# City of Malibu

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# GUIDELINES FOR THE PREPARATION OF ENGINEERING GEOLOGY AND GEOTECHNICAL ENGINEERING REPORTS

## AND PROCEDURES FOR REPORT SUBMITTAL

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November 2013

# 1 INTRODUCTION

## 1.1 Purpose

These "Guidelines for the Preparation of Engineering Geology and Geotechnical Engineering Reports" provide the minimum standards and recommended format for the preparation of engineering geology and geotechnical engineering reports for development projects within the City of Malibu. The purpose of these guidelines is to provide the project engineering geologist and project geotechnical engineer with the information necessary to prepare adequate and acceptable reports. It is not the intent of these guidelines to establish strict requirements for development projects; they are however intended to serve as a guide for the preparation of geotechnical engineering reports to meet the requirements of the City and governing codes, regulations, and policies. The guidelines are not proposed to supersede geologic and engineering judgment of the Project Geotechnical Consultants; however it is expected that such judgments be thoroughly discussed within the geotechnical engineering report.

These guidelines are intended to facilitate the development of engineering geology and geotechnical engineering reports to meet the City's requirements such that projects are not needlessly delayed in the geotechnical review process. It is therefore imperative that the Project Geotechnical Consultants are familiar with these guidelines and the requirements to reduce the length of review and the responses required.

These guidelines also discuss the submittal procedures for projects requiring review by the City Geotechnical Staff during the initial Planning Department review phase and the Environmental Sustainability Department review phase.

## 1.2 Definition of Roles

Registered and licensed professional consultants under contract with the City of Malibu serve as City Geotechnical Staff. The City Geotechnical Staff conducts technical reviews of engineering geology and geotechnical engineering reports and plans. The City Geotechnical Staff consists of the City Geologist and the City Geotechnical Engineer along with associated support staff.

*City Geologist*

At the direction of the Building Official, the City Geologist reviews engineering geology reports, building and/or grading plans, and as-built engineering geology and compaction reports for proposed projects. The City Geologist first performs a field reconnaissance, and then subsequently reviews submitted documents to evaluate the geologic site conditions and the adequacy of the project engineering geologist's assessment of the project. When reports are not submitted with the plans, City Geotechnical Staff makes a determination of the need for such reports based on the field reconnaissance. Emphasis is placed on the effect of geologic conditions on the proposed development and the effect of the proposed project on nearby properties. The City Geotechnical Staff notifies the Building Official and the applicant of potential geologic hazards that are associated with the development and that have been previously identified. In addition, the City Geologist reviews reports to assure compliance with the guidelines established herein and with all applicable Federal, State, and local codes and ordinances pertaining to geologic conditions.

*City Geotechnical Engineer*

At the direction of the Building Official and/or City Geologist, the City Geotechnical Engineer reviews geotechnical engineering reports and related documents (e.g., geology reports, building plans, grading plans) from a geotechnical engineering perspective for the proposed development. The reports and related documents are reviewed to evaluate the project geotechnical engineer's assessment of the development. The City Geotechnical Staff notifies the Building Official and the applicant of potential hazards that are associated with the development and that have been previously identified.

*Building Official*

The Building Official is responsible for determining whether or not to issue a building permit for a development project. The City Geologist and City Geotechnical Engineer provide geologic and geotechnical input and advice to the City Building Official and make recommendations to the City either to request more information or to approve the project from engineering geology and geotechnical engineering perspectives. The City Geologist and City Geotechnical Engineer serve as the reviewers for the City and prepare written comments in a geotechnical review that discusses issues related to engineering geology and geotechnical engineering. If the City Geologist and City Geotechnical Engineer determine that there are issues that are not within their purview, they bring the issues to the attention of the Building Official, who then makes a decision on the issues in accordance with established City policies and procedures.

*Project Applicant*

Applicants may include architects, developers, landowners, permit specialists, homeowners, contractors, and others directly involved with development and remediation activities. Applicants are responsible for a complete submittal of all documents needed for geotechnical review and are responsible for payment of fees due with plan and report submittals.

