PROPOSED GOLDEN STATE WARRIORS ARENA
MISSION BAY, BLOCKS 29-32, SAN FRANCISCO
GEOTECHNICAL ENGINEERING REVIEW

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FOUNDATIONS, WALLS, PILES
UNDERPINNING, TIEBACKS
DEEP RETAINED EXCAVATIONS
SHORING & BULKHEADS
EARTHWORK, & SLOPES
CAISSONS, COFFERDAMS
COASTAL & MARINE STRUCTURES

SOIL MECHANICS, GEOLOGY
GROUNDWATER HYDROLOGY
CONCRETE TECHNOLOGY

July 21, 2015

Osha Meserve
Soluri Meserve, A Law Corp.
1010 F Street, Suite 100
Sacramento, CA 95814

Subject: Proposed Golden State Warriors Arena
Mission Bay, Blocks 29-32, San Francisco
Geotechnical Engineering Review

Dear Ms. Meserve:

As authorized, this review is based on information necessary to update a 1998 EIR for a current project proposed within an area bordered by 3rd, South, and 16th Streets, and Terry Francois Boulevard located on Mission Bay fills over Bay Mud. The four blocks are mapped within a seismic hazard area (CDM&G 2000*a*) requiring investigation (CDM&G 2000*b*) and mitigation of potential liquefaction hazards (CGS 2008). The site is also subject to amplification of strong motion due to soft ground (2013 SFBC, ASCE 2013). None of the geotechnical engineering reports for the property classify the site as required by current codes and standards. The data in the existing geotechnical reports underestimates site response to strong motion required for risk to a structure whose primary occupancy will be public assembly with an occupant load greater than 300.

Proposed Project

The project considered, on Blocks 29, 30, 31 and 32, is an event center and parking for the Golden State Warriors basketball team. The project includes two 160 foot office towers, gatehouse, food hall, and retail spaces. 17 years ago an EIR for another project was prepared (C&CSF 1998) based on information for an unspecified location in Mission Bay as no subsurface investigation for the proposed arena site had been undertaken. Later, the four blocks were investigated and reported (Treadwell & Rollo 2007, 2008*a*, 2008*b*) for other projects. Composite reports for four commercial buildings for the four blocks was produced for Alexandraa (Treadwell & Rollo 2008*a*) and salesforce.com (Langan Treadwell Rollo 2011). Subsequent evaluation reports for the arena (LTR 2014*a*, 2014*b*), marked "...privileged...confidential...", have been issued but they do not classify the site nor do they address the Risk III Importance (ASCE 2013, 2013 SFBC) for a known project primarily intended for public assembly. The recent draft EIR (C&CSF 2015) does not address these issues and the current California requirements for mitigation of seismic hazards have not been followed.

Ground Conditions

Several years after the 1998 EIR was prepared, California's seismic hazard mapping program delineated the area of the proposed project (CMD&G 2000*a*) as being subject to liquefaction-induced ground displacement resulting from the shaking of saturated granular sediments that comprise the sands and other artificial fills placed in Mission Bay 100 to 150 years ago.

The property, which was not the subject of a subsurface exploration program when the 1998 EIR was prepared, also includes deposits of Bay Mud of varying thicknesses under the fills that will produce ground amplification from strong motion generated by earthquakes. These hazards are different but related; liquefaction potential (sand) can be mitigated but the structure must be designed to resist soft ground (clay) amplification from strong motion. The data (exploratory boring logs showing materials, sampling, and testing) in the composite reports for the four block area (Treadwell & Rollo 2008a, Langan Treadwell Rollo 2011) verify that both potential hazards exist at the proposed project site.

Seismic Environment

The site is located in the earthquake active San Francisco Bay Area which is seismically dominated by the presence of the San Andreas Fault System. In the theory of plate tectonics, the San Andreas is the boundary between the northward moving Pacific Plate (west of the fault) and North American Plate (east of the fault) which is manifested by the San Andreas system. The faults in the system produce dextral horizontal shear movements resulting from the relative motion of the Pacific and North American plates. Based on history and theory, the land of the proposed project site (sand and rubble fill over Bay Mud)[1] will be subjected to strong shaking from earthquakes generated along both the active San Andreas (8 miles to the west) and Hayward (10 miles to the east) faults.

