**Administrative Bulletin**

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| **SUBJECT** | **:** | Permit Processing and Issuance |
| **DATE** | **:** | June 15, 2020 |
| **TITLE** | **:** | **Guidelines for Preparation of Geotechnical and Earthquake Ground Motion Reports for Foundation Design and Construction of Tall Buildings** |

**PURPOSE :** The purpose of this Administrative Bulletin is to present requirements and guidelines for developing geotechnical site investigations and preparing geotechnical reports for the foundation design and construction of tall buildings.

**REFERENCES :** 2019 San Francisco Building Code (SFBC)

Administrative Bulletin AB-082: Guidelines and Procedures for Structural, Geotechnical, and Seismic Hazard Engineering Design Review

CCSF (2014) – Guidance for Incorporating Sea Level Rise into Capital Planning In San Francisco: Assessing Vulnerability and Risk to Support Adaptation.

CCSF (1206) – San Francisco Sea Level Rise Action Plan.

NRC (2012) – Sea Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future.

NIST / NEHRP (2012) – Soil-Structure Interaction for Building Structures, GCR 12-917-21.

PEER (2017) – Tall Buildings Initiative, Guidelines for Performance-Based Seismic Design of Tall Buildings, Version 2.01, PEER Report No. 2017/06, May.

Poulos, H.G. 2017 – Tall Building Foundation Design, publishing house CRC Press, ISBN 9781138748033.

Ellen Plane, Kristina Hill, and Christine May “A Rapid Assessment Method to Identify Potential Groundwater Flooding Hotspots as Sea Levels Rise in Coastal Cities,” October 25, 2019

K. Yasuhara; S. Murakami; N. Mimura; H. Komine; and J. Recio, “Influence of global warming on coastal infrastructural instability,” December 2006

Many relevant and useful references are provided in the following document: ATC 119 (2019) – Seismic Safety and Engineering Consulting Services for the Earthquake Safety Implementation Program (ESIP), City and County of San Francisco, 2019.

**DISCUSSION :**

### *1. SCOPE OF THIS BULLETIN*

This bulletin presents guidelines for developing a geotechnical site-investigation program and preparing geotechnical reports for foundation design and construction of tall buildings in San Francisco. Sections 2 and 3 of this bulletin are requirements and therefore are stated in mandatory language. The remaining sections are guidelines, which use non-mandatory language.

For the purposes of this bulletin, tall buildings are defined as those with $h_n$ (ASCE 7), greater than 240 feet.

The height, $h_n$, is defined in the San Francisco Building Code (SFBC) as the height of level *n* above the average level of the ground surface adjacent to the structure. Level *n* is permitted to be taken as the roof of the structure, excluding mechanical penthouses and other projections above the roof whose mass is small compared with the mass of the roof.

Early in a project, the Geotechnical Engineer of Record (GEOR) shall develop a geotechnical site-investigation program and geotechnical report document in accordance with this bulletin.

### *2. GEOTECHNICAL DESIGN REVIEW*

The review of geotechnical design shall meet the requirements of AB-082. The geotechnical member(s) of the Engineering Design Review Team (EDRT) shall participate in the Early Site Permit phase of the project to review the GEOR's plan for geotechnical site investigations and the GEOR's geotechnical basis-of-design document. During the subsequent design review, the EDRT will use the guidelines below to review the geotechnical report prepared for foundation design and construction.

At the conclusion of the review, the geotechnical members of the EDRT shall provide a written statement that, in their professional opinion, the geotechnical site-investigation plan and geotechnical reports meet the requirements of the SFBC and this bulletin.

> **Commentary:** The Draft of this bulletin was developed by a volunteer group of experienced geotechnical engineers as an ad-hoc committee of the Structural Engineers Association of Northern California (SEAONC). The draft was requested of SEAONC by SFDBI. Subsequently, the draft of this bulletin was processed (and in some places revised) through subcommittees of the Building Inspection Commission according to the Administrative Bulletin process.

### *3. SUBMITTAL REQUIREMENTS*

Project submittal documents shall be in accordance with the SFBC and Department of Building Inspection (DBI) interpretations, Administrative Bulletins, and policies. In addition, documents relevant to the Geotechnical Design Review shall be submitted by the Engineer of Record to the Director and to the geotechnical members of the EDRT.

### *4. PROJECT DEFINITION AND DESIGN CRITERIA*

In coordination with the project architect and structural engineer, the following information (if available at the time of preparation of the geotechnical report) should be provided: The project description; a site location map; height of the structure; number of stories; number of basement levels; lateral and gravity loads resisting systems; anticipated gravity foundation loads or bearing pressures; applicable codes and design guidelines for seismic design of the building (e.g., PEER TBI 2017 performance-based design of tall buildings); description of the energy dissipation system (if used); and the approach for development of design ground motions.

### *5. SITE SURFACE CONDITIONS*

Description of existing structure(s) on the site should be presented with information related to the foundations (if known); the site's historical and current use; site surface elevation including, the reference datum; and description of adjacent facilities and structures with information related to their foundation system (if known) within the foundation zone of influence. The GEOR should determine the foundation zone of influence based on site's subsurface conditions, foundation type, and building configuration.

> **Commentary:** For a mat foundation bearing on the Colma sand layer, the lateral extent of the zone of influence could be estimated as approximately ½ of distance between the base of the mat and bedrock.