*Project Geotechnical Consultants*

Project Geotechnical Consultants are consulting professionals who are registered and licensed in the State of California and who provide engineering geology and geotechnical engineering services for the project applicant. The Project Geotechnical Consultants consist of:

- A California registered Certified Engineering Geologist or a California registered Professional Geologist with experience in engineering geology ("project engineering geologist"), and
- A California registered Geotechnical Engineer or a California registered Civil Engineer with experience in geotechnical engineering ("project geotechnical engineer").

Project Geotechnical Consultants typically provide two types of services for an applicant. The first type is to provide data and analyses to determine the feasibility of the site for development with regard to geologic and geotechnical hazards during the Planning Department review stage. The second type is to provide design recommendations, to review and approve project plans and specifications, to provide construction observation services, and to prepare as-built reports in the Environmental Sustainability Department review stage.

### 1.3 Applicable Codes

The building code applicable to developments within the City of Malibu is the Malibu Building Code, which references the current Los Angeles County Building Code with amendments (adopted from the applicable California Building Code). For the purposes of these guidelines the term "Malibu Building Code" refers to the City building code that is in effect at the time an engineering geology or geotechnical engineering report is submitted to the City. References to specific sections within the California Building Code are identified with the term "CBC." Applicants and consultants may find the applicable County of Los Angeles modifications to the California Building Code on the County of Los Angeles' web site.

These guidelines do not supersede applicable Federal, State, and local codes. In particular, engineering geology and geotechnical engineering reports should comply with:

- Alquist-Priolo Earthquake Fault Zoning Act of 1972.
- Seismic Hazards Mapping Act of 1990.
- City of Malibu's Local Coastal Program and Local Implementation Plan (LCP-LIP).

In addition to applicable codes and guidelines, applicants and consultants are encouraged to review the selected references listed in Appendix A.

If any differences exist between these guidelines and other references, guidelines, and codes, the more restrictive requirement governs.

### 1.4 Courtesy Calling

The City Geotechnical Staff has a policy of "courtesy calling" that facilitates and encourages communication between the reviewer, the Project Geotechnical Consultants, the applicant, and other professional consultants. This policy allows the reviewers to discuss concerns and advise the Project Geotechnical Consultants about resolving issues regarding both feasibility of the proposed development and building plan-check items. This policy helps to avoid long review processes that involve numerous written iterative responses. Consultants are urged to contact the City Geotechnical Staff at their convenience to discuss review comments and to get answers to questions regarding current codes, policies, and ordinances in effect.

### 1.5 Counter Hours and Appointments

Public counter hours for the City Geotechnical Staff are on Tuesdays and Thursdays between 8:00 AM and 11:00 AM, or by appointment. Appointments are encouraged, especially for any discussions on current reviews as well as for final sign-off on grading and building plans. Such appointments are subject to applicable fees. Contact (310) 456-2489 extension 306 to schedule an appointment.

# 2 PROCEDURES - SUBMITTAL OF DOCUMENTS

## 2.1 Pre-Application Review

City Geotechnical Staff recognizes that applicants may seek assurance that vacant properties are buildable from both engineering geology and geotechnical engineering perspectives. With permission from the Building Official, City Geotechnical Staff may review projects as part of a pre-application review. Reports and a fee are collected (as discussed below), and the project is reviewed only for geologic and geotechnical feasibility. Planning stage "in-concept" approval cannot be granted from City Geotechnical Staff until a formal application for the project is submitted to the Planning Department. Please contact the City Geologist at (310) 456-2489 extension 306 regarding this matter.

## 2.2 Planning Stage Review

The initial step in the permitting process is the submittal of an application and documents to the City's Planning Department for approval. City Planning Department personnel maintain a list of categories of development projects that are subject to geotechnical review in the planning stage. Planning Department staff determines the category of the project and the associated fees for the geotechnical review. A member of City Geotechnical Staff is available during public counter hours to address questions regarding such determination.