The northwestward movement of the Pacific Plate relative to the North American Plate persistently causes right-lateral slip across the major faults and deformation between the faults. In the Bay Area, this movement is distributed across a complex system of strike-slip, right lateral parallel and subparallel faults. The San Andreas fault ruptured on 4/18/1906 (estimated M = 8.0) and last severely shook the area on 10/17/89; other earthquakes that epicentered relatively recently along the San Andreas fault occurred on 10/1/69 (Santa Rosa, M = 5.7) and 3/22/57 (Daly City, M = 5.3). Maximum moment magnitudes (scaled size of earthquakes in terms of energy released)[2] are San Andreas $M_W = 7.9$, and Hayward $M_W = 6.9$.

The U. S. Geological Survey forecasted a 67% probability that one or more earthquakes of M = 7.0 (0.20 to 0.45g) or greater will occur on the San Andreas or Hayward faults by the year 2020 (Peterson 1996). Shortly afterwards, the Working Group on California Earthquake Probabilities concluded that the Hayward - Rogers Creek fault system has a 32% probability of generating a large earthquake (M = 6.7 to 7.4) by the year 2030. The average earthquake recurrence interval for the East Bay is roughly 220 years, give or take 100 years As for ground rupturing, there has been a quiescent period of seismic activity after the great 1906 earthquake on the San Andreas fault and there has been no rupturing along the Hayward fault for more than 100 years. The 1998 EIR does not cogently explain the seismic environment of the site.

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[1] A layered sequence of soft, plastic, expansive sediments forming the bottom of San Francisco Bay (often referred to as "Younger Bay Mud"). Bay mud is a very weak, compressible soil, consisting of clay-sized and silt-sized particles interspersed with stringers and pockets of peat, fine sand, and minor amounts of gravel, and having a water content ranging from 30 to 92% (commonly 50 to 60% in the uppermost 50 to 100 feet of the deposit).

[2] The moment magnitude scale is used to measure earthquake magnitude M_W taking into account the size of the fault rupture, the stiffness of rock, and the amount of the movement of the fault using values that can be estimated from the size of several types of seismic waves; while the older Richter scale is a numerical scale used to measure the magnitude M of an earthquake using values based on the size of the earthquake's largest seismic waves.

Research, including trenching by the USGS at the Mira Vista Country Club in the Berkeley Hills, indicates that the northern segment of the Hayward fault is overdue for a characteristic major earthquake (Schwartz & Lettis 1998). On 8/24/14, in not unusual ground conditions, a damaging M = 6.0 earthquake occurred off the northern segment in Napa.

Liquefaction (cyclic mobility, which occurs when loose granular soils that are saturated undergo a rapid loss in shear strength as a consequence of ground shaking), and movement amplification of the Bay Mud due to strong motion, will occur at the proposed project site (and nearby sites) during significant earthquakes. This is the reason why California mapped the seismic hazard zones in the state in 2000 and requires mitigation of the seismic hazards.

Ground Motion Parameters

The National Earthquake Hazards Reduction Program ("2009 NEHRP") document "Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" (FEMA 450-1) feeds into the ASCE (American Society of Civil Engineers) 7-10 "Minimum Design Loads for Buildings & Other Structures" (ASCE 2013) development process, and ASCE 7 in turn serves as the primary referenced standard in the 2012 International Building Code (2012 IBC). The 2013 San Francisco Building Code (2013 SFBC) is the City's iteration and adoption of the 2013 California Building Code, which is the State's iteration and adoption of the 2012 IBC. At the time the 1998 EIR was written the San Francisco Building Code was based on superficial maps in the Uniform Building Code (ICBO 1998) when seismic design standards were much less stringent than those of today.

