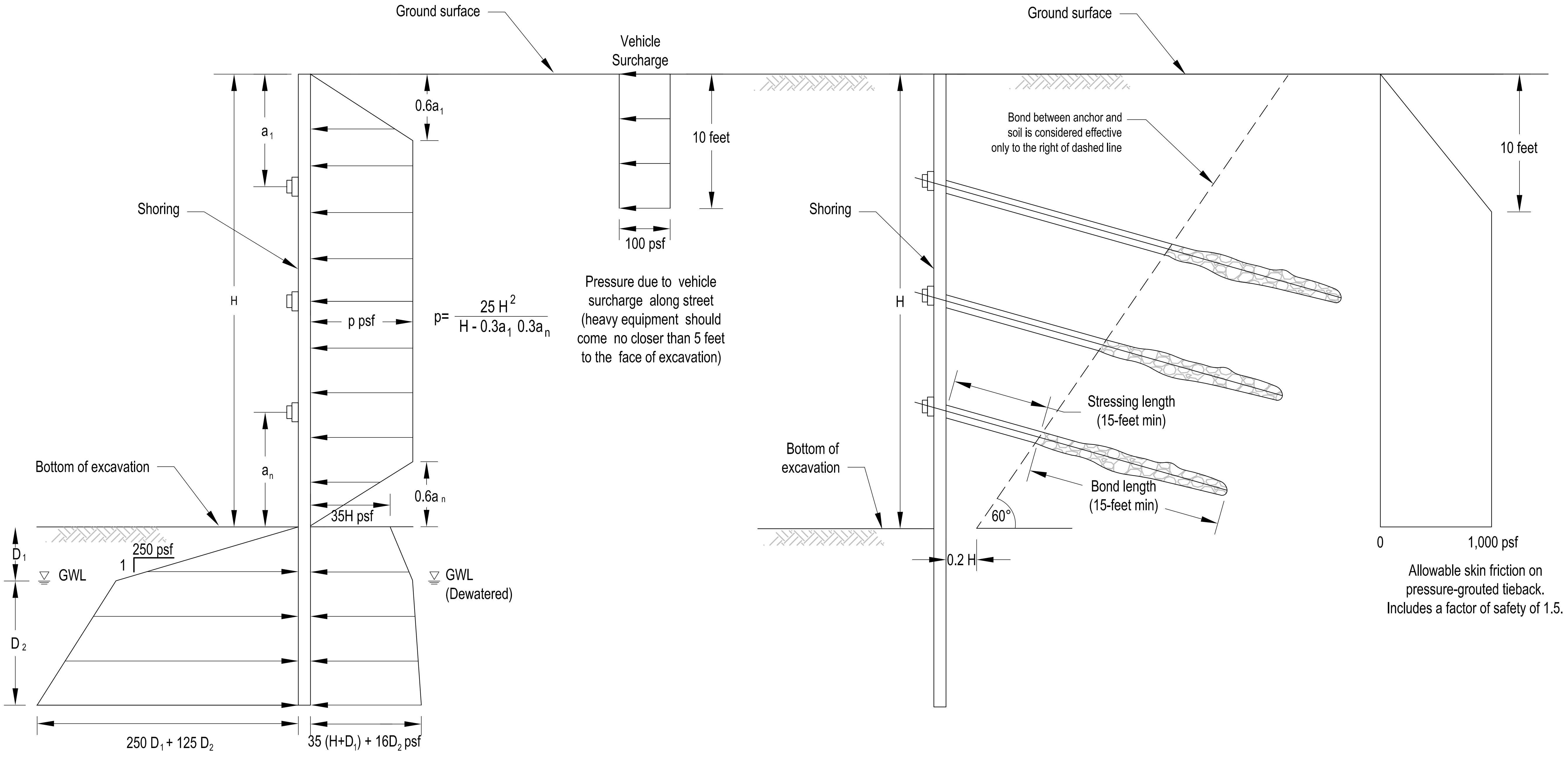
Reference:
State of California "Seismic Hazard Zones"
City and County of San Francisco
Released on November 17, 2000

14TH & STEVENSON
San Francisco, California

ROCKRIDGE
GEOTECHNICAL

SEISMIC HAZARDS ZONE MAP

Date 12/18/15 | Project No. 15-1019 | Figure 5



NOT TO SCALE

Notes:

1. Passive pressures include a factor of safety of about 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over two and three diameters for non-structural concrete and structural concrete, respectively.
3. Active pressure should be assumed to act over one pile diameter.
4. Assumes the site is dewatered at least three feet below bottom of excavation.

14TH & STEVENSON
San Francisco, California

DESIGN PARAMETERS FOR SOLDIER-PILE-AND-LAGGING TEMPORARY SHORING SYSTEM

Date 01/29/16	Project No. 15-1019	Figure 6

ROCKRIDGE GEOTECHNICAL

APPENDIX A

Boring Logs and Cone Penetration Test Results

PROJECT: **14TH & STEVENSON**
San Francisco, California

Log of Boring B-1

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Logged by: M. Hachey

Date started: 12/8/15

Date finished: 12/8/15

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Automatic Hammer

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–2					SP	2.5 inches Asphalt Concrete 2 inches Aggregate Base SAND (SP) brown, very dense, dry, trace fines FILL						
3–4	SPT	◪	6 20 36	81		Corrosion Test, see Appendix B						
5–7						cobbles and gravel in cuttings 8-inch diameter angular cobble						
8–10	S&H	■	5 3 2	4		very loose to loose, moist, subangular to angular gravel, trace debris						
11–13	S&H	■	0 0 3	3	SM ▽	SILTY SAND (SM) dark gray-brown to black, very loose, wet, fine sand Particle Size Distribution, see Appendix B				13	18.9	112
14–16	SPT	◪	0 1 3	6	ML	SILT with SAND (ML) black, medium stiff, wet, trace organics Non-Plastic, see Appendix B					62.9	
16–19	S&H	■	15 20 41	51	SP	SAND (SP) gray, very dense, wet, fine to medium sand, trace rootlets						
19–23	SPT	◪	0 0 4	6	SM	SILTY SAND (SM) black, loose, wet, fine sand, trace organics Particle Size Distribution, see Appendix B				14		
23–28	SPT	◪	4 9 21	43	SP	SAND (SP) gray to gray-brown, dense, wet, fine to medium sand						
29–31	SPT	◪	22 50/6"	72/6"		very dense, trace silt and a thin lenses (2-3 inches thick) of gray, sandy clay						

ROCKRIDGE GEOTECHNICAL

Project No.: 15-1019

Figure: A-1a

PROJECT: **14TH & STEVENSON**
San Francisco, California

Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						SAND (SP) (continued)						
32												
33												
34	SPT	◪	11 36 50/5"	123/ 11"								
35												
36												
37												
38												
39	SPT	◪	15 44 50/5"	135/ 11"	SP							
40												
41												
42												
43												
44	SPT	◪	12 30 41	102								
45												
46												
47						CLAY (CL) gray to dark gray, soft, wet						
48												
49	SPT	◪	0 0 0	0								
50												
51												
52												
53												
54	SPT	◪	0 0 0	0	CL	with trace organics						
55												
56												
57												
58												
59						medium stiff						
60	ST	◨	200 psi									
61												
62												

Boring terminated at a depth of 61 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 12 feet during drilling.

---

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.84 and 1.44, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 15-1019

Figure: A-1b

PROJECT: **14TH & STEVENSON**
San Francisco, California

Log of Boring B-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Logged by: M. Hachey

Date started: 12/8/15

Date finished: 12/8/15

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Automatic Hammer

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

DEPTH (feet)	SAMPLES Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					2.5 inches Asphalt Concrete						
					2 inches Aggregate Base						
1					SAND (SP) brown, loose, moist, fine sand, trace angular gravel						
2											
3											
4											
5											
6	SPT	6 9 4	18	SP	FILL						
7											
8											
9											
10											
11											
12					∇ (12/08/15) SAND (SP) brown, medium dense, wet, fine to medium sand						
13	S&H	12 13 13	22								
14											
15											
16	SPT	3 4 6	14		gray-brown Particle Size Distribution, see Appendix B				2		
17											
18					dense, with a thin 2-inch thick lense of black silty sand						
19	SPT	4 7 18	36								
20											
21				SP	thin lenses of black organics and sandy silt						
22	SPT	6 12 19	45								
23											
24											
25					dark gray-brown, medium dense Particle Size Distribution, see Appendix B						
26	SPT	5 8 8	23						5		
27											
28											
29											
30											
31	SPT	14	66		dense						

ROCKRIDGE GEOTECHNICAL

Project No.: 15-1019

Figure: A-2a

PROJECT: **14TH & STEVENSON**
San Francisco, California

Log of Boring B-2

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
	SPT	◪	14 32	66		SAND (SP) (continued)						
32												
33					SP							
34												
35						CLAYEY SAND (SC) dark brown to black, very dense, wet, abundant organics						
36												
37												
38												
39												
40												
41	SPT	◪	2 11 34	65								
42												
43					SC							
44												
45												
46												
47												
48												
49												
50												
51	S&H	■	5 10 14	20		dark gray-brown to black, medium dense						
52												
53												
54												
55												
56												
57												
58												
59												
60												
61												
62												

Boring terminated at a depth of 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 12 feet during drilling.