The planning stage review is performed to determine whether or not the proposed project appears feasible from a geologic and geotechnical perspective, based on conformance with the City's codes and ordinances. When sufficient data and analyses are presented that demonstrate feasibility, City Geotechnical Staff may provide a recommendation to the City to consider planning stage approval of the project. Details of the foundation design are not reviewed until the project is in the building plan-check stage.

## 2.3 Building and Grading Plan - Check Stage Review

After having received the Planning Department's "approval in concept," the applicant may submit plans to the Environmental Sustainability Department for building and/or grading plan review. It is the responsibility of the Project Geotechnical Consultants to incorporate any building and grading plan design recommendations from reports that were submitted during the planning stage review. On occasion, if requested by the Building Official or the project applicant, City Geotechnical Staff may participate in the building plan-check stage by reviewing engineering geology and/or geotechnical engineering reports and related plans.

Upon submittal of adequate data and analyses that substantiate the Project Geotechnical Consultants' recommendations, and when the consultants' recommendations appear reasonable to City staff, City Geotechnical Staff may recommend that the City Building Official consider building stage approval of the project from both engineering geology and geotechnical engineering perspectives.

### 2.4 Submittal Requirements

When it is determined that the proposed development project requires review by City Geotechnical Staff, the applicant is responsible for submitting the items listed below to City Geotechnical Staff during public counter hours at City Hall, Tuesdays or Thursdays between 8:00 AM and 11:00 AM, or by appointment.

The City Geotechnical Staff requests that geology, geotechnical, and on-site wastewater treatment system (OWTS) reports with associated maps and cross-sections be submitted on a CD in searchable PDF format and that each map and cross-section be shown in its entirety on one page (i.e., please do not scan maps or cross-sections in sections and show them on multiple pages). Faxed or emailed PDF reports are not accepted. Clearly labeling the CD with the following information can help prevent delays in the review process:

- Project address.
- Name and address of consulting firm preparing the report.
- Date of consultant's report.

The applicant is responsible for submitting the following:

- One CD with an electronic signature form. The electronic signature form is available on the City's website or at the City Hall public counter.
- One copy of the completed planning application.
- One set of dated development plans, including grading, OWTS, and drainage plans depicting all surface and subsurface nonerosive drainage devices, flow lines, catch basins, etc.
- Fee receipt from the Planning Department for review fees paid.

City Geotechnical Staff reviews the project after receipt of the listed items and schedules a site visit with the applicant to review site conditions. A "Geotechnical Review Sheet" is typically issued to the applicant following the project review in approximately 10 working days. Complex sites, including multi-family, commercial developments, and subdivisions, may entail longer review periods. City Geotechnical Staff evaluates the applicant's submittal for completeness and conformance to standards of practice and to City, County, and State requirements. The review generally recommends either: 1) approval of the project in the planning stage, or 2) that the project not be approved in the planning stage, providing comments to be addressed by the Project Geotechnical Consultants prior to planning stage approval. Review comments issued during the planning stage may include items to be addressed prior to building/grading plan-check stage and/or recommendations for building/grading plan-check items.

In January 2011, the City adopted a fixed fee schedule for City Geotechnical Staff reviews of geotechnical engineering reports and building and grading plans. Planning Department staff and City Geotechnical Staff assign review categories and associated fees based on the type of development project proposed. The fixed fee collected includes an initial geotechnical review and one subsequent geotechnical review. Additional reviews may be performed on a time-and-materials basis in accordance with the adopted hourly rate of the current fixed fee schedule.

Please do not hesitate to contact City Geotechnical Staff, if there are any questions regarding these guidelines.

## 2.5 Plan-Check Requirements

Typical geotechnical plan-check comments are listed below. It is expected that these comments be addressed by the civil engineer, structural engineer, or architect prior to the final building plan-check review. A separate geotechnical submittal responding to these comments is generally not requested.