### *6. REGIONAL AND LOCAL GEOLOGY*

This section should include a description of regional and local geology, including fill placement as part of land reclamation, if any. The description of local site geology should provide information about the anticipated engineering soil and rock properties likely to be encountered. Hazard maps and information from the USGS and the State should also be presented including anticipated sea level rise during the design life of the structure (e.g., NRC 2012, CCSF 2014, and CCSF 2016), seismic ground motion, soil liquefaction and lateral spreading, landslides, and tsunami/seiche (for sites near the shoreline).

### *7. SEISMICITY*

A fault map should be provided showing the location of Holocene active faults within a 100 km radius of the site, with the epicenter and magnitude of historical earthquake events shown on the map. A table should be provided containing the pertinent fault information for sources that contribute significantly to the probabilistic seismic hazard analysis (PSHA) performed for a return period of 2,475 years at the key periods of interest to the building design using Uniform California Earthquake Rupture Forecast (UCERF) fault data file.

**Commentary:** The version of UCERF fault data file that is referenced by the latest California Building Code (CBC)/SFBC and ASCE 7 Standard should be identified and used. In current practice, when performing deterministic seismic hazards analysis (DSHA), maximum fault magnitudes are obtained either from UCERF2 fault data file or mean/mode magnitude from deaggregation of 2,475-year PSHA results.

### *8. FIELD INVESTIGATION AND LABORATORY TESTING*

The subsurface conditions should be explored by drilling borings, and if appropriate, conducting cone penetration test (CPT) soundings. When considering the plan area of the proposed development and the magnitude of building loads, the number of borings and CPTs should be sufficient for characterizing the site's subsurface conditions and physical properties of soils and bedrock encountered.

The quality of samples should be appropriate for the anticipated laboratory strength or compressibility tests conducted to obtain load-deformation characteristics of soil in support of advanced numerical modeling.

**Commentary:** Integrated field and laboratory tests should be performed as appropriate to support the anticipated methods of analysis, which commonly include standard general limit equilibrium (GLE) methods and 2D or 3D nonlinear seismic soil-structure interaction (SSI) analyses. Historically, the selection of soil properties for static and seismic design of building foundations has been accomplished through parameter correlations with field tests such as the CPT and field vane shear test (FVST). Correlations with the results of field and soil-index tests are useful; however, it is recommended that relationships used in support of tall-building design in San Francisco be checked against local geotechnical data and adjusted, if need be, to provide representative properties of local soils. With the evolution and widespread adoption of performance-based seismic design for tall buildings, advanced numerical analyses incorporating soil-structure interaction (SSI) may be performed. Appropriate SSI analyses require substantial characterization of soil behavior such as strain-dependent shear modulus and material damping curves, and residual shear strength. The use of field test data should be supplemented with laboratory tests that provide soil parameters across the range of deformation anticipated for the project. Strain-dependent soil parameters must also account for the rate effects and the potential for cyclic degradation of soil stiffness and strength. Laboratory tests on soil, such as cyclic direct simple shear and cyclic triaxial, can provide insight into the soil behavior during seismic loading. The integration of suitability extensive field and laboratory test programs improves the reliability of site characterization, thereby reducing uncertainty.

Information and data from existing geotechnical borings and CPTs could be used to supplement new borings as long as existing geotechnical borings and CPTs are located reasonably close to the project site and are drilled in accordance with currently acceptable methods and standards. However, borings drilled only for environmental soil and/or groundwater sampling and testing or for water wells should not be used as a substitute for project-specific geotechnical borings or CPTs.

For sites with depth-to-bedrock of more than 100 feet, at least one boring should extend a minimum of 50 feet below the surface of bedrock; other borings should be as deep as deemed appropriate as determined by the GEOR and reviewed by the geotechnical members of the EDRT, based on the site's subsurface conditions, structural loads, and below-grade structural geometry.

For depth-to-bedrock of less than 100 feet, all borings should extend to the bedrock surface with one boring extending at least 50 feet below bedrock surface.

If used, CPTs should be pushed to refusal using a 20-ton CPT rig, if it is possible to access the site with it. At least one CPT sounding should be near a geotechnical boring for calibration purposes. If site conditions prohibit access for a CPT rig within the site, additional CPTs and/or borings adjacent to the site may be necessary and may be required by the EDRT.

**Commentary:** Field vane shear tests (FVST) are useful for evaluating the peak and remolded undrained shear strength of soft clay. For evaluation of soil liquefaction potential, lateral spreading, and slope instability adjacent to the site, it is suggested that CPT soundings be performed as much as practical because they provide continuous, reliable measurements that can be correlated to physical soil properties. CPTs are also useful for characterizing denser and stiffer units, such as Old Bay Clay (OBC) and for characterizing groundwater conditions with a pore pressure dissipation test. However, because liquefiable and soft soils are bypassed by using deep foundations or by using ground improvement to provide appropriate bearing support for building foundation, CPT soundings are of limited use under the building footprint because CPTs will most likely encounter refusal within the dense sand layer present at many sites in San Francisco. A sufficient number of borings should be drilled for adequate sampling within the OBC layer. Typically, at least one boring should be drilled or one CPT sounding should be performed in every 5,000 square feet of plot area.