Ground motion parameters, for this review of data in reports of subsurface investigation for the project site, all of which were gathered and presented after the 1998 EIR, were determined for the site using USGS ASCE 7 (2013) based calculation tools derived from published ground motion maps. Seismic ground motion values for use in characterizing and classifying the site for the current project are as follows:

General:

Site Location (USGS):	Latitude 37.7678°N Longitude -122.3875°W
Risk Category (2013 SFBC Table 1604.5)[3]:	III
Seismic Importance Factor I_e (ASCE 7 Table 1.5-2):	1.25

Mapped Acceleration Parameters (2013 CBC §1613.3.1):

Determination of Maximum Considered Earthquake (MCE) spectral response accelerations, mapped at short (0.2 second) period S_s and at a full second (1.0 second) period S_1, for the site:

Determined Site Classification (input Latitude/Longitude):	E
Short period (0.20 second) mapped spectral acceleration S_s:	1.500g
Site Coefficient F_a (2013 SFBC Table 1613.3.3(1); function/Site Class E & S_s):	0.900
Adjusted MCE 0.20 second period spectral response acceleration $S_{MS\text{-}B} = F_a S_s$:	1.350g

[3] "Buildings and other structures that represent a substantial hazard to human life in the event of failure."

One second period mapped spectral acceleration S_1:	0.600g
Site Coefficient F_v (2013 SFBC Table 1613.3.3(2); function/Site Class E & S_1):	2.400
Adjusted MCE one second period spectral response acceleration $S_{M1\text{-}B} = F_v S_1$:	1.440g

Design Spectral Response Acceleration Parameters (2013 SFBC §1613.3.3):

Site Classification definitions are dependent on geotechnical data (2013 SFBC §1613.2.1; ASCE 7 §§20.3.2, 20.3.3(3) [softer soil category to be used due to differing criteria][4]).

Defined Site Classification (2013 SFBC §1613.3.2 & ASCE 7 Table 20.3-1):	E
Site Coefficient F_a (2013 SFBC Table 1613.3.3(1); function/Site Class E & S_s):	0.900
Adjusted MCE 0.20 second period spectral response acceleration $S_{MS\text{-}D} = F_a S_s$:	1.350g
5% damped short period design spectral acceleration $S_{DS} = 0.67 S_{MS\text{-}D} = 0.67(1.350)$:	0.905g
Site Coefficient F_v (2013 SFBC Table 1613.3.3(2); function/Site Class E & S_1):	2.400
Adjusted MCE one second period spectral response acceleration $S_{M1\text{-}D} = F_v S_1$:	1.440g
5% damped one sec. period design spectral acceleration $S_{D1} = 0.67 S_{M1\text{-}D} = 0.67(1.440)$:	0.965g

Seismic Design Categories (SDC); Risk Category III, $S_1 \geq 0.75$ (2013 SFBC §1613.3.5, ASCE 7 §11.6):

Determination of Seismic Design Category (SDC) is based on occupancy or use and level of expected soil/rock-modified seismic ground motion at the site (adjusted per ASCE 7 §11.6).

Short period response acceleration SDC_{DS} (2013 SFBC Table 1613.3.5(1) adjusted):	E
One second period response accel. SDC_{D1} (2013 SFBC Table 1613.3.5(2) adjusted):	E

Mapped MCE Geometric Mean Peak Ground Acceleration PGA (ASCE 7 §11.8.3, 2013 SFBC §1805.5.12(2)):

PGA (USGS output):	0.523
Site Coefficient F_{PGA} (Site Class E, ASCE Table 11.8-1, PGA ≥ 0.50):	0.900
Peak Ground Acceleration adjusted for site class effects $PGA_M = F_{PGA} PGA$:	0.471g

The above ground motion parameters, reporting just recently required per ASCE 7 (ASCE 2013) where applicable under 2013 SFBC §1805.5.12, and calculated for a structure having an occupant load greater than 300, must be used for analysis in a new EIR. Lateral force resisting systems must meet seismic detailing requirements and limitations set forth in ASCE 7 (2013 SFBC §1604.10).