### *2.5.1 General Requirements*

Plan-checks may contain general comments such as:

- Provide the name, address, and phone number of the Project Geotechnical Consultants on the cover sheet of the Building Plans.
- Include the following note on Grading and/or Foundation Plans: "Subgrade soils shall be tested for Expansion Index prior to pouring footings or slabs; Foundation Plans shall be reviewed and revised by the Project Geotechnical Consultants, as appropriate."
- Include the following note on the Foundation Plans: "All foundation excavations must be observed and approved by the Project Geotechnical Consultants prior to placement of reinforcing steel."
- Include the following note on the grading plans: "Geologic conditions exposed during grading shall be depicted on an as-built geologic map signed by the project engineering geologist."
- On Foundation Plans for the proposed project, clearly depict the recommended embedment material and minimum depth of embedment for the foundations in accordance with the Project Geotechnical Consultants' recommendations.
- Ensure that foundation setback distances from descending slopes are in accordance with those contained in the Malibu Building Code or with the Project Geotechnical Consultants' recommendations, whichever are more stringent. Illustrate minimum foundation setback distances on the foundation plans, as applicable.
- Illustrate the on-site wastewater treatment system on the Site Plan.
- Please contact the Environmental Sustainability Department regarding the submittal requirements for a grading and drainage plan review.
- Include a comprehensive Site Drainage Plan, incorporating the Project Geotechnical Consultants' recommendations, as applicable, and illustrating all area drains, outlets, and nonerosive drainage devices. Water should not be allowed to flow uncontrolled over descending slopes.

### *2.5.2 Grading Plans*

For grading plans, plan-checks may contain comments such as:

- Ensure that grading plans clearly depict the limits and depths of all cuts, fills, and overexcavations, as applicable.
- Prior to final approval of rough grading, submit for the City's review an as-built compaction report prepared by the Project Geotechnical Consultants. Include in the report the results of all density tests as well as a map depicting the limits of fill, locations of all density tests, locations and elevations of all removal bottoms, locations and elevations of all keyways and back drains, and locations and

elevations of all retaining wall backdrains and outlets. Depict on an as-built geologic map any geologic conditions exposed during grading.

#### *2.5.3 Retaining Walls*

If the project includes retaining walls, plan-checks may contain comments such as:

- Illustrate retaining wall backdrain and backfill design, as recommended by the Project Geotechnical Consultants, on the plans.
- Apply for separate permits for retaining walls that are separate from the proposed structure. Contact the Environmental Sustainability Department for permit information. Submit one set of retaining wall plans to the City for review by City Geotechnical Staff. Additional concerns raised at that time may require a response by the Project Geotechnical Consultants and applicant.

#### *2.5.4 Swimming Pools*

If the project includes a swimming pool, plan-checks may contain comments such as:

- Include the swimming pool subdrain and suitable outlet details on the swimming pool plans.

### 2.6 Fire Rebuild Guidelines

The City of Malibu's Fire Rebuilding-Geology and Geotechnical Guidelines, revised June 1, 1994, are no longer applicable. Redevelopment of properties destroyed in the Malibu-Old Topanga firestorm of November 1993 is considered new development and is reviewed using current geotechnical guidelines.

## 3 GUIDELINES

The guidelines contained in the following sections have been prepared for the purpose of providing a general format and minimum standards for analysis and report preparation by Project Geotechnical Consultants for compliance with City standards and governmental regulations. It is emphasized that these guidelines define minimum standards and that site-specific conditions on most proposed development sites may necessitate that these standards be exceeded.

### 3.1 General Guidelines

Common factors may contribute to geology/geotechnical review concerns that can delay the review and approval process. Generally, these factors are related to the omission or lack of supporting data and/or an insufficient discussion to support the key assumptions made by the Project Geotechnical Consultants. It is the intent of these guidelines to assist consultants in avoiding such problems and to aid in expediting permits for grading and building. In all cases, the data presented in the report needs to substantiate, with appropriate discussions and comments, the conclusions and recommendations of the Project Geotechnical Consultants.

Use of these guidelines enables the Project Geotechnical Consultants, civil engineer, and project engineering geologist to provide the data necessary for the design engineer/architect to more accurately comprehend the geologic setting and thus to best prepare a safe and cost-effective design that is compatible with known or inferred site conditions. These guidelines have been prepared to serve only as a reference during both preparation and review for those striving to facilitate the processing of permits in an expeditious manner. The subjects addressed in these guidelines are not meant to be all-inclusive, but to cover areas that concern the majority of projects in the City of Malibu.