Shear-wave velocity should be measured at least at one location using downhole techniques, seismic CPT, suspension logging, or surface-wave method, as appropriate. The number of tests should reflect the lateral variability of the soil deposits across the site. The shear-wave-velocity measurement should be conducted in such a manner as to allow for accurate determination of variation of shear-wave velocity with depth for computing the Vs30 parameter and for conducting site response analysis (if performed). If downhole logging is used, the shear-wave velocity of bedrock should be measured within the boring that extends at least 50 feet below the surface of bedrock.

To capture the variability in groundwater conditions over time, at least one piezometer should be installed, and piezometric levels should be observed from the time of original geotechnical exploration. In some cases, additional piezometers may be necessary and may be required by the EDRT.

Soil borings should be drilled using rotary wash drilling methods (unless the groundwater table is below the bottom of the boring). Drilling fluid or casing should be used to prevent collapse of borings and bottom instability.

Where compressibility and strength tests are planned in soft clays (e.g., Bay mud - BM), samples should be obtained using a thin-walled tube sampler.

**Commentary:** Osterberg-type hydraulic fixed-piston sampler with thin-wall tubes of constant inside diameter can provide high-quality samples.

In stiff clays (e.g., OBC) where strength and consolidation tests are planned, Pitcher Barrel sampler or approved equivalent should be used.

Standard penetration tests (SPT) should be performed in cohesionless soils. California modified sampler or Sprague and Henwood (S&H) sampler may be used in the alluvium often found between the bottom of OBC and bedrock and where strength and compressibility tests are not required. Hammer energy measurements should be performed for drive sample system (e.g., SPT and S&H) on at least one boring for the project.

**Commentary:** Pressuremeter test results have been successfully correlated with large strain modulus of various geological units in the east coast of the United States and overseas.

Rock coring should be used to obtain rock cores within bedrock for borings that extend at least 50 feet into rock. Rock cores should be reviewed and classified by a registered professional geologist. Parameters defining degree of rock weathering, rock strength, rock hardness, and rock mass properties such as the RQD, spacing of discontinuities, conditions of discontinuities, and dip angle should be recorded as directed by the GEOR.

For all soil types, sample intervals should be no greater than 5 feet or at layer interface unless a larger interval is deemed appropriate by the GEOR based on thickness and uniformity of soil layer, data from field vane tests or CPT soundings.

For sandy soils, one or more of the following laboratory tests, as deemed appropriate by GEOR, should be conducted: moisture-density (if S&H sampler is used), moisture test (if SPT sampler is used), fines content (minus sieve No. 200), sieve analysis, and plastic and liquid limits (if silty or clayey sand).

For cohesive soils, one or more of the following tests, as deemed appropriate by the GEOR, should be conducted: (1) unconsolidated or consolidated undrained triaxial tests, or (2) a direct simple shear test. Unconfined compressive strength may be used on representative rock samples but should not be used for cohesive soils.

The GEOR should determine the adequate number of pairs of consolidation and undrained shear strength tests to be performed on undisturbed samples of OBC for evaluation of settlement if a mat foundation is not supported by a deep foundation and is placed above the surface of OBC or if the foundation bears above or within OBC. One pair of consolidation and undrained shear strength tests should be considered for every 30 feet of OBC depth in four representative borings, unless the variability of the site is evaluated through CPTs. The minimum number of pairs should be four. Additional tests would be required if the preconsolidation stress is exceeded.

If OBC is expected to be subjected to vertical effective stresses higher than the preconsolidation pressure, additional tests are also required to measure the secondary consolidation characteristics of the OBC.

Field “index” tests such as the Pocket Penetrometer or Torvane tests may be used on clayey soil samples but should not be considered as a substitute for any laboratory tests described above.

## *9. SUBSURFACE CONDITIONS*

At least two perpendicular cross sections should be provided. A full description of soil layers and geologic units with engineering properties (consistency and consolidation characteristic for clayey soils and potential for soil liquefaction and settlement for sand layers) should be provided.

A design groundwater elevation with consideration of sea level rise during the design life of the structure and seasonal fluctuation of groundwater level (if known) should be presented. The groundwater table expected to be encountered during construction should also be identified.

**Commentary:** The GEOR should use her/his judgement as to how far inland the influence of sea level rise would impact the groundwater level.

## *10. FOUNDATION AND GEOTECHNICAL EARTHQUAKE ENGINEERING STUDIES*

### 10.1 Code-Based Site Classification

The Site Class designation should be made following the current edition of the applicable code and standard (e.g., ASCE 7, SFBC). The Site Class definitions should be based on $V_{s30}$ and presence of soft clay or liquefiable soils. According to the code-based Site Class designation, $V_{s30}$ is defined in the free field from the ground surface to the depth of 30 m (100 ft). However, Site Class may be defined below the bottom of the mat foundation (see Section 10.2.2 Ground Motion Characterization Commentary), if deemed appropriate.

## 10.2 Ground Motion and Seismic Ground Deformation Characterization

The regional seismic hazard assessment and ground-motion characterization should follow the procedures provided in applicable seismic guidelines and code provisions (e.g., PEER TBI 2017, ASCE 7). These procedures include the application of Probabilistic and Deterministic Seismic Hazard Analyses (PSHA, DSHA) incorporating specific seismic source models (e.g., UCERF, USGS NSHMP 2014 or 2018) and ground motion models (GMMs). The GEOR may use updated, widely adopted models in PSHA and DSHA in site-specific analysis. The ground-motion characterization should address pertinent issues such as near-fault effects, basin effects, and dynamic soil response (site effects). Embedment and base averaging effects may be accounted for, as applicable.