---

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.84 and 1.44, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 15-1019

Figure: A-2b

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	Gravels (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⬒ Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

◩ Classification sample taken with Standard Penetration Test sampler

◨ Undisturbed sample taken with thin-walled tube

☒ Disturbed sample

⊙ Sampling attempted with no recovery

◫ Core sample

● Analytical laboratory sample

▥ Sample taken with Direct Push sampler

▥ Sonic

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

14TH & STEVENSON
San Francisco, California

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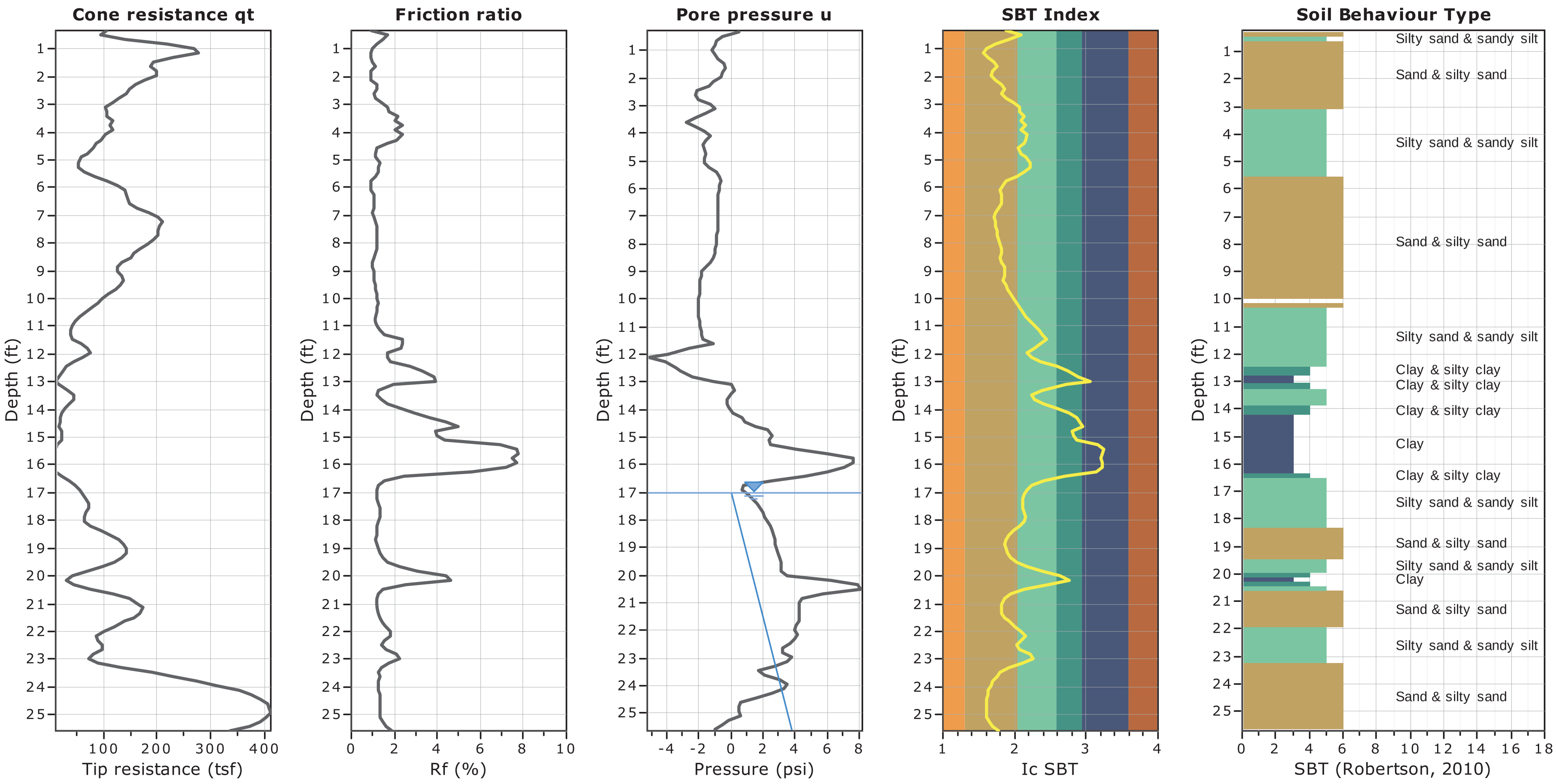
GEOTECHNICAL

CLASSIFICATION CHART

Date 12/18/15	Project No. 15-1019	Figure A-3



SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty clay	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬜ 8. Very stiff sand to clayey sand
🟦 3. Clay to silty clay	🟫 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

Total depth: 25.59 ft, Date: 12/18/2015
Measured Groundwater Depth: 17 feet
Cone Operator: Middle Earth Geo Testing, Inc.

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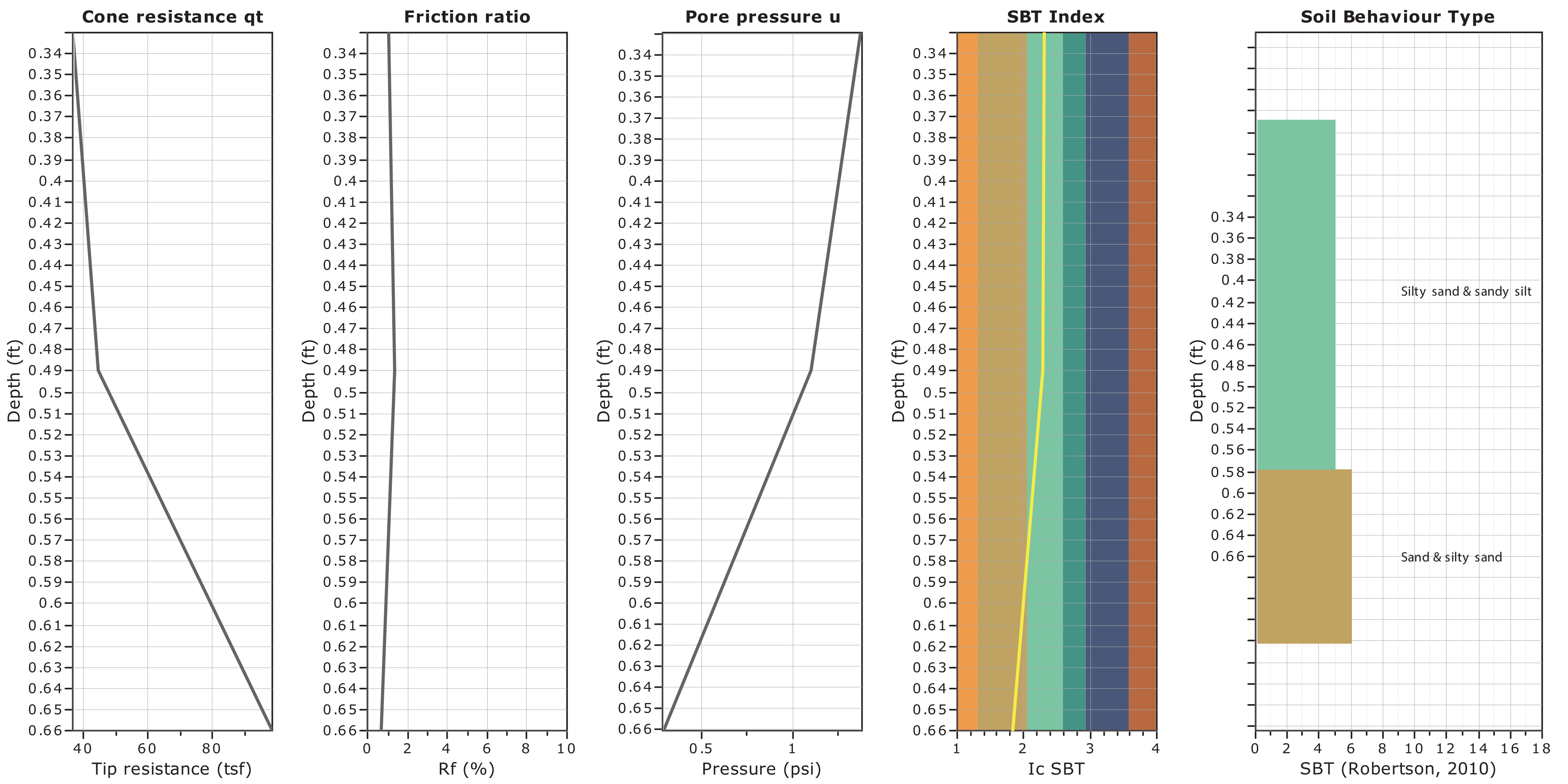
San Francisco, California

ROCKRIDGE
GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 01/13/16	Project No. 15-1019	Figure A-4



SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

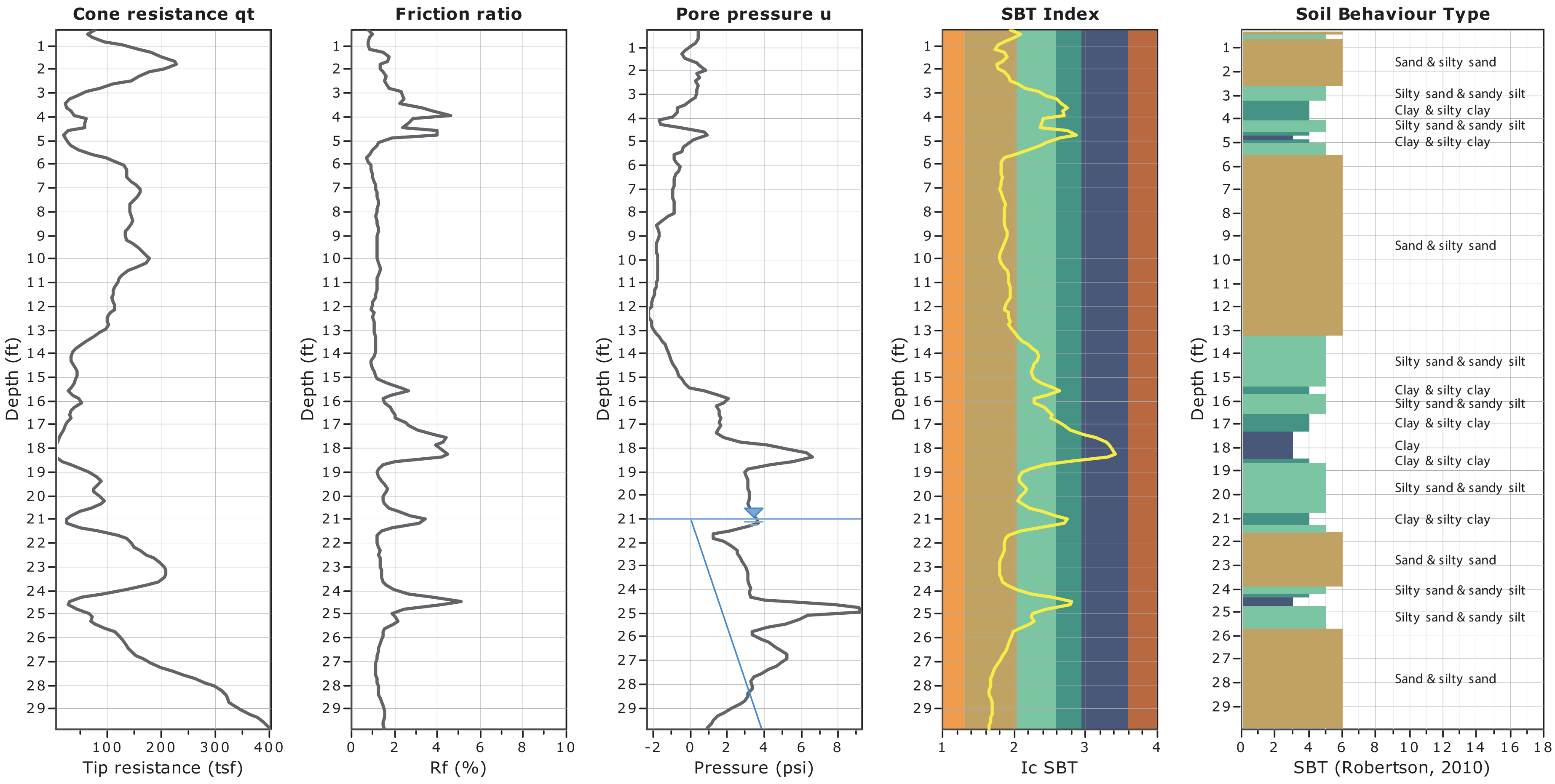
Total depth: 0.66 ft, Date: 12/18/2015
Groundwater not measured
Cone Operator: Middle Earth Geo Testing, Inc.

14TH & STEVENSON
San Francisco, California

ROCKRIDGE
GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 01/13/16	Project No. 15-1019	Figure A-5



SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬜ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

Total depth: 29.86 ft, Date: 12/18/2015
Measured Groundwater Depth: 21 feet
Cone Operator: Middle Earth Geo Testing, Inc.

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San Francisco, California

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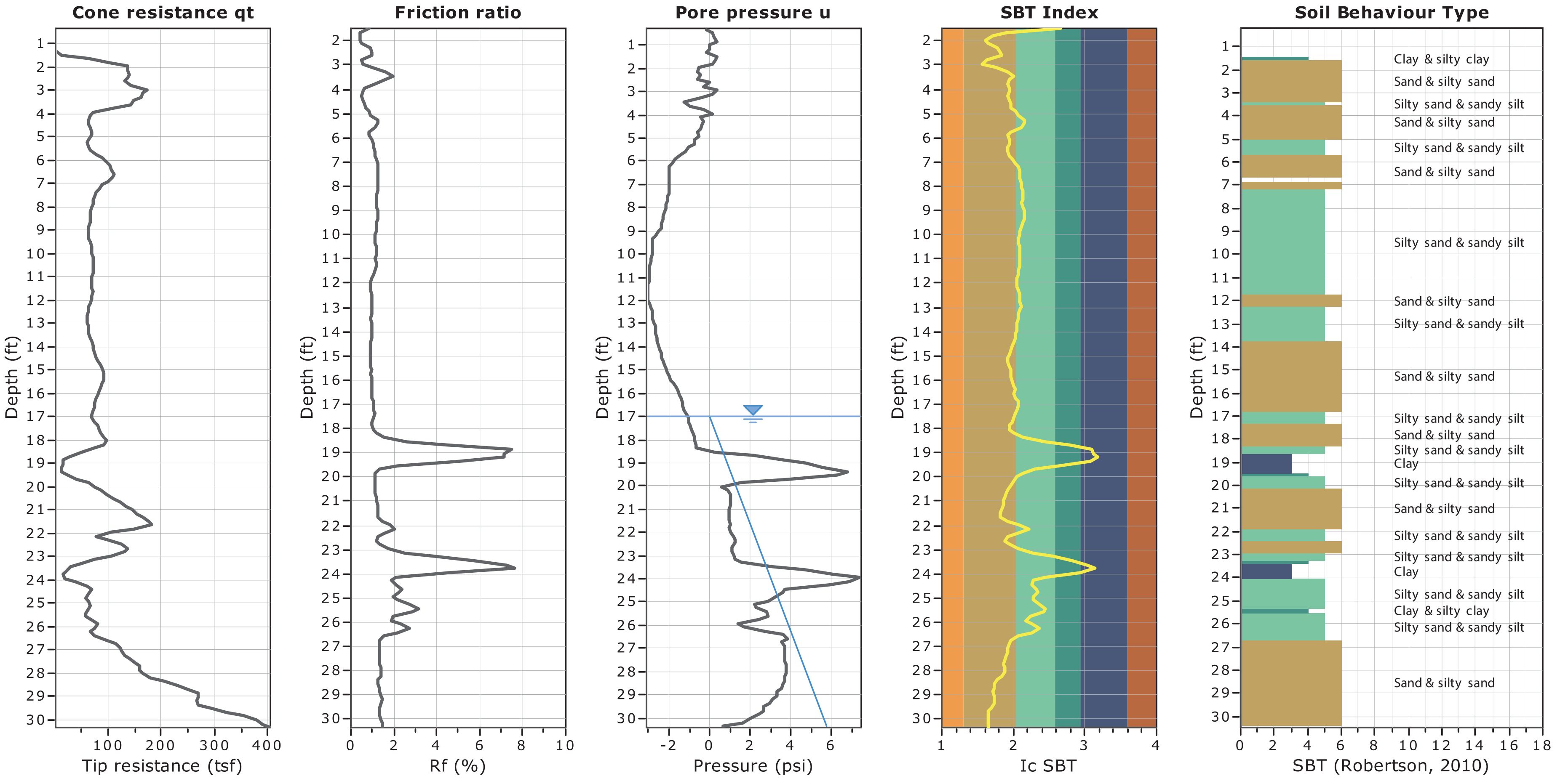
CONE PENETRATION TEST RESULTS

CPT-3

Date 01/13/16 | Project No. 15-1019 | Figure A-6



SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty clay	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to clayey sand
🟦 3. Clay to silty clay	🟫 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

Total depth: 30.35 ft, Date: 12/18/2015
Measured Groundwater Depth: 17 feet
Cone Operator: Middle Earth Geo Testing, Inc.