It is expected that the project geotechnical engineer and project engineering geologist provide the City with all the available technical data and information of the site geologic and geotechnical conditions that are obtained during the course of exploration, laboratory testing, and subsequent evaluation and analysis. To best facilitate an effective, efficient, and economic exploratory program, it is strongly recommended that the exploratory needs of all of the Project Geotechnical Consultants be integrated and coordinated to assure the best possible product. To minimize potential failure and ensuing losses resulting from improper care and maintenance after construction, the project geotechnical engineer’s and project engineering geologist's (i.e., the Project Geotechnical Consultants’) recommendations need to be implemented in the design and construction of the project. Recommended site maintenance also needs be acknowledged and implemented. Conformance to the City guidelines and approval of the grading and/or building permits by the City of Malibu shall not be construed in itself as an assurance of safe performance of the proposed development.

## 3.2 Types of Projects

### *3.2.1 New Construction*

New construction projects may include, but are not limited to, new single-family residences, multi-family structures, commercial/industrial buildings, detached guest houses, detached studios, and accessory buildings (i.e., those considered habitable by the Malibu Building Code). Projects involving the redevelopment of existing sites are considered new construction (for example, demolish single-family residence and construct new residence). For all new construction projects, comprehensive engineering geology and geotechnical engineering reports are required that conform to the City guidelines and all applicable codes and ordinances.

### *3.2.2 Remodels*

In accordance with the Local Coastal Program and Local Implementation Plan (LCP-LIP), remodels may include, but are not limited to, interior remodels of existing structures such as conversions of existing buildings from one occupancy type to another. A review by City Geotechnical Staff may not be required if new foundations are not part of the remodel or conversion. Remodels proposing an enlargement of the on-site wastewater treatment system in landslide-prone areas such as Big Rock Mesa, La Costa, Las Flores Mesa/Eagle Pass, and Malibu Road may require some level of review, determined by City Geotechnical Staff on a case-by-case basis. Geotechnical recommendations addressing modifications to existing foundations, new foundations, underpinning foundation elements, floor slabs, and upgrades to meet the Malibu Building Code may be required on a case-by-case basis. It is expected that the engineering geology and geotechnical engineering reports, when required, conform to the City’s geotechnical guidelines and all applicable codes and ordinances.

### *3.2.3 Additions to Existing Structures*

Additions needing City Geotechnical Staff review may include, but are not limited to, first-floor, second-floor, and two-story additions to existing single-family residences, multi-family structures, commercial structures, detached garages, detached guest houses, detached studios, detached pool houses/cabanas, and barns.

Small additions are additions that: (1) add less than 750 square feet of floor area to an existing single- or multi-family residential structure or commercial/industrial structure, and (2) do not exceed 50 percent of the existing building floor area. Additions that do not meet these criteria are considered large additions. Large additions are additions that: (1) add 750 square feet or more of floor area to an existing single- or multi-family residential structure, commercial structure, or industrial structure, or (2) exceed 50 percent of the existing building floor area.

Special study projects generally consist of projects within the Seismic Hazard Zones, Fault Zones, or hillside areas (gradients steeper than 4:1(horizontal:vertical)). Large additions and special study projects should include site-specific geotechnical explorations.

Project Geotechnical Consultants need to address geotechnical issues for both large and small additions within the State of California Seismic Hazard Zones, Fault Zones, and hillside areas. Project Geotechnical Consultants are responsible for providing geotechnical recommendations to address modifications to the existing foundations, modifications to floor slabs, and structural upgrades to meet the Malibu Building Code, as applicable. It is expected that all submitted engineering geology and geotechnical engineering reports conform to the City guidelines and all applicable codes and ordinances.