The selection and modification of ground motions (acceleration time series) should be consistent with recommendations found in the applicable codes and standards.

The subsequent sections address ground-motion characterization at the surface and at depth.

### 10.2.1 Ground-Motion Characterization at Surface

- For Site Classes A, B, and C, the ground-motion development may be based on $V_{s30}$ measured from the ground surface using ground motion models (GMMs). The resulting ground-surface acceleration response spectra ($MCE_R$ and DE) should be checked against minimum code requirements.
- For Site Class D determined based on $V_{s30}$ measured from the ground surface, the ground motion may be developed using site response analyses or GMM's, as determined by the GEOR and approved by the geotechnical members of the EDRT. The resulting acceleration response spectra should be checked against the minimum code requirements. Consideration of site response analysis is warranted because of the breadth of the $V_{s30}$ values defining Site Class D soil profiles (i.e., $V_{s30}$ of 600 to 1,200 ft/sec) and the range in anticipated ground-surface motions for the wide variety of soil conditions represented by Site Class D sites.

  Other factors influencing the decision to perform a site response analysis include: (1) depth to material with shear wave velocity equal to or greater than 1,200 ft/s, (2) depth to bedrock defined as the Site Class B/C boundary (2,500 ft/sec), and (3) the trend of site-specific $V_s$ with depth (i.e., the site period).
- For Site Classes E and F, site response analysis using methods suitably calibrated by the GEOR should be performed and the design spectrum calculated at the ground surface should be in conformance with the applicable Building Code requirements. For sites where (1) surficial soil (e.g., liquefiable fill and soft Bay mud) are removed through basement excavation and foundation installation or (2) ground improvement is used to bypass liquefiable or soft soil, the GEOR should evaluate whether the site could be reclassified as site class D with concurrence with the geotechnical members of the EDRT.

  The number and characteristics of ground motions, variation in shear-wave velocity profile, and variation in soil shear modulus reduction and material damping curves used in site response analysis should be adequate to capture the potential variation in surface ground motion in a realistic and defensible manner.

  The following procedure is suggested for consideration by the GEOR:

  After a thorough review of site-specific geotechnical and geophysical data, evaluate the applicability of GMMs (i.e., $V_{s30}$-based estimation) for approximating the dynamic response of the soil profile.

  If site-specific aspects of the soil profile are not reasonably approximated by the "average, characteristic $V_s$ profile" implied by the GMMs, ground response analysis should be considered. The ground-surface motions developed through ground response analysis should be checked against minimum code requirements.

**Ground Motion Characterization Commentary:** The level of analysis required for establishing surface, or near-surface, ground motions should reflect site-specific factors such as stratigraphy, geotechnical characteristics and properties of the soils, depth to bedrock, the trend of Vs from the ground surface to competent bedrock, and the amplitude of the bedrock motions (e.g., $MCE_R$, DE). Methods of analysis can be generalized as consisting of (1) numerical dynamic site response analyses, (2) estimation using current GMMs that include regression terms for $V_{s30}$ (e.g., NGA-West2 GMMs), and (3) simplified, code-based site class designation and site coefficients ($F_{pga}$, $F_a$, and $F_v$), which are required as a check on the ground motions developed using methods 1 or 2. The applicability and suitability of site response analysis and GMMs for the development of design-level ground surface motions should be evaluated prior to adoption on a project-specific basis for all Class D sites.

The potential range of representative ground-surface motions anticipated at Class D sites due to the inherent variability of subsurface conditions and dynamic response of soil profiles falling under this general $V_{s30}$-based classification in the San Francisco Bay Area necessitates critical evaluation of the procedures applied for developing design ground motions. It is suggested that the GEOR engage the geotechnical members of the EDRT as soon as practical after pertinent site-specific geotechnical and geophysical data have been collected to identify the appropriate method of developing ground-surface motions prior to analysis. The following suggestions are deemed pertinent to local practice and provided for demonstration and guidance.

For sites containing soft-to-medium stiff fine-grained soils (e.g., BM), numerical ground response analysis is preferred and considered the primary method for developing ground-surface motions. This suggestion also applies to sites with lower $V_{s30}$ (600 ft/s to 900 ft/s). In this situation, methods 2 and 3 are performed as checks on the results of the numerical site response analyses.

For stiffer soil profiles with $V_{s30}$ in the upper range of the Site Class D category (900 ft/s to 1,200 ft/s), methods 2 and 3 may be acceptable for characterizing ground motion.

Many sites in the San Francisco Bay Area are underlain by dense sand and stiff clays that contribute to $V_{s30}$ values in the range of 1,000 to 1,200 ft/s. The development of ground motion for these sites may be based on site-specific $V_{s30}$ measured from the ground surface using GMMs.