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[4] Langan Treadwell Rollo 2011 (ASCE 7 Table 20.3-1):

B 29-8 8/31/11 Bay Mud, soft-wet 12-35' (21'>10')
B 32-1 5/1/07 Bay Mud, soft-wet 11-42' (31'>10'), MC=57% (>40%)
B 30-4 5/5/07 Bay Mud, soft-wet 25-50' (25'>10'), MC=63-74% (>40%)
B 31-4 9/1/11 Bay Mud, soft-wet 12-35' (23'>10'), s_u=400 psf (<500 psf)

Treadwell & Rollo 2008*a* (ASCE 7 Table 20.3-1):

1030 (AGS) 3/1/00 Bay Mud, moist-soft 22-51' (29'>10'), PI=58% (>20%)
1031 (AGS) 2/29/00 Bay Mud, moist-soft 16-55' (39'>10'), PI=38-62% (>20%)

Mitigation of Seismic Hazards

California's Special Publication 117A (CDM&G 2008) mandates countermeasures to liquefaction because liquefaction has been a major source of damage during past earthquakes where deposits of saturated sands were present. The risk of liquefaction and associated ground deformation can be reduced by various ground-improvement techniques, but consideration of also lessening the effects of strong motion in the underlying Bay Mud (from transient porewater pressure increases) during earthquakes must also be part of mitigation. The EIR of 17 years ago (C&CSF 1998) contains no mitigation measures, and the newest draft EIR (C&CSF 2015) does not include sufficient countermeasures.

The latest composite report for the site (Langan Treadwell Rollo 2011) anticipated four buildings. Alternative mitigation measures were recommended in the report for those buildings including "rapid impact compaction" ("RIC") "stone columns" and "compaction grouting". A more appropriate countermeasure, deep soil mixing of slurry at depth, has been suggested (Langan Treadwell Rollo 2014*a*). Gravel drains in backfilled bored holes to dissipate pore pressures are an effective countermeasure to liquefaction (Seed & Booker 1977). However, the proposed arena would probably be supported by piles arranged in concentric circular or elliptical patterns, and those piles will be subject to not only liquefaction loads from saturated relatively loose granular materials in the sand and rubble fill but from strong motion amplification of the relatively soft cohesive materials of the Bay Mud.

By embedding the piles into a mat capping the piles, and strengthening the liquefiable sand in the fill (not by "compaction grouting" but by permeation grouting using microfine cement or Portland cement slurry mixed with the sand), and socketing the piles into the Colma (or bedrock near the south end of the site), the effective length of the prestressed concrete piles will be reduced considerably by fixing end conditions and shortening the effective lengths of piles within the Bay Mud. The undersigned believes a program of combination of techniques should be modeled and tested before project approval.

Arena Foundation System

The latest composite report for the site (Langan Treadwell Rollo 2011) was for four separate buildings, one on each of the four lots. The proposed arena (Langan Treadwell Rollo 2014*a*) will be the principal structure in a complex that includes other structures. The 2011 report provides foundation alternatives for each building mainly because the Colma formation (dense to very dense sand, silty sand, clayey sand) is thin at the southeastern part of the site. Structural steel piles should not be used as the Bay Mud is highly corrosive and cathodic protection systems are problematical (Karp 1977).

If the proposed arena project were to proceed, it is more than likely that the foundation system, arranged in a pattern of concentric circles or ellipses. would be comprised of either precast prestressed concrete piles or cast-in-place concrete piles that are drilled through casing that is part of the machinery with the piles concreted as the casing is withdrawn. Piles would derive their support from the Colma formation, except at the southern part of the site bedrock would be the supporting medium. For embedment in the Colma formation or very stiff to hard clay and bedrock where the Colma formation is not present, depth-limited augered piles could penetrate dense materials or precast prestressed concrete piles could be driven with steel stingers and where the Colma formation is not present, the piles could be piloted into the very stiff to hard clay or bedrock. Although various deep foundation alternatives are theoretically possible, the proposed current project, which is particularly sensitive due to its public assembly nature, should have a testing program instituted to test alternatives.