### *3.2.4 Swimming Pools / Spas*

For purposes of these guidelines a swimming pool is defined as "*any structure intended for swimming or recreational bathing that contains water over 24" deep. This includes in-ground, above-ground, and on-ground swimming pools; hot tubs; portable and nonportable spas; and fixed in-place wading pools.*"

Due to unique geologic settings within the City of Malibu, complete and detailed pool/spa plans need to be submitted for geology review to ensure that pools/spas can be constructed in conformance with applicable codes.

To assist with the applicant's understanding of the geology review of pool/spa plans and with expediting the applicant's permit, please refer to Section 7.3 of these guidelines. In order to effectively and efficiently process and plan-check swimming pool and/or spa plans, applicants should provide, at the time of initial building plan-check application, the information in Section 7.3.

### *3.2.5 Repairs to Existing Structures / Remedial Grading*

Remedial grading and/or repair projects may result from existing structures and/or properties having been damaged by storm surges and wave action, landslide movement, ground movement resulting from volumetric soil changes (including hydrocollapse, settlement, and expansive soil) or from an earthquake (including ground rupture, liquefaction, seismic settlement, or lateral spread), flooding, fires, wood rot and fungi, and other natural disasters. For such projects, engineering geology and geotechnical engineering reports may be required by City Geotechnical Staff in accordance with the Malibu Building Code and City's ordinances. The reports should address the causes and extent of damage and should provide repair alternatives in accordance with standards of practice and the City's guidelines. It is the responsibility of the applicant to record an "Assumption of Risk and Release" for geotechnical hazards not fully mitigated before a permit can be issued. It is the responsibility of the Project Geotechnical Consultants to ensure that the accompanying geotechnical engineering report clearly makes the property owner aware of the extent of the potential damage that may occur to the property if hazards are not fully mitigated.

## 3.3 Types of Studies / Reports

Engineering geology and geotechnical engineering reports may be prepared by Project Geotechnical Consultants for a variety of scopes of services depending on the proposed development project and the stage of review (e.g., planning or building/grading plan-check). Each report submitted should clearly indicate the purpose and scope of the study as well as the proposed development, as discussed in the previous section.

### *3.3.1 Feasibility / Preliminary Design / Design-Level Reports*

Feasibility studies, including EIR documents, focus on feasibility of the proposed development and potential impacts that the proposed land uses could have on the geologic environment. Specific mitigation measures are not required at this stage. Project Geotechnical Consultants should sufficiently demonstrate that all potential geotechnical hazards that may affect the proposed development can be mitigated.

Preliminary design reports address a project at the stage where general development plans have been prepared, although specific development plans may not have been. Preliminary design reports discuss the feasibility of site development for a particular development concept and provide general recommendations for site development. Feasibility reports and preliminary geotechnical engineering reports are often prepared in advance of detailed building or grading plans. Therefore, a supplemental building/grading plan review report may be needed to confirm that the actual building and grading plans comply with the preliminary geotechnical recommendations.

Design-level reports provide site-specific design recommendations related to a specific developmental concept, but the design-level report frequently precedes development of grading and/or building plans. Studies at this stage should relate to specific design recommendations and mitigation of engineering and geologic hazards as they relate to grading and building of the proposed development. For many projects, the preliminary design report is intended by the applicant to serve both as the feasibility design report and the design-level report due to time constraints. In such cases, minor or major changes may occur in development plans between the time the geotechnical engineering report is prepared and the time of submittal. Depending on the magnitude and type of changes, additional geotechnical work may be necessary, and a Building/Grading Plan Review report may be necessary.

In some cases the current development plan may differ significantly from that for which the geotechnical engineering report was prepared, but in the opinion of the Project Geotechnical Consultants additional geotechnical work is not required. In such cases, the Project Geotechnical Consultants are responsible for submitting a letter when the plans are submitted for review. The letter should state that the Project Geotechnical Consultants have reviewed the current plans and that the recommendations in the geotechnical engineering report remain applicable, or the Project Geotechnical Consultants should provide revised recommendations as appropriate.