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San Francisco, California

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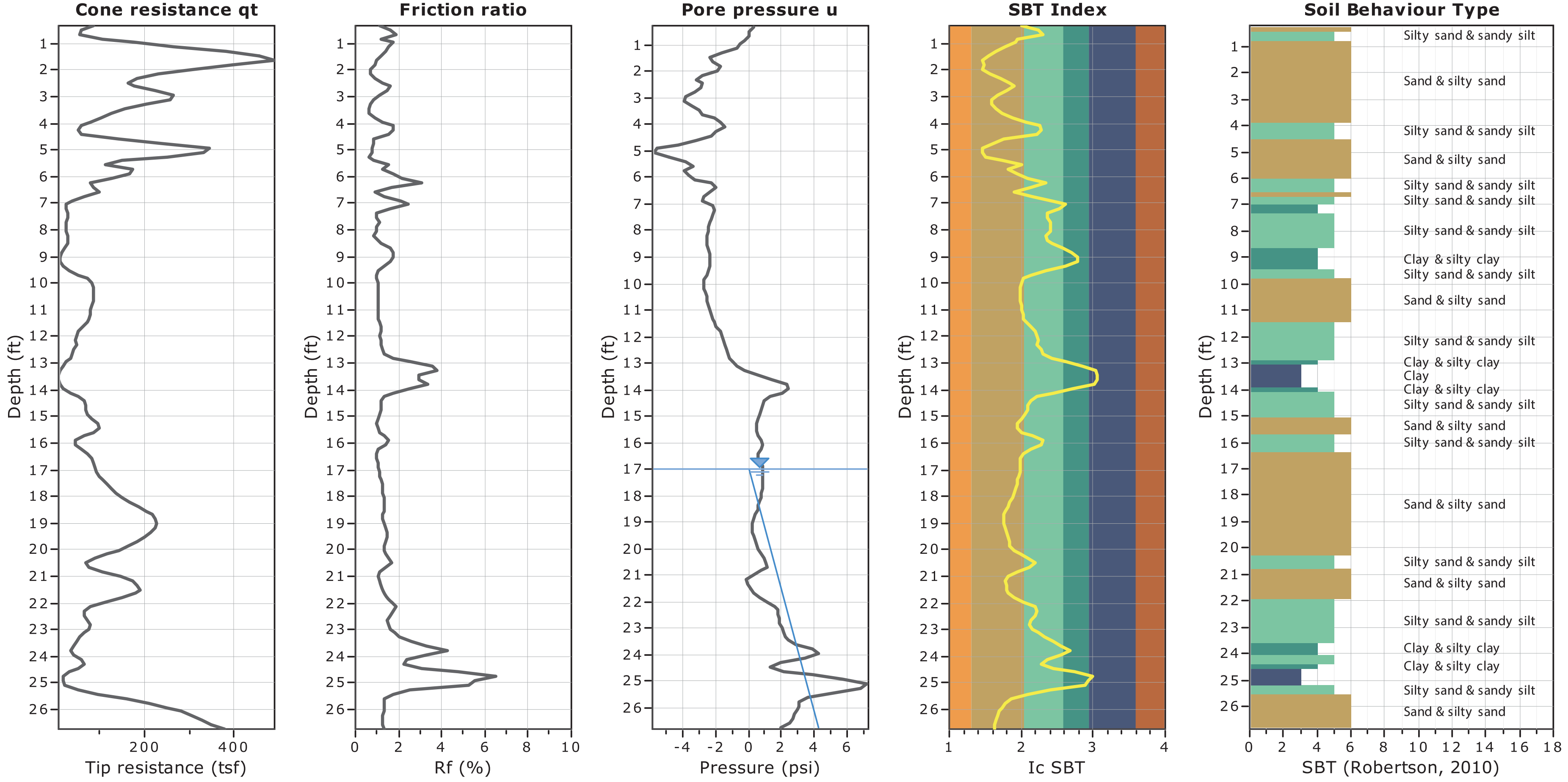
GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-4

Date 01/13/16	Project No. 15-1019	Figure A-7



SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬜ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

Total depth: 26.74 ft, Date: 12/18/2015
Measured Groundwater Depth: 17 feet
Cone Operator: Middle Earth Geo Testing, Inc.

14TH & STEVENSON
San Francisco, California

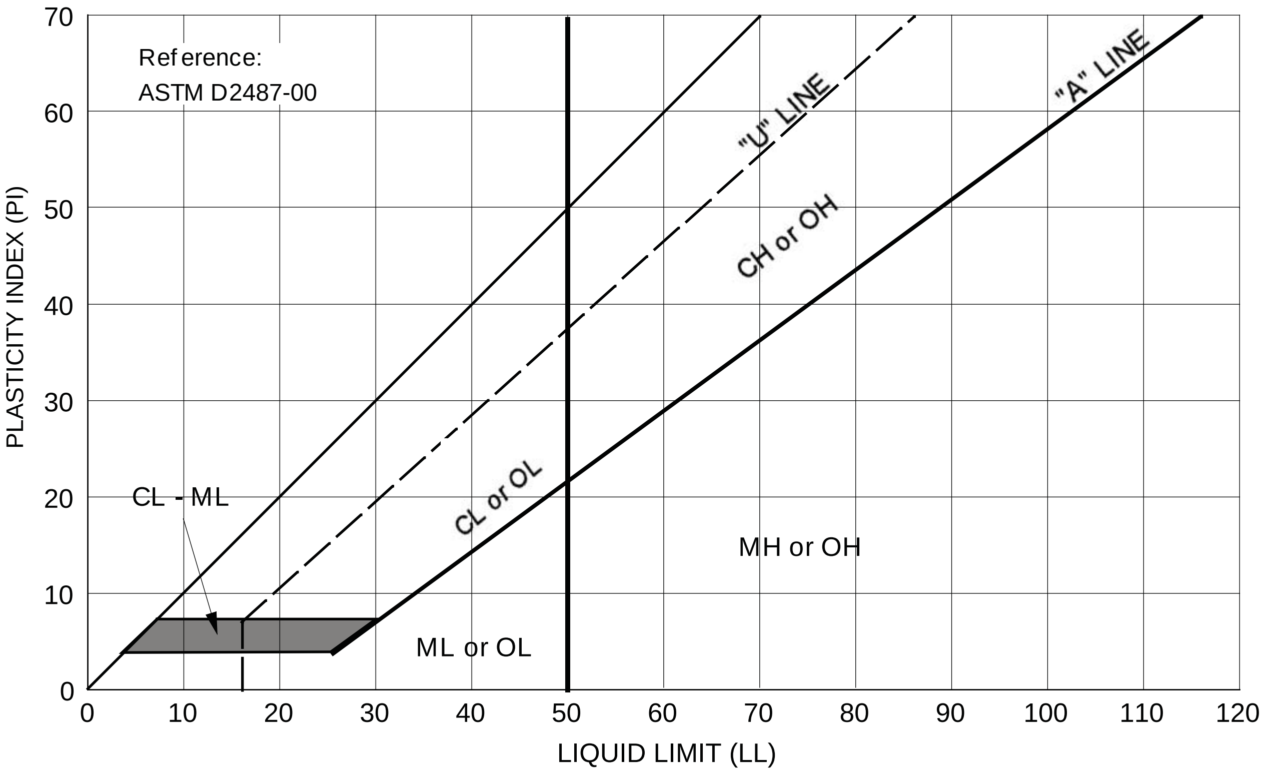
ROCKRIDGE
GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-5

Date 01/13/16 | Project No. 15-1019 | Figure A-8

APPENDIX B

Laboratory Test Results



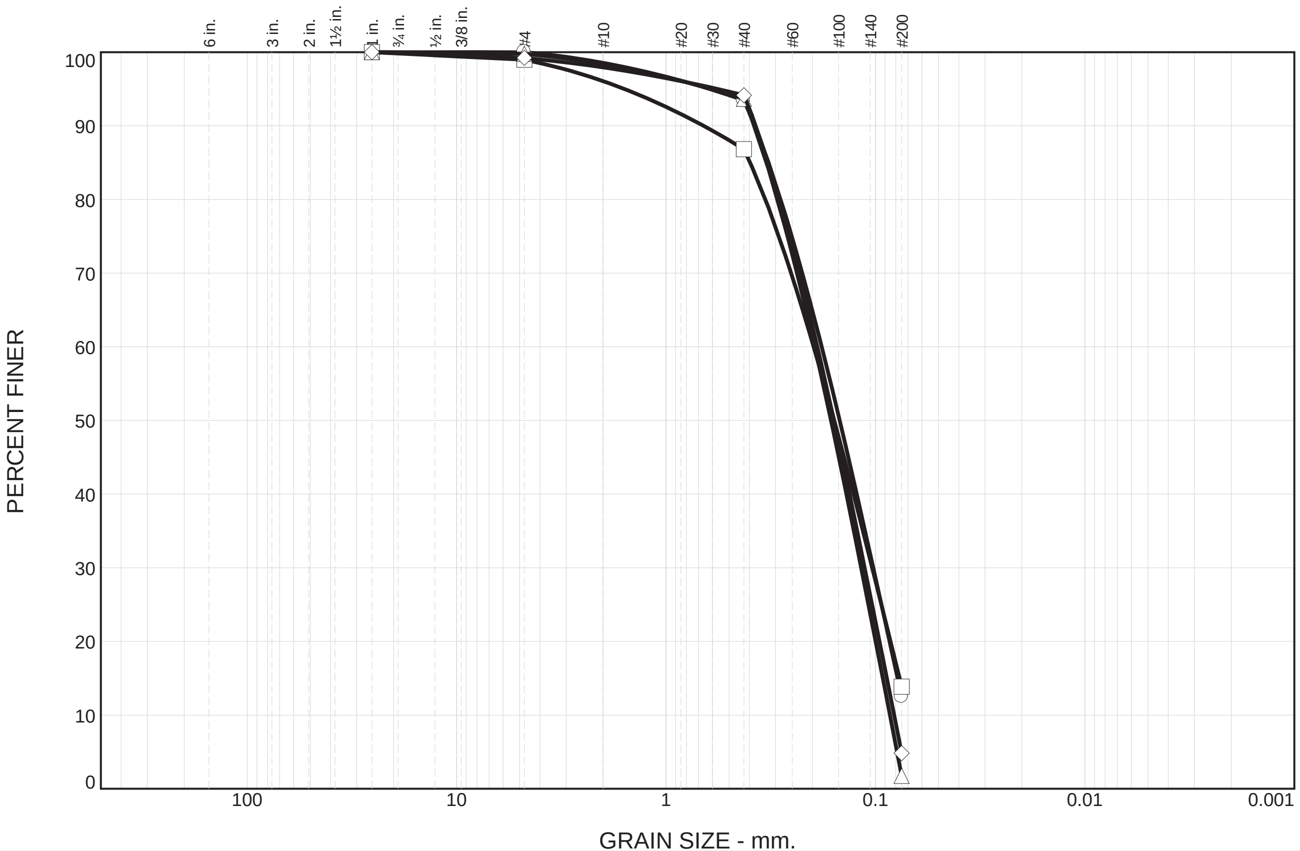
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 14.0 - 15.5 feet	SILT with SAND (ML), black	62.9	NV	NP	--

14TH & STEVENSON
San Francisco, California

ROCKRIDGE
GEOTECHNICAL

PLASTICITY CHART

Date 01/13/16 | Project No. 15-1019 | Figure B-1



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

MATERIAL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-1	3	12.0 - 12.5'	SILTY SAND, dark gray-brown to black	SM
□	B-1	6	20.0 - 21.5'	SILTY SAND, black	SM
△	B-2	3	15.0 - 16.5'	SAND, gray-brown	SP
◇	B-2	6	25.0 - 26.5'	SAND, dark gray-brown	SP

14TH & STEVENSON
San Francisco, California

ROCKRIDGE
GEOTECHNICAL

PARTICLE SIZE DISTRIBUTION REPORT

Date 01/13/16 | Project No. 15-1019 | Figure B-2

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 12/18/2015
Date Submitted 12/15/2015

To: Craig Shields
Rockridge Geotechnical, Inc.
270 Grand Ave
Oakland, CA 94610

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 15-1019 Site ID : B-1 1-1@3-4.5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 70990-148084.