Vibrations During Construction

Driving displacement piles causes noise and vibrations from impact that are transferred though dense subgrade materials to nearby structures. As the configuration of the proposed arena will likely be circular or elliptical and vibrations, particularly driving those at the western side of the project, would likely affect the UCSF Medical Center building at 1650 3rd Street. Prior to project approval, an indicator pile test program must be implemented to monitor vibrations and verify the suitability of the intended foundation system for the area.

Drilling and casting-in-place reinforced concrete shafts, if feasible to required depths, may be an appropriate suitable alternative to driven piles. As noted below for shoring, shafts are augered and spoils removed through casing contained in the rig that is withdrawn as concrete is placed. Using tremie methods, concrete displaces water in the hole so it rises and is pumped out with low groundwater loss. Before the project is approved, a test program should be implemented to ascertain the feasibility of using cast-in-place piles or where appropriate, a combination of drilled and driven piles.

Shoring & Groundwater

As an underground parking garage would be part of the project, secant piles, drilled in a circular or elliptical pattern to form a tension ring, would likely be the shoring, but drilling/concreting operations will encounter and displace groundwater that would have to be continuously tested for contaminants and otherwise managed under an advance plan. A Memorandum (Langan Treadwell Rollo 2015) suggests "Construction Dewatering Discharge Options" which may be helpful for that problem but the actual engineering effects of dewatering (increase in effective stress that causes areal subsidence) was not addressed. The effects upon surface improvements from dewatering in the area of the project must be studied before project approval.

Shoring of the excavations for the intended subgrade portions of the proposed current project, the appropriate method would be, as noted above, secant piles. Secant piles are sequentially drilled shafts that intersect each other to form a solid wall. Primaries (soft piles) are drilled apart in rows (or curves) closer together than the pile diameter. Primary shafts are augered and spoils removed with low water loss. Secondary shafts (hard piles) are augered between and arched into both of adjacent primaries, and wet-set reinforced with steel. In the saturated sand, it would be at this stage (casing/augering, and reinforcing) and afterwards (tolerance deviation from verticality, joints between overlapping piles, and movement) when groundwater and sand will be lost.

Depending on depth below groundwater level, hydrostatic pressures (head) are about one-half psi which will allow water and sand to migrate into the excavation. Pressure is only reduced if groundwater level drops outside the wall. When water is lost, increases in effective stress with vibrations from hard pile installations will densify the sand with differential settlement of improvements. The only methods to minimize water and sand flowing into the excavation with simultaneous drawdown of the groundwater level is to recharge outside the wall or construct the shoring in a circular pattern with large overlaps acting in ring compression.

Under current codes and standards, below grade walls for the proposed underground structures will require dynamic analysis (2013 SFBC §1803.5.12(1)) as well as engineered design to protect surface improvements, wall backdrainage, groundwater collection, piping, and discharge facilities.

Contamination

Although it is understood that others will discuss contamination, the subject is a very important environmental and geotechnical engineering concern for reasons that include intended subgrade excavation and construction. Mission Bay was used for many years as a dump and then a railroad yard. Bayward of the site there were fuel terminals that included tanks and pipelines which are known contributors to contamination. The Pier 64 area has received past attention under the auspices of developers (Langan Treadwell Rollo 2014*b*) but the extent and sufficiency of actual clean-up is not really known from second hand information. The report of geotechnical investigation produced for salesforce.com (Langan Treadwell Rollo 2011), 327 pages, contains no contaminant sampling, testing, or even recognition of the potential problem.

Contamination seems to have been dismissed as a thing of the past, but contaminants in groundwater do not simply go away without complete ground remediation. The 1998 environmental document is vague so "change" from then to now cannot be quantified. For instance, the "2001 Phase I Remedial Excavation" resulted in a record that "Soil containing residual oil below the target zone was left in place." (Langan Treadwell Rollo 2014*b*, pg 9). The observance of living birds congregating where water has ponded is not a reliable yardstick for declaring a site free of contamination. Hands-on testing by an independent laboratory would be appropriate measures that should be undertaken before a public assembly project at this site is approved.

Yours truly,

Lawrence B. Karp

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