***Exemption:*** Section 6.1 discusses exemptions to field exploration guidelines.

### *3.3.2 Seismic Hazard Evaluation Reports*

For sites within a Seismic Hazard Zone as identified in accordance with the Seismic Hazards Mapping Act, Project Geotechnical Consultants are responsible for ensuring either that the geotechnical engineering report incorporates a section evaluating seismic hazards or that a separate report is provided, meeting all requirements set forth in said Act and these guidelines. Seismic hazards are to be addressed in accordance with the Seismic Hazards Mapping Act, 1990. Seismic Hazards Maps published by the California Geological Survey (CGS) [formerly California Division of Mines and Geology (CDMG)] include the Topanga Quadrangle released in April 1997, the Malibu Beach Quadrangle released in late 2001, and the Pt. Dume and Triunfo Pass Quadrangles released as final maps in February 2002. These maps are available for review and reproduction at City Hall or from the CGS website.

### *3.3.3 Fault Rupture Hazard Reports*

The California Geological Survey (CGS) has zoned one area of Malibu as an Earthquake Fault Zone in accordance with the Alquist-Priolo Earthquake Fault Zoning Act of 1972. These maps are available for review and reproduction at City Hall. The City's guidelines regarding fault rupture hazard studies are outlined in Section 5.3.1. Requirements depend on the location of the proposed development project in the City and on the scope of the project.

### *3.3.4 Geology Reconnaissance Reports*

Geology reconnaissance reports incorporate a review of the City's files on the site and adjacent properties, regional geologic and geotechnical maps, pertinent pairs of stereographic aerial photographs, and a site reconnaissance. A subsurface exploration is typically not required, however the report needs to be prepared by and signed by a California registered Certified Engineering Geologist or by a California registered ***Professional Geologist*** with experience in engineering geology (i.e., project engineering geologist as defined in Section 1.2).

### *3.3.5 Geotechnical Engineering Reconnaissance Reports*

Geotechnical engineering reconnaissance reports incorporate a review of the City's files on the site and adjacent properties, a discussion of existing geotechnical conditions on the site, an evaluation of proposed geotechnical work on the site, and a site reconnaissance. The engineer should address any potential geotechnical-related hazard and should provide recommendations as to the need for additional geotechnical data and analyses. Subsurface exploration may not be required, however this report needs to be prepared by and signed by a California registered Geotechnical Engineer or by a California registered Civil Engineer with experience in geotechnical engineering (i.e., project geotechnical engineer as defined in Section 1.2).

### *3.3.6 Building / Grading Plan Review Reports*

Building / grading plan review reports involve the review of plans for conformance with the site-specific, approved geotechnical engineering recommendations. If the Project Geotechnical Consultants who are reviewing grading and building plans deem them acceptable for construction, the consultants are responsible for indicating in the applicable reports that the plans conform to all the recommendations made. Reports should be signed and wet-stamped by the project geotechnical engineer and project engineering geologist, as appropriate. When the latest geotechnical engineering report is based on the current building and grading plans or plans with only minor revisions, it is acceptable to review, sign, and stamp current building and grading plans without submitting a new geotechnical review report. Specific guidelines are discussed in later paragraphs. Typical issues to be addressed include:

- Specific grading recommendations (in conformance with the Malibu Building Code).
- Specific surface and subsurface drainage recommendations.
- Slope stability mitigation (e.g., buttress fills, stabilization fills, slope trims or lay-backs, soldier pile stabilization).
- Static settlement mitigation.
- Liquefaction mitigation and mitigation of associated phenomena (e.g., settlement, lateral spreading, sand boils).
- Expansive soil mitigation.
- Collapsible soil mitigation.
- Construction stabilization and shoring plans and specifications.
- Foundation recommendations.
- Retaining wall recommendations.
- Swimming pool design recommendations.
- Flatwork recommendations.

### *3.3.7 Update Reports*

Update reports from Project Geotechnical Consultants may be required at the discretion of the Building Official. Such reports may also be necessary when:

- The scope of the project changes.
- Site conditions change.
- Previous reports are sufficiently old so as to be outdated with regard to industry standards of practice or building codes. Typically the threshold time would be a report that is more than one year old.
- The Project Geotechnical Consultants included a statement in the report that an update would be needed after a certain amount of time, typically one year.