EVALUATION FOR SOIL CORROSION

Soil pH	8.28		
Moisture	5.1 %		
Minimum Resistivity	1.29	ohm-cm (x1000)	
Chloride	67.6	ppm	00.00676 %
Sulfate	198.4	ppm	00.01984 %
Redox Potential	(+) 205	mv	
Sulfides	Presence - NEGATIVE		

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422
Redox Potential ASTM G-200, Sulfides AWWA C105/A25.5

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 12/18/2015
Date Submitted 12/15/2015

To: Craig Shields
Rockridge Geotechnical, Inc.
270 Grand Ave
Oakland, CA 94610

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : 15-1019 Site ID : B-1 1-1@3-4.5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 70990-148085.

Extractable Sulfide Analysis

TYPE OF TEST	RESULTS	UNITS
Sulfide	ND	mg/kg

DETECTION LIMITS
Sulfide 0.05

Mehtod 9031m, ND = Below Detection Limits

Dewatering Sites within 600 feet of 344 14th Street

address	year built
245 Valencia Street	2018
380 14th Street	2012
1800 Mission Street (SF Armory)	1912
1801 Mission Street	2019
1863 Mission Street	2019
1875 Mission Street	2015
1600 15th Street/1880 Mission Street (VARA)	2013

Memorandum

To: Manouch Moshayedi, Mx3 Ventures, LLC
From: Tessa Williams, Rockridge Geotechnical, Inc.
Date: December 10, 2018
Project: 14th & Stevenson, San Francisco
Project No.: 15-1019

This memorandum presents the results of our evaluation of the potential impacts to groundwater conditions (Mission Creek) within the site vicinity caused by construction of the proposed mixed-use development at 344 14th Street and 1463 Stevenson Street in San Francisco. We previously performed a geotechnical investigation for this project, the results of which were presented in our report dated May 6, 2016.

The project site is located on the northeastern corner of the intersection of 14th and Stevenson streets and consists of two adjacent rectangular parcels that form an L-shaped project site with maximum plan dimensions of 130 by 237 feet. The site is currently used as a parking lot. Current plans prepared by BAR Architects, dated December 3, 2018, call for two buildings to be constructed on the site. The proposed building on Lot 2 will consist of a three-story building with one level of below-grade parking extending to a depth of approximately 12 feet below existing site grade at the eastern portion of the building and stacked parking extending about 19-1/2 feet below site grade along the western perimeter of building. The proposed building on Lot 1 will consist of a 4- to 7-story building over one level of below-grade parking. We anticipate the ground improvement elements will consist of 20-inch-diameter columns comprised of controlled low-strength material (CLSM) spaced at 6 to 7 feet on center. Conservatively assuming a 6-foot spacing between the soil-improvement elements, the replacement ratio (area of columns divided by tributary area for each column) would be approximately 6 percent.

We understand there are concerns regarding impacts the proposed new basement will have on the groundwater conditions (Mission Creek) within the site vicinity and, specifically, the effects on the armory building located across 14th Street directly south of the project site.

The armory building, located approximately 50 feet south of the project site, is a four-story structure with one basement level and a deeper sub-basement in the southwestern corner, which is on the order of 200 to 250 feet south of the subject property. Previous investigations by others indicate the groundwater level at the armory building generally slopes down to the east with elevations ranging from about 10.5 feet (SFCD) at the western perimeter to 6.5 feet at the eastern perimeter. According to existing site plans, the armory basement floor slab elevations generally range from approximately 5.25 to 10.0 feet to about elevation 0.33 feet in the sub-basement. Groundwater that flows into the sub-basement through an opening in the basement wall is continually pumped into the City and County of San Francisco storm/sewer system so that water does not rise above the main basement floor level. There is also an underslab drainage system below the main basement floor; however, it is not clear if that underslab drainage system is still functioning.

Considering the proposed building closest to the armory building will only have one basement level that will extend a few feet below the groundwater table and the ground improvement elements that will be installed below the buildings will only comprise approximately six percent of the total soil volume in which the elements are installed, we conclude the rise in groundwater elevation in the site vicinity as a result of the proposed construction will be negligible and, therefore, will not negatively impact the active dewatering system at the neighboring armory building.

If you have any questions regarding this memorandum, please call.

January 8, 2019
Project No. 15-1019

Mr. Manouch Moshayedi
Mx3 Ventures, LLC

Newport Beach, California 92663

Subject: Geotechnical Consultation
Modifications to Proposed Mixed-Use Development
344 14th Street, 1463-1499 Stevenson Street, 86-98 Woodward Street
San Francisco, California

Dear Mr. Moshayedi,

We previously performed a geotechnical investigation for the properties at 344 14th Street, 1463-1499 Stevenson Street, 86-98 Woodward Street in San Francisco, the results of which were presented in our report dated May 6, 2016. When we prepared our report, the proposed development consisted of a mixed-use building with one level of below-grade parking, a one-story concrete podium at grade, and 2 to 4 stories of residential units above the podium. Current plans prepared by BAR Architects, dated December 3, 2018, call for two buildings to be constructed on the site and include a 10-foot buffer between the basement of the proposed project and the adjacent buildings. The proposed building on Lot 2 will consist of a three-story building with one level of below-grade parking extending to a depth of approximately 12 feet below existing site grade at the eastern portion of the building and stacked parking extending about 19-1/2 feet below site grade along the western perimeter of building. The proposed building on Lot 1 will consist of a 4- to 7-story building over one level of below-grade parking.

In our May 6, 2016 report, we recommend the foundation system for the proposed development consist of a mat foundation on improved soil or a deep foundation system. The recommendations for foundation design and other geotechnical aspects of the project presented in our May 6, 2016 report are also applicable to the currently proposed buildings.

Sincerely,
ROCKRIDGE GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
CRAIG S. SHIELDS
GE2116
3/31/19
GEOTECHNICAL
STATE OF CALIFORNIA

Craig S. Shields, P.E., G.E.
Principal Geotechnical Engineer

Table 2: Forecast Growth by Rezoning Option

2025 Totals							
	Eastern Neighborhoods						
	Mission	Showplace Sq./ Potrero Hill	Eastern SoMa	Central Waterfront	**Subtotal**	**Rest of City**	**Total**
Baseline (2000)							
Housing Units	13,309	5,539	5,818	798	25,464	304,239	329,703
Household Population	41,788	13,501	10,211	1,704	67,204	689,763	756,967
PDR Jobs	12,071	6,966	6,579	6,851	32,467	63,080	95,547
Non-PDR Jobs	11,038	13,769	11,013	4,368	40,188	498,700	538,888
Total Jobs	23,109	20,735	17,592	11,219	72,655	561,780	634,435
2025 No-Project							
Housing Units	13,729	6,190	7,399	1,017	28,335	320,446	348,781
Household Population	43,906	14,293	13,276	2,014	73,489	725,728	799,217
PDR Jobs	11,086	5,280	5,514	7,211	29,091	74,226	103,317
Non-PDR Jobs	13,922	19,376	15,251	4,669	53,218	607,619	660,837
Total Jobs	25,008	24,656	20,765	11,880	82,309	681,845	764,154
Option A							
Housing Units	14,091	7,833	8,112	4,443	34,479	332,607	367,086
Household Population	45,116	16,911	14,049	8,314	84,390	752,100	836,490
PDR Jobs	11,210	7,718	5,357	7,175	31,460	74,757	106,218
Non-PDR Jobs	13,291	18,736	14,215	4,672	50,914	609,305	660,218
Total Jobs	24,500	26,454	19,572	11,847	82,374	684,062	766,436
Option B							
Housing Units	14,427	8,174	8,326	1,922	32,849	333,362	366,211
Household Population	46,089	17,550	14,410	3,632	81,681	752,767	834,448
PDR Jobs	11,038	5,176	5,099	7,038	28,351	72,064	100,415
Non-PDR Jobs	14,125	19,374	15,649	4,653	53,801	606,720	660,522
Total Jobs	25,162	24,550	20,748	11,691	82,152	678,784	760,936
Option C							
Housing Units	15,363	9,430	8,901	1,628	35,322	330,998	366,320
Household Population	48,865	20,360	15,388	3,079	87,692	747,058	834,750
PDR Jobs	5,602	5,063	5,122	7,211	22,998	73,265	96,263
Non-PDR Jobs	22,637	18,699	16,278	4,580	62,195	600,861	663,056
Total Jobs	28,239	23,762	21,400	11,791	85,193	674,126	759,319

SOURCE: San Francisco Planning Department, 2005.