**Ground Response Commentary:** Dynamic ground response analyses are routinely performed in practice using equivalent-linear and nonlinear models. The strengths and limitations of both methods of analysis have been addressed in the technical literature, and one of the primary differences in the two approaches is simulation of moderate- to large-strain behavior in cyclic loading. The combination of soft or medium stiff soil (i.e. BM or other marine deposits) and liquefiable sands that are prevalent in San Francisco, and the strength of design-level cyclic loading leads to highly nonlinear soil behavior. Therefore, nonlinear models that have been suitably calibrated are preferred over the equivalent linear model; however, equivalent linear site response analysis results are often used for comparison with nonlinear site response analysis results. Numerous computer programs have been used to perform nonlinear site response analysis on local projects. The GEOR may select the preferred model for the project. It is suggested that the GEOR provide documentation supporting calibration of the proposed model for analysis of similar soil profiles subjected to ground motions that are similar in nature to the design-level motions required for the project. Irrespective of the model used on the project, the results of the dynamic response analysis should be reviewed by the geotechnical members of the EDRT.

The slope of bedrock in the vicinity of the site should be evaluated and the GEOR, with approval from geotechnical members of the EDRT, should determine whether a two-dimensional site response analysis is required.

For sites at which lateral and vertical variability of the soil profile and depth to bedrock is significant enough to result in dual Site Class designations, two-dimensional or three-dimensional site response analysis may be required to develop an appropriate ground motions for design. The required check against code-based ground motions should be provided for both Site Classes, and the proposed design motions presented to the geotechnical members of the EDRT for review.

### 10.2.2 Site Response and Ground Motion Characterization at Depth of Interest

Time series selected and modified by the GEOR for use in structural dynamic analyses by the structural engineer of record (SEOR) should be representative of the ground motions at the depth of interest for the structural model. The depth of interest is a function of the modeling approach implemented by the SEOR. Primary considerations for the ground motions used in dynamic structural analyses are well presented in numerous documents (e.g., NEHRP 2015, NIST 2011, NIST 2012). In most cases, ground surface motions should not be used in structural models for buildings with multiple basement levels. Therefore, the acceleration response spectrum used as the basis for modification of time series should be developed using either (1) calibrated ground response analysis allowing development of the acceleration response spectrum at the appropriate depth, or (2) validated simplified methods that account for foundation embedment effects. The latter would be required, for example, on projects for which the ground surface motions were developed using GMM's and trends in the motions with depth are not provided.

The design team, with review by the geotechnical members of the EDRT, should determine whether ground response analysis should be performed using ground motions corresponding to $MCE_R$, DE (or both), and possibly the Serviceability Level earthquake (SLE).

For sites where (1) surficial soils (e.g., liquefiable fill and BM) are removed through basement excavation or foundation installation, or (2) ground improvement is used to bypass liquefiable and soft soil, the GEOR, with concurrence of the geotechnical members of the EDRT, should evaluate whether the site could be reclassified for the sake of ground-motion comparison to code-based requirements based on a representative 30 m (100 ft) time-averaged interval velocity that is computed using site-specific Vs data over a depth range deemed appropriate for configuration of the basement, foundation, or ground treatment.

**Ground Motion Characterization Commentary:** For a surface foundation, the energy transmitted to the structure is applied through soil in contact with the base of the foundation. For embedded structures, the basement walls may be in contact with liquefiable soil or soft clayey soil over a certain depth and then in contact with competent soil down to the lowest elevation of the basement walls. In this case, the presence of soft or liquefiable soil may be ignored and $V_{s30}$ could be evaluated from the surface of competent soils. The rationale behind this is; while ground motion within soft or liquefiable soil may be higher than ground motion within the competent soils, the energy transmitted to the structure from these layers is relatively small due to their low stiffness (i.e., the product of ground-motion intensity and soil stiffness controls the amount of energy transmitted to the structure from each layer). However, seismic earth pressures should consider the effects of soft soil against basement walls.

### 10.2.3 Kinematic Soil-Structure Interaction (KSSI)

KSSI analysis may be performed using (1) simplified methods accounting for base averaging and embedment effects (e.g., NIST 2012), or (2) finite element or finite difference kinematic SSI analysis. It should be noted that the provisions of ASCE 7-16 (Chapter 19) provide a maximum allowable reduction of ground motion due to combined (base averaging and embedment) kinematic SSI effects when performing nonlinear response history analyses. Per ASCE 7-16 Section (19.2.3), the site-specific response spectrum modified for kinematic SSI shall not be less than 70% of $S_a$ as determined from the design response spectrum and $MCE_R$ response spectrum motions developed using the code-based approaches. When using the simplified method (Chapter 19, ASCE7-16) for evaluation of ground motion with embedment effects, the $V_{s30}$ computed from the ground surface (as opposed to from the bottom of the basement) should be used. To compute the ground motion reduction due to embedment effects, the average shear wave velocity over the height of the basement should be used.

If finite element or finite difference kinematic SSI analysis is performed (1) the ground motion near the boundary of the model should be similar to those obtained from one dimensional site response analysis, and (2) kinematic ground motion should meet ASCE 7 requirements.

**Commentary:** If soil conditions at the boundary of the FEM model vary from those at the site, the ground motion calculated at the boundary may be compared with results of one-dimensional finite element or finite difference site response analysis using soil profile at the boundary.