Table 2: Forecast Growth by Rezoning Option (continued)

Change: Difference between 2025 Totals and Baseline(2000) Totals

	Eastern Neighborhoods						
	Mission	Showplace Sq./ Potrero Hill	East SoMa	Central Waterfront	**Subtotal**	**Rest of City**	**Total**
2025 No Project							
Housing Units	420	651	1,581	219	2,871	16,207	19,078
Household Population	2,118	792	3,065	310	6,285	35,965	42,250
PDR Jobs	-985	-1,686	-1,065	360	-3,376	11,146	7,770
Non-PDR Jobs	2,884	5,607	4,238	301	13,030	108,919	121,949
Total Jobs	1,899	3,921	3,173	661	9,654	120,065	129,719
Option A							
Housing Units	782	2,294	2,294	3,645	9,015	28,368	37,383
Household Population	3,328	3,410	3,838	6,610	17,186	62,337	79,523
PDR Jobs	-861	752	-1,222	324	-1,007	11,677	10,671
Non-PDR Jobs	2,253	4,967	3,202	304	10,726	110,605	121,330
Total Jobs	1,391	5,719	1,980	628	9,719	122,282	132,001
Option B							
Housing Units	1,118	2,635	2,508	1,124	7,385	29,123	36,508
Household Population	4,301	4,049	4,199	1,928	14,477	63,004	77,481
PDR Jobs	-1,033	-1,790	-1,480	187	-4,116	8,984	4,868
Non-PDR Jobs	3,087	5,605	4,636	285	13,613	108,020	121,634
Total Jobs	2,053	3,815	3,156	472	9,497	117,004	126,501
Option C							
Housing Units	2,054	3,891	3,083	830	9,858	26,759	36,617
Household Population	7,077	6,859	5,177	1,375	20,488	57,295	77,783
PDR Jobs	-6,469	-1,903	-1,457	360	-9,469	10,185	716
Non-PDR Jobs	11,599	4,930	5,265	212	22,007	102,161	124,168
Total Jobs	5,130	3,027	3,808	572	12,538	112,346	124,884

SOURCE: San Francisco Planning Department, 2005

Transportation Calculations

Project Name

Project Number

🟦 *INPUT*

🟨 **OUTPUT**

RESIDENTIAL

TRIP GENERATION	
Square Feet of Residential Space	56,630
Number of Studio/One-Bedroom Units	27
Number of Two-Bedroom or more Units	18
Trip rate for Studio/One-Bedroom Unit	7.5
Trip rate for Two-Bedroom or more	10.0
P.M. Peak-Hour Percentage of Daily Trips	17.3%
Daily Person-Trips	383
P.M. Peak-Hour Person-Trips	66

*CENSUS DATA **	
Census Tract Number	201
Workers 16 years and over (TOTAL)	2,848
Car, truck, or van	808
Workers per car, truck, or van	1.07
Public transportation	1,094
Motorcycle	31
Bicycle	302
Walked	426
Other means (Include Taxi)	55
Worked at home	132
TOTAL	2,848
"TOTAL" - "Worked at home"	2,716

* 2000 Census - Journey to Work

PARKING DEMAND	
Studio/One-Bedroom rate (vehicles/unit)	1.1
Two-Bedroom plus rate (vehicles/unit)	1.5
Studio/One-Bedroom Parking Demand	30
Two-Bedroom plus Parking Demand	27
TOTAL (number of parking spaces)	**57**

LOADING DEMAND	
Average Hour Truck-Trips	0.08
Peak Hour Truck-Trips (10 a.m. - 1 p.m.)	0.10

MODE SPLIT

	Percentage	***Daily Person-Trips***	***P.M. Peak-Hour Person-Trips***
Auto	30%	114	20
Transit	40%	154	27
Walked	16%	60	10
Other means	14%	55	9
TOTAL	100%	383	66

AUTOMOBILE

	Daily	P.M. Peak-Hour
Vehicle-Trips	106	18

RETAIL

TRIP GENERATION	
Square Feet of Retail Space	5,800
Trip Rate for Retail Use	150
P.M. Peak-Hour Percentage of Daily Trips	9.0%
Daily Person-Trips	870
P.M. Peak-Hour Person-Trips	78

EMPLOYEES	
Average gross square foot per employee	350
No. of Employees	17

WORK / NON-WORK SPLIT						
	Daily Person-Trips			**P.M. Peak-Hour Person-Trips**		
	Work	Non-Work	TOTAL	Work	Non-Work	TOTAL
Percentage	4%	96%	100%	4%	96%	100%
Person-trips	35	835	870	3	75	78

PARKING DEMAND	
Short-Term	26
Long-Term	10
TOTAL (no. of spaces)	**35**

MODE SPLIT						
	Daily Person-Trips			**P.M. Peak-Hour Person-Trips**		
	Work	Non-Work	TOTAL	Work	Non-Work	TOTAL
Auto	25	535	560	2	48	50
Transit	7	98	105	1	9	9
Walk	2	187	189	0	17	17
Other	1	15	16	0	1	1
TOTAL	35	835	870	3	75	78

*Work / Non-Work Percentages **		
	Work	Non-Work
Auto	71.1%	64.1%
Transit	20.2%	11.7%
Walk	5.8%	22.4%
Other	2.9%	1.8%

* From Appendix E of the *Guidelines*

AUTOMOBILES						
	Daily Vehicle-Trips			**P.M. Peak-Hour Vehicle-Trips**		
	Work	Non-Work	TOTAL	Work	Non-Work	TOTAL
Persons/auto	1.23	1.90	--	1.23	1.90	--
Vehicle-Trips	20	282	302	2	25	27

LOADING DEMAND	
Average Hour Truck-Trips	0.06
Peak-Hour Truck-Trips	0.07

OFFICE

TRIP GENERATION	
Square Feet of Office Space	19,000
Trip Rate for Office Use	18.1
P.M. Peak-Hour Percentage of Daily Trips	8.5%
Daily Person-Trips	344
P.M. Peak-Hour Person-Trips	29

EMPLOYEES	
Average gross square foot per employee	276
No. of Employees	69

WORK / NON-WORK SPLIT

	Daily Person-Trips			**P.M. Peak-Hour Person-Trips**		
	Work	Non-Work	TOTAL	Work	Non-Work	TOTAL
Percentage	36%	64%	100%	83%	17%	100%
Person-trips	124	220	344	24	5	29

PARKING DEMAND	
Short-Term	5
Long-Term	40
TOTAL (no. of spaces)	**45**

MODE SPLIT

	Daily Person-Trips			**P.M. Peak-Hour Person-Trips**		
	Work	Non-Work	TOTAL	Work	Non-Work	TOTAL
Auto	88	125	213	17	3	20
Transit	25	41	66	5	1	6
Walk	7	36	43	1	1	2
Other	4	18	22	1	0	1
TOTAL	124	220	344	24	5	29

*Work / Non-Work Percentages **

	Work	Non-Work
Auto	71.1%	56.8%
Transit	20.2%	18.6%
Walk	5.8%	16.3%
Other	2.9%	8.3%

* From Appendix E of the *Guidelines*

AUTOMOBILES

	DAILY			**P.M. PEAK-HOUR**		
	Work	Non-Work	TOTAL	Work	Non-Work	TOTAL
Persons/auto	1.23	2.26	--	1.23	2.26	--
Automobiles	72	55	127	14	1	15

LOADING DEMAND	
Average Hour Truck-Trips	0.18
Peak-Hour Truck-Trips	0.23

SUMMARY

TRIP GENERATION	
Daily Person-Trips	1,596
P.M. Peak-Hour Person-Trips	174

MODE SPLIT (Person-Trips)		
	Daily	P.M. Peak-Hour
Auto	887	90
Transit	325	42
Walk	292	30
Other	93	

AUTOMOBILES		
	Daily	P.M. Peak-Hour
Vehicle-Trips	535	61

PARKING DEMAND	
No. of Parking Spaces	137

LOADING DEMAND	
Average Hour Truck-Trips	0.32
Peak Hour Truck-Trips	0.40

San Francisco Planning Department
1650 Mission Street
San Francisco, CA 94103

Subject: **Eastern Neighborhoods Citizen Advisory Committee (EN CAC) Response to the EN Monitoring Reports (2011-2015)**

Dear President Fong and Members of the Planning Commission:

At your September 22, 2016 Regular Meeting, you will hear a presentation on the Eastern Neighborhoods Five Year Monitoring Report (2011 – 2015). Attached, please find the statement prepared by the Eastern Neighborhoods Citizen Advisory Committee (EN CAC) in response to this report.

As you know, we are a 19 member body created along with the Eastern Neighborhoods Plans in 2009. We are appointed by both the Mayor and the Board of Supervisors and are made up of wide range of residents, business and property owners, developers, and activists. Our charge is to provide input on many aspects of the EN Plans' implementation including but not limited to: (1) how to program funds raised through impact fees, (2) proposed changes in land use policy, and (3) the scope and content of the Monitoring Report.

We have been working closely with staff over the course of the last year to assure the Monitoring Report is accurate and contains all of the material and analysis required by the Planning and Administrative Codes. At our regular monthly meeting in August, we voted to endorse the Monitoring Report that is now before you. We understand that while the Monitoring Report is to provide data, analysis, and observations about development in the EN, it is not intended to provide conclusive statements about its success. Because of this, we have chosen to provide you with the attached statement regarding the where we believe the EN Plan has been successful, where it has not, and what the next steps should be in improving the intended Plans' goals and objectives.

Several of our members will be at your September 22 hearing to provide you with our prospective. We look forward to having a dialog with you on what we believe are the next steps.

Please feel free to reach out to me, Bruce Huie, the CAC Vice-Chair or any of our members with questions or thoughts through Mat Snyder, CAC staff. (mathew.snyder@sfgov.org; 415-575-6891)

Sincerely,

Chris Block
Chair
Eastern Neighborhoods Citizen Advisory Committee

Eastern Neighborhoods Citizen Advisory Committee
Response to the Five-Year EN Monitoring Report (2011-2015)

INTRODUCTION

The Eastern Neighborhoods Citizen Advisory Committee (EN CAC) is comprised of 19 individuals appointed by members of the Board of Supervisors and the Mayor to represent the five neighborhoods included in the Eastern Neighborhoods Plan (EN Plan) - Mission, Showplace Square/Potrero Hill, Central Waterfront, East SoMa and Western SoMa.

The EN CAC has prepared this document in response to the five-year monitoring report, which was prepared under the specifications of the EN Plan adopting ordinance and approved for submittal to the Planning Commission by the EN CAC on September 22, 2016. This response letter was prepared to provide context and an on-the-ground perspective of what has been happening, as well as outline policy objectives and principles to support the community members in each of these neighborhoods who are most impacted by development undertaken in response to the Plan.

BACKGROUND

High Level Policy Objectives and Key Planning Principles of the EN Plan:
The Eastern Neighborhoods Plans represent the City's and community's pursuit of two key policy goals:

1. Ensuring a stable future for PDR businesses in the city by preserving lands suitable to these activities and minimizing conflicts with other land uses; and
2. Providing a significant amount of new housing affordable to low, moderate and middle income families and individuals, along with "complete neighborhoods" that provide appropriate amenities for the existing and new residents.

In addition to policy goals and objectives outlined in individual plans referenced above, all plans are guided by four key principles divided into two broad policy categories:

The Economy and Jobs:

1. Reserve sufficient space for production, distribution and repair (PDR) activities, in order to support the city's economy and provide good jobs for residents.
2. Take steps to provide space for new industries that bring innovation and flexibility to the city's economy.

People and Neighborhoods:

1. Encourage new housing at appropriate locations and make it as affordable as possible to a range of city residents.

2. Plan for transportation, open space, community facilities and other critical elements of complete neighborhoods.

The ordinances that enacted the EN Plan envision an increase of 9,785 and over 13,000 new jobs in the Plan Area over the 20 year period - 2009 to 2029.

The Eastern Neighborhood's approval included various implementation documents including an Interagency Memorandum of Understand (MOU) among various City Departments to provide assurances to the Community that the public benefits promised with the Plan would in fact be provided.

COMMENTARY FROM THE EN CAC

The below sections mirror the four key principles of the EN Plan in organization. Below each principle are the aspects of the Plan that the EN CAC see as "working" followed by "what is not working".

PRINCIPLE 1. Reserve sufficient space for production, distribution and repair (PDR) activities, in order to support the city's economy and provide good jobs for residents.

What Seems to be Working:

PDR has been preserved and serves as a model for other cities

Job Growth in the EN, including manufacturing, is almost double the amount that was anticipated in the EN Plan.

What Seems to Not be Working

Loss of PDR jobs in certain sectors.

There is much anecdotal evidence of traditional PDR businesses being forced out of their long-time locations within UMU zones. In certain neighborhoods, the UMU zoning has lead to gentrification, as long standing PDR uses are being replaced with upscale retail and other commercial services catering to the large segment of market rate housing.

The relocation and displacement of PDR has been especially severe in the arts and in auto repair businesses.
Outside of the PDR zoning, there is no mechanism to preserve the types of uses that typified existing light industrial neighborhoods, such as traditional PDR businesses that offered well-paying entry level positions, and arts uses. This has resulted in a fundamental loss of the long-time creative arts community character of the South of Market, and now also in the Mission District and Dogpatch Neighborhood, with more to come. Traditional PDR businesses cannot afford the rents of new PDR buildings and do not fit well on the ground floor of multi-unit residential buildings. The CAC suggests that the City develop mechanisms within the Planning Code to encourage construction of new PDR space both in the PDR-only zones and the mixed-use districts suitable for these traditional uses, including exploring mandatory BMR PDR spaces.

PRINCIPLE 2: Take steps to provide space for new industries that bring innovation and flexibility to the city's economy.

What Seems to be Working:
The Mixed Use Office zone in East SOMA has produced a number of ground-up office projects which provide space for new industries that can bring innovation and flexibility to the City's economy.

There has been a substantial growth in jobs (approx 32,500 jobs) between 2010-2015 - this far exceeds what was expected over the 20 year term (13,000 jobs). The EN Growth rate appears to be much higher than most other areas of SF.

In other PDR areas, the focus of the EN Plan was to preserve land and industrial space (as opposed to constructing new industrial space) in the various PDR zones within the Plan. Based in part on the robust amount of job growth including job growth within the PDR sector and the need for new industrial space, the City did amend some of the PDR zoning controls on select sites to encourage new PDR space construction in combination with office and/or institutional space. One project has been approved but not yet constructed and features approximately 60,000 square feet of deed-restricted and affordably priced light industrial space and 90,000 square feet of market rate industrial space, for a total of 150,000 square feet of new PDR space.

What Seems to Not be Working
The EN Plan includes a Biotechnology and Medical Use overlay in the northern portion of the Central Waterfront that was put in place to permit expansion of these types of uses resulting from the success of Mission Bay. As of the date of this document, no proposal has been made by the private sector pursuant to the Biotechnology and Medical Use overlay. It's the CAC's view that

the residential uses of the UMU zoning in this specific area supports greater land values then those supported by the Overlay. In addition, the relatively small parcel sizes that characterize the Central Waterfront / Dogpatch area are less accommodating of larger floorplate biotechnology or medical use buildings.

PRINCIPLE 3: Encourage new housing at appropriate locations and make it as affordable as possible to a range of city residents.

What Seems to be Working:
Affordable Housing has been created beyond what would have otherwise:
Throughout San Francisco and certainly in the Eastern Neighborhoods, San Franciscans are experiencing an affordable housing crisis. That being said, the EN Plan's policy mechanisms have created higher levels of inclusionary units than previously required by the City (see Executive Summary, pg. 7). For example, at the time of enactment, UMU zoning required 20% more inclusionary where density controls were lifted, and higher where additional heights were granted. In this regards, UMU has shown to be a powerful zoning tool and is largely responsible for the EN Plan's robust housing development pipeline & implementation. At the same time, community activists and neighborhood organizations have advocated for deeper levels of affordability and higher inclusionary amounts contributing to the creation of additional affordable housing.

Affordable housing funds for Mission and South of Market have been raised:
Some of the initial dollars of impact fees (first $10M) were for preservation and rehabilitation of existing affordable housing that would not have otherwise existed if not for the EN Plan.

A new small-sites acquisition and rehab program was implemented in 2015, and has been successful in preserving several dozen units as permanent affordable housing, protecting existing tenants, and upgrading life-safety in the buildings.

After a few slow years between 2010-2012, the EN Plan is now out-pacing housing production with 1,375 units completed, another 3,208 under construction and 1,082 units entitled with another 7,363 units under permit review (in sum 13,028 units in some phase of development).

What Seems to Not be Working
There is a growing viewpoint centered on the idea that San Francisco has become a playground for the rich. Long-established EN communities and long-term residents of these neighborhoods (people of color, artists, seniors, low-income and working class people,) are experiencing an economic disenfranchisement, as they can no longer afford to rent, to eat out, or to shop in the neighborhood. They see the disappearance of their long-time neighborhood-serving businesses and shrinking sense of community.

Insufficient construction of affordable housing
Although developments have been increasing throughout the Eastern Neighborhoods, we have seen a lack of affordable housing included in what is being built compared to the needs of the current community members. Market-rate development, often regarded as “luxury,” is inaccessible to the vast majority of individuals and families living in the city. The demand for these units has been the basis for a notable level of displacement, and for unseen pressures on people in rent controlled units, and others struggling to remain in San Francisco. A robust amount of affordable housing is needed to ensure those with restricted financial means can afford San Francisco. We have yet to see this level of development emulated for the populations who are most affected by the market-rate tremors. It is time for an approach towards affordable housing commensurate with the surge that we have seen for luxury units.

High cost of housing and commercial rents
Due to the high cost of housing in San Francisco, many long-term residents are finding it increasingly difficult, if not outright impossible, to even imagine socioeconomic progress. As rents have entered into a realm of relative absurdity, residents have found it ever more challenging to continue living in the city. The only way to move up (or even stay afloat, in many cases), is to move out of San Francisco. This situation has unleashed a force of displacement, anxiety, and general uneasiness within many segments of the Eastern Neighborhoods.

Pace of Development
The pace of development within the Eastern Neighborhoods has far exceeded the expectations originally conceived by the City. Since the market is intended to ensure situations are harnessed to maximize profit, we have seen development unaffordable to most. With a few thousand units in the pipeline slated for the Eastern Neighborhoods, much yet needs to be done to ensure that the city can handle such rapid change without destroying the essence of San Francisco.

PRINCIPLE 4: Plan for transportation, open space, community facilities and other critical elements of complete neighborhoods.