### 10.2.4 Development of Ground Motion Time Series

If ground acceleration time series are used (i.e., performance-based design approach), seed motions should be selected based on the controlling earthquake scenarios (e.g., magnitude, site-to-source distance, significant duration (D5-75, D5-95), Arias Intensity, peak ground velocity (PGV), and period of pulse for forward-directivity motions), and the $V_{s30}$ at the recording station. The percentage of seed motions that have near-source (directivity) characteristics can be defined from deaggregation of the regional seismic hazard (PSHA) for the 2,475-year average return period and across the structural period range of interest, identification of the primary seismic hazards, and the amplitude of the motions from the predominate seismic sources relative to the uniform hazard (NEHRP 2015, NIST 2011).

If spectral matching of seed motions is performed, care should be exercised not to eliminate or unreasonably elongate the pulse period.

Ground motions with velocity pulse characteristics should be rotated and oriented along fault normal (FN) and fault parallel (FP) directions. Furthermore, the modified motions in FN and FP directions should be rotated again based on the orientation of the building axis relative to the causative fault. Seed motions that do not exhibit near-fault effects (i.e., without the forward-directivity or fling step) may be used in a random orientation.

**Commentary:** Applying seed motions that do not exhibit near-source effects in a random orientation deviates from ASCE 7 requirements but is judged to be appropriate. However, care should be taken that the mean spectra for each direction of response meets ASCE limits so as to avoid design that do not meet minimum strength criteria in any direction.

It is recommended that orbit plots at structural periods of interest be made before and after spectral matching and before and after rotation of ground motion along the building axis to confirm that the appropriate orientation of ground motion is used in the structural dynamic analysis.

For structures on continuous foundations with plan dimensions of greater than 400 feet, effects of wave passage and incoherency of ground motion on design ground motions should be evaluated and addressed.

### 10.2.5 Seismic Slope Stability and Soil Liquefaction Hazards

The potential for and consequences of liquefaction or cyclic degradation of soils should be evaluated using current and widely adopted methods of analysis. The evaluation of liquefaction hazard should be based on standard semi-empirical methods.

If potentially liquefiable soil layers are present below the foundation level, the effects of soil liquefaction (strength loss, settlement and down-drag loads acting on deep foundations) and potential for lateral spreading should be evaluated. The GEOR should review published maps and reports regarding potential for soil-liquefaction-induced ground settlement and lateral spreading at the site and in its vicinity.

**Commentary:** Existing reports include Lawson Report on the 1906 Great San Francisco Earthquake (1908), Harding Lawson Associates, City and County of San Francisco Soil Liquefaction Report (1992), GHD-GTC Port of San Francisco Seawall Stability Report (2016), and Port of San Francisco, Seawall Resiliency Study currently underway.

For sites underlain by BM, the potential for seismically induced slope deformation should be evaluated, and mitigation measures should be identified.

### 10.3 Settlement Analysis

Settlement calculations should account for various stages and durations of construction (i.e., estimates of the time required for each stage of dewatering and construction should be made). Stages include, but are not limited to, placement of shoring, dewatering, excavation for construction of basement and foundation, termination of dewatering, and long-term recharge of groundwater table. In some cases, and depending on soil permeability, recharging of the groundwater table may not occur until sometime after completion of construction. This delay in groundwater recharge should be accounted for when evaluating the hydrostatic uplift pressure during and after termination of dewatering (i.e., accounting for full and immediate groundwater recharge after termination of dewatering may be unconservative). The geotechnical report should state the estimated long-term groundwater conditions at the site.

#### 10.3.1 Shallow Foundations

Short-term and long-term (consolidation) settlement analysis should be performed using appropriate models as approved by the geotechnical members of the EDRT.

> **Commentary:** For shallow foundations, consolidation analysis may be conducted using computer programs that perform one-dimensional settlement analyses at several locations across the building footprint and within the zone of influence based on a three-dimensional stress distribution.

#### 10.3.2 Deep Foundations

For deep foundations that terminate above bedrock, short-term and long-term (consolidation) settlement analysis should be performed using appropriate models capable of modeling deep foundations as single piles or pile groups, as reviewed by the geotechnical members of the EDRT.

> **Commentary:** Finite-element or finite-difference computer programs, which are capable of modeling single piles or pile groups, should be used to calculate consolidation settlement of structures supported on deep foundations.

### 10.4 Sea Level Rise

The effects of sea level rise during the design life of structures should be evaluated based on NRC 2012; CCSF 2014; CCSF 2016; Plane et al. 2019; Yasuhara, et al. 2007; and others. Effects considered should include, but are not limited to, the potential for increased flooding and the effect of rising groundwater on increasing hydrostatic pressure, increasing liquefaction potential, saltwater intrusion, and decreasing bearing capacity.

### 10.5 Static and Seismic Design of Basement Walls

Basement walls should be designed against the more critical of the following conditions: (1) At-rest soil pressure and (2) active soil pressure plus dynamic increment. In addition, effects of surcharge loads (traffic and adjacent building foundation, if not underpinned) should be considered.

When calculating hydrostatic pressure, the design groundwater table with consideration of sea level rise and seasonal fluctuation of groundwater table should be identified and used. If a drainage system is not installed behind the basement walls above the design groundwater table, the basement walls should be waterproofed beginning at the ground surface. In this case, the basement walls should be designed per code requirements and checked for the groundwater table being at the ground surface, but using a load factor of 1.0 as opposed to 1.6 for this check.

> **Commentary:** According to the load combination in current building code, a factor of 1.6 is applied to hydrostatic pressure. The resulting pressure in most cases accounts for effects of sea level rise, fluctuation in groundwater table, or effects of a temporary buildup of water behind the basement walls due to a possible breakage in a water conveying pipe adjacent to the site. Care should be exercised to avoid undue conservatism in design against hydrostatic pressure.

Resistance to lateral loads could be calculated by considering frictional resistance on basement walls and beneath the foundation (if not pile-supported) and passive pressure against the basement walls, pile caps, grade beams, and foundation edge extending below the basement walls. In calculating frictional resistance, the effects of the presence of a waterproofing membrane (if used) on allowable frictional resistance should be accounted for.

A load-deflection curve for passive resistance should be developed by the GEOR and used by the SEOR to account for displacement compatibility within various components contributing to lateral resistance.

For basement walls in contact with slopping ground conditions, the effects of unbalanced soil pressure on basement walls should be considered.

## 11. Foundation Support

Shallow or deep foundation systems may be appropriate for support of tall buildings depending on the ground conditions, structural loads, and performance criteria. Unless it could be demonstrated through comprehensive geotechnical and structural studies that the computed total and differential settlement will not compromise the safety and functionality of the structure and its components, foundation systems should be designed to meet the following criteria using the best estimate soil properties: (1) the total short-term and long-term computed settlement of the foundation under gravity and seismic loads should not exceed 4 inches, and (2) its differential settlement under gravity and seismic loads should not exceed an angular distortion of 1/500. Nonstructural components such as cladding or partition walls may control the acceptable threshold of differential settlement. The amount of dishing of the site under building load should be communicated to the SEOR in the geotechnical report, so that the appropriate building camber could be provided.

**Commentary:** The inherent variability of natural soil deposits often causes tilting of the foundation (rigid body rotation), which would add to differential settlement (dishing) caused by the applied structural loads. The magnitude of foundation tilting is directly related to the extent of total settlement. Some tilting can be compensated for during construction; however, some tilting may occur after construction is completed. If settlement of more than 4 inches is calculated, GEOR and SEOR should work together and carefully evaluate the impact of settlement larger than 4 inches on the structural system and nonstructural components. Factors to be considered include the amount of settlement occurring after placement of the mat and before the lowest floor is constructed, the timing of placement of cladding and ability to correct foundation tilting before cladding is installed, and of course, the tolerance of cladding to differential settlement caused by tilting and/or by dishing of the mat foundation.

Settlement analyses are often made using the approximation that the foundation soil deposits are uniform, homogeneous layers. If this simplification is adopted, it is recommended that the GEOR perform analyses to evaluate the sensitivity of the computed settlement on the input soil parameters.

For shallow foundations, the factor of safety against bearing failure (both global failure mechanism and punching shear failure mechanism) should be evaluated. A minimum factor of safety of 2.0 should be maintained under anticipated gravity loads considering the above bearing failure mechanisms.

If ground improvement is used to mitigate the effects of compressible, weak, liquefiable, or other problematic soil conditions, the GEOR should review design calculations by the design-build (DB) contractor to check that the integrity of ground improvement elements is maintained during both static and seismic loading conditions; that is to say, the replacement ratio and geometry of grid pattern should be such that the ground improvement system maintains its integrity under structural gravity loads, seismic loads (base shear and overturning moment applied by the structure), and seismic loads due to vertical propagation of seismic waves.

**Commentary:** Recent research indicates that individual columns of deep soil mix (DSM) would bend during design-level ground shaking, thereby limiting the effectiveness of DSM columns for prevention of soil liquefaction. In addition to lateral movement, individual unreinforced DSM columns could crack in bending and with excessive repeated loading and extensive cracking, could have the effect of losing the cohesive strength associated with cementation, with a residual strength related to contact through friction only. Unreinforced individual columns of DSM are brittle and could fail to transfer gravity loads to more competent soils at depth.

If deep foundations are used to bypass compressible, soft, or liquefiable soils, the following construction design issues should be addressed:

### 11.1 Driven Concrete and Steel Piles

The geotechnical report should address axial and lateral pile capacity, driving criteria, noise and vibration effects, corrosion protection, indicator-pile driving program, and pile load testing.

### 11.2 Augered Cast-in-Place Piles

The geotechnical report should address axial and lateral pile capacity, integrity testing requirements (especially in case of loose to medium dense saturated sandy soils and soft clayey soils) using, for example, cross hole sonic logging, cross hole Gamma-Gamma logging, thermal testing, or a combination of these methods, as appropriate, pile load testing, and requirements for an automated data-acquisition system.

### 11.3 Drilled Shafts

The geotechnical report should address axial and lateral pile capacity, axial pile load test for drilled shafts with reaction piles or bidirectional load cells, integrity testing using cross-hole sonic logging, cross-hole Gamma-Gamma logging, thermal testing, or a combination of these methods, as appropriate.

End bearing for shafts is normally ignored unless pile capacity can be verified by top-down or by using bidirectional load tests. For end bearing in dense sand or bedrock, the bottom of a shaft should be cleaned out thoroughly and tested using Mini SID (Shaft Identification Device) or similar tools for evaluating proper clean out.