What Seems to be Working:
The EN Plan leverages private investment for community benefits by creating predictability for development.
With a clear set of zoning principles and codes and an approved EIR, the EN Plan has successfully laid a pathway for private investment as evidenced by the robust development pipeline. While in some neighborhoods the pace of development may be outpacing those benefits – as is the case in the throughout the Eastern Neighborhoods, there are community benefits being built alongside the development – and a growing impact fee fund source, as developments pay their impact fees as required by the EN Plan.

Funds have been raised for infrastructure that would not otherwise be raised**.** To date $48M has been raised and $100M expected in the next five years (see Tables 6.2.3; 6.2.2)

Priority Projects have been incorporated into the City's Ten Year Capital Plan and the Implementing Agencies' Capital Improvement Plans and work programs.

The Plan has lead to the development of parks and open space recreation. Streetscape improvements to 16th Street, Folsom and Howard, 6th, 7th and 8th Streets are now either fully funded or in process of being funded.

It is expected that more street life will over time support more in-fill retail and other community services.

New urban design policies that were introduced as part of the EN Plan are positive. The creation of controls such as massing breaks, mid-block mews, and active space frontages at street level create a more pedestrian friendly environment and a more pleasant urban experience. In Western Soma, the prohibition of lot aggregation above 100' has proven useful in keeping the smaller scale.

What Seems to Not be Working

A high portion of impact fees (80%) is dedicated to priority projects, such as improvements to 16th Street and, Folsom and Howard Streets. The vast majority of impact fees have been set aside for these large infrastructure projects that might have been better funded by the general fund. This would allow for more funding for improvements in the areas directly impacted by the new development. This also limits the availability of funds for smaller scale projects and for projects that are more EN-centric. There are very limited options in funding for projects that have not been designated as "priority projects".

Absence of open space

The Eastern Neighborhoods lag behind other neighborhoods in San Francisco and nationwide in per capita green space (see Rec and Open Space Element Map 07 for areas lacking open space). Although the impact fees are funding the construction of new parks at 17th and Folsom in the Mission, Daggett Park in Potrero Hill and the rehabilitation of South Park in SOMA, there is a significant absence of new green or open space being added to address the influx of new residents. The Showplace Square Open Space Plan calls for four acres of new parks in the neighborhoods where only one is being constructed.

As a finite and valuable resource, we believe the City has an obligation to treat the waterfront uniquely and should strive to provide green and open waterfront space to the residents of the Eastern Neighborhoods and all City residents in perpetuity.

The pace of infrastructure development is not keeping up with development
There is a lag time between development and the implementation of new infrastructure, seemingly with no clear plan for how to fund the increased infrastructure needs. The plan is now 8 years old: the number of housing units that were projected to be built under the Plan is being exceeded, and we have to date not identified additional infrastructure funds to make up the funding gap. This appears to be a clear failure in the EN Plan implementation, especially because we now have little chance to fill that gap with higher development fees.

The data contained in the Monitoring Report indicates that the EN Plan has been successful in the development of new housing. However, the pace of development appears to have far exceeded the pace of new infrastructure. This is true in each of the EN areas. There is a deficiency in transit options and development of new open space within all plan neighborhoods. A single child-care center in the Central Waterfront has been built as a part of the Plan. As of this time, not one new open space park has opened within the Plan area. The deficiency in public transportation is especially apparent. Ride services have become an increasingly popular option. However, their use contributes to the traffic congestion that is common throughout the city of San Francisco.

The impact fees inadequate
Although the amount of impact fees currently projected to be collected will exceed the sums projected in the Plan, the funding seems inadequate to address the increasing requirements for infrastructure improvements to support the EN Plan. The pace of development has put huge pressure on transportation and congestion and increased the need and desire for improved bike and pedestrian access along major routes within each Plan neighborhood. There is a striking absence of open space, especially in the Showplace/Potrero neighborhood. There has been a significant lag time in the collection of the Plan impact fees and with the implementation of the community benefits intended to be funded by the fees.

Large portions of impact fees are dedicated, which limits agility with funding requests from discretionary fees. The CAC has allocated funding for citizen-led initiatives to contribute a sustainable stream of funding to the Community Challenge Grant program run out of the City Administrators' office. Our past experience is that this program has doubled capacity of local "street parks" in the Central Waterfront from 2 to 4 with the addition of Tunnel Top Park and Angel Alley to the current street parks of Minnesota Grove and Progress Park.

Impacts of non-EIR projects

Data in the report does not properly reflect the impacts of non-EIR projects, such as Pier 70, recent UCSF expansion into Dogpatch and the Potrero Annex. These very large projects are not required to provide impact fees; the public must rely on the developers working with the community to add benefits to their projects.

Upcoming non-EIR projects such as the Warriors arena, Seawall 337 / Pier 48, continued housing development in Mission Bay and UCSF student housing further increase the pressures of density on the neighborhoods. The square footage included in these various projects may equal or exceed all of the projects under the EN Plan. Although these projects are not dependent on the EN Plan to provide their infrastructure, their impacts should be considered for a complete EN approach to infrastructure and other improvements.

Deficiency in Complete Neighborhoods

Complete neighborhoods recognize the need for proximity of daily consumer needs to a home residence. Combining resources to add shopping for groceries, recreation for families, schools for children will create a complete neighborhood. This will then have the additional benefit of reducing vehicle trips.

Many new developments have been built with no neighborhood -serving retail or commercial ground floor space. The UMU zoning has allowed developers to take advantage of a robust real estate market and build out the ground floor spaces with additional residential units, not neighborhood services such as grocery and other stores.

Evictions and move-outs

There are many reports of long-term residents of the neighborhoods being evicted or forced or paid to move out of the area. Younger, high wage-earning people are replacing retirees on fixed incomes and middle and low wage earners.

Traffic congestion and its impact on commercial uses

Transportation improvements have not kept pace with the amount of vehicular traffic on the streets, leading to vehicular traffic congestion in many parts of the Eastern Neighborhoods. While the slow movement of traffic has affected all residents, it has become a serious burden for businesses that rely on their ability to move goods and services quickly and efficiently. The additional transit that has been implemented through MUNI Forward is welcome but not sufficient to serve new growth. There does not seem to be sufficient increase in service to meet the increase in population.

Loss of non-profit and institutional space

There are many reports of non-profits and institutions being forced to relocate due to rent pressures.

Urban Design Policies and Guidelines

While the EN Plans did provide urban design provisions to break up building and provide active frontages, additional urban design controls are warranted. New buildings would be more welcome if they provided more commercial activity at the ground level. Other guidelines should be considered to further break down the massing of new structures.

PROPOSED STRATEGIES TO ADDRESS WHAT'S NOT WORKING:

Retaining PDR:

- Study trends of specific PDR sectors, such as repair and construction to see what is happening to them.
- Implement temporary or permanent relocation assistance programs for displaced PDR tenants through the OEWD.
- Consider implementing programs to transition workers from PDR sectors being lost.
- Potentially preserve additional land for PDR - both inside and outside of the EN (i.e. Bayshore).
- Establish new mechanisms and zoning tools to encourage construction and establishment of new and modern PDR space within the PDR districts.
- The EN Plan should consider making a provision for temporary or permanent relocation assistance for PDR uses displaced by implementation of the EN Plan and/or use impact fees to assist in the acquisition/development of a new creative arts facility similar to other city-sponsored neighborhood arts centers like SOMArts.

Retaining Non-Profit Spaces:

- Study impacts of rent increases on non-profit office space.
- Where preservation/incorporation of PDR uses will be required (i.e. Central Waterfront), consider allowing incorporation of non-profit office as an alternative.
- Consider enacting inclusionary office program for non-profit space, PDR, and similar uses.

Housing

- Consider increases in affordability levels.
- More aggressively pursue purchasing opportunity sites to ensure that they can be preserved for affordable housing before they are bought by market-rate developers.

Infrastructure / Complete Neighborhoods

- Work with Controller’s Office, Capital Planning Office, and the Mayor’s Budget Office to solve the existing known funding gap for EN Infrastructure Projects.
- Deploy impact fees more quickly or find ways to use impact fees to leverage other sources that could be deployed sooner (i.e. bond against revenue stream).
- Consider increasing impact fee levels.
- Increase amount of infrastructure, such as additional parks, given that more development has occurred (and will likely continue to occur) than originally anticipated.
- Study how to bring infrastructure improvements sooner.
- Study new funding strategies (such as an IFD or similar) or other finance mechanisms to supplement impact fees and other finance sources to facilitate the creation of complete neighborhoods, a core objective of the EN Plan.
- Improve the process for in kind agreements.
- Consider allocation of waterfront property to increase the amount of green and open space for use by the general public, as illustrated by the successful implementation in Chicago.
- Review structure of the EN CAC. Consider how the CAC can deploy funds faster. Possibly broaden the role of the CAC to include consideration of creation of complete neighborhoods.
- Consider decreasing the number of members on the EN CAC in order to meet quorum more routinely. Impress on the BOS and the Mayor the importance of timely appointments to the CAC.
- Consider legislation that would enable greater flexibility in spending between infrastructure categories so that funds are not as constrained as they are currently set to be by the Planning Code.
- Explore policies that maximize the utilization of existing and new retail tenant space for neighborhood serving retail, so that they are not kept vacant.

Non EN-EIR Projects

- Encourage the City to take a more holistic expansive approach and analysis that include projects not included in the current EN EIR or the EN Geography.