## 12. Shoring, Dewatering, Excavation and Underpinning

The geotechnical report should address shoring, dewatering, and underpinning. Design of the shoring, dewatering, and underpinning system is usually provided by specialty contractors, with design parameters (soil and groundwater pressure) provided by the GEOR. If shoring is used to support an adjacent building, the design soil pressure should correspond to the at-rest pressure and account for building surcharge. The GEOR and the EDRT should review the contractor’s analysis and design to evaluate that the design has used appropriate soil and groundwater pressures. The GEOR and the EDRT should also review the contractor’s Plan of Action for trigger levels (e.g., Warning Level or Design Limit) of lateral and vertical movement of the shoring and underpinning system before the start of construction.

Because of the potential presence of confined aquifers within or below the BM and OBC, nested piezometers should be installed outside of the excavation for monitoring of drop in groundwater table and water head within various sand layers, as appropriate.

Bottom of excavation should be evaluated for expected conditions for stability. If cohesionless soil is exposed at the bottom of the excavation, the factor of safety against bottom instability should be calculated to check that piping (quick sand condition) is prevented. If cohesive soil is exposed at bottom of excavation, the factor of safety against basal heave should be calculated. Finally, if a cohesionless soil layer at depth is overlain by a layer of cohesive soil at the bottom of excavation, the blowout condition should be carefully analyzed and, if necessary, the cohesionless soil layer should be depressurized to prevent a bottom blowout condition.

## 13. Instrumentation and Construction Monitoring

The GEOR should provide recommendation for geotechnical instrumentation and construction monitoring at locations where ground conditions, type of loading, or proximity of existing structures could be adversely affected by planned construction.

### 13.1 Selection of Instrumentation and Monitoring Requirements

The type, location, and requirements for instrumentation should be determined by the GEOR based on the impact of construction related to excavation, shoring, dewatering, foundation installation including noise and vibration, and implementation of ground improvement on groundwater conditions and performance of adjacent structures, roadways, utilities, and other improvements.

The geotechnical report should provide the rationale for selection of instrumentation type and number, installation method, and the frequency of monitoring for each type of instrumentation.

The frequency of monitoring should be defined based on the type of loading and construction activities. Monitoring should be initiated before the construction work starts to obtain ambient or baseline conditions. As appropriate, monitoring rates may be adjusted after initial period of monitoring, if data from instrumentations indicate that the rate of change is diminishing with time.

The instrumentation used for monitoring during construction should be sufficient to meet accuracy and reliability requirements needed for the duration of monitoring.

### 13.2 Pre-Construction Monitoring

The GEOR should develop a plan for preconstruction monitoring of adjacent buildings and improvements. The GEOR should request that the shoring and dewatering contractor(s) evaluate the effects of lowering of the groundwater on adjacent structures and improvements, and define the allowable drop in groundwater level outside of the excavation. The allowable lowering of the groundwater elevation should account for the duration of the anticipated construction-related change in the groundwater level. The GEOR should request that the dewatering contractor prepare for review and approval a plan of action in case the groundwater table drop below the contractor's specified limit.

### 13.3 Reporting

The baseline and data collected during construction from piezometers and inclinometers, and field warnings (see section 9 for discussion on warning level or design limit) should be reported to the design and construction team in a timely manner. If in response to a field warning any changes are made to the original design, the revised design should be presented to the GEOR and the geotechnical members of EDRT for further review.

## 14. Other Construction Considerations

The geotechnical report should address the following construction considerations:

- The effects of construction on adjacent buildings, notably where ground improvements or new foundations extend below the foundation of the adjacent buildings;
- The potential of loss of ground and displacements due to construction of large-diameter drilled shafts installed deeper than the foundation of an adjacent buildings;
- Impact of installation of deep foundations on previously installed foundations;
- The potential impact of ground-surface heave or vibrations on adjacent structures and improvements;
- The effect of construction on the groundwater level inside and outside of the construction area.

## 15. Settlement Monitoring Requirements

Prior to completion of all new tall building projects where the building is planned to be supported on a shallow foundation underlain by soil (i.e. the foundation is not bearing directly on bedrock) or on a deep foundation system not gaining axial support within bedrock or not driven to bedrock / bedrock-type material, the project Sponsor shall secure a contract with qualified Monitoring Surveyors and Instrumentation Engineers (MSIEs) to monitor the settlement of the buildings for a period of 10 years after the issuance of CFC/TCO. A notarized legal document, completed by the Project Sponsor and recorded against the property title, with the MSIE's contact details, shall be submitted to DBI prior to issuance of the CFC/TCO and shall be retained with the project's permanent records and readily retrievable within DBI's inspection records on this project.

Settlement monitoring data are to be submitted annually to DBI's Building Inspection Division each year of this 10-year period. Should the settlement monitoring data exceed the project sponsor's geotechnical engineer's estimated time rate of settlement in any annual data reporting period, the project sponsor/owner is required to immediately notify the DBI's Deputy Director for Inspection Services and bring this condition to his/her attention for immediate additional investigation.

6/24/20

Daniel Lowrey — Date
Permit Services Deputy Director
Department of Building Inspection

6/24/20

Patrick O'Riordan — Date
Interim Director
Department of Building Inspection

6/24/20

Gary Ho, S.E. — Date
Plan Review Services Manager
Department of Building Inspection

Approved by the Building Inspection Commission on 06/17/2020.