Update reports should: describe the currently proposed development; include a site reconnaissance, plan review, and an up-to-date site plan or geotechnical/geologic map (see Section 4.3); and reference prior reports. The update report should either state whether all recommendations of the prior report(s) are applicable, or provide revised recommendations, as appropriate.

### *3.3.8 As-Built Reports*

City Geotechnical Staff or the Environmental Sustainability Department determines whether the Project Geotechnical Consultants need to prepare as-built reports for a site, depending upon the scope of grading. It is expected that the project geotechnical engineer prepare these reports upon completion of grading of a site. City Geotechnical Staff typically requests this in the comments of the "Geotechnical Review Sheet." Reports need to comply with the Malibu Building Code.
As-built reports typically include, but are not limited to, the following topics:

- Results of all in-place density tests and maximum density determinations.
- Results of all expansion index tests.
- Deep (e.g., pile and drilled pier) foundation observations and documentation.
- Results of revised as-built slope stability analyses (if warranted).
- Documentation of all bottom approvals.
- Results of all settlement monitoring.
- A map depicting the limits of grading, locations of all density tests, removal bottom locations and elevations, keyway bottom locations and elevations, all keyway, cleanout, and swimming pool subdrain locations and flow-line elevations, and all retaining wall backdrain locations and flow-line elevations.

The dry density and moisture content data should be presented in a form to show in-place values along with the associated laboratory maximum dry densities and optimum moisture contents. All failed tests should be clearly marked along with the associated retests.

The Project Geotechnical Consultants are responsible for preparing an "as-built" geotechnical engineering report to document the installation of deep foundations such as caissons, friction piles, soldier piles, etc.

Footing and slab inspections are to be documented in field memos, which are submitted by the Project Geotechnical Consultants to a field representative of the Building Official along with results of expansion index tests to confirm the expansive characteristics of the supporting materials.

#### *3.3.9 As-Built Engineering Geology Reports*

The project engineering geologist is responsible for preparing as-built engineering geology reports upon completion of grading a site. City Geotechnical Staff determines whether the project engineering geologist needs to prepare this report for a site, depending on the scope of grading and geologic conditions exposed. The engineering geology report should discuss geologic conditions exposed during grading, provide additional recommendations for the proposed development if unusual or unexpected conditions are encountered, and include a map depicting the geologic conditions exposed during grading.

### 3.4 Change of Consultant Letters

When a change in Project Geotechnical Consultants occurs after the review process has been initiated or ownership of the property has changed, it is the responsibility of the new Project Geotechnical Consultants to provide written notification to the City. Such letters typically state that the new Project Geotechnical Consultants have reviewed the work by the previous Project Geotechnical Consultants, concur with their recommendations and conclusions, and agree to assume responsibility as Project Geotechnical Consultants of record from this date forward. When the new consultants do not concur with the previous Project Geotechnical Consultants' conclusions and recommendations, additional subsurface exploration, testing, and analyses may be warranted. It is the responsibility of the new Project Geotechnical Consultants to provide two printed copies of the letter that are stamped and signed by the new Project Geotechnical Consultants, and to provide an electronic copy of the notification to City Geotechnical Staff for review and for the City's files. No new permits are issued for a project, and all previously permitted work stops, until the City is officially notified of the name, address, and telephone number of the new project engineering geologist and project geotechnical engineer, or until otherwise approved by the Building Official.

### 3.5 Level of Professional Responsibility

*Geology Work*

In accordance with Article 3, Section 7835 of the Geologist and Geophysicist Act, "*all geologic plans, specifications, reports, or documents shall be prepared by a professional geologist, or registered certified specialty geologist, or by a subordinate employee under his or her direction. In addition, they shall be signed by such professional geologist, or registered certified specialty geologist or stamped with his or her seal, either of which shall indicate his or her responsibility for them.*" All documents that include engineering geology data, interpretations, or recommendations should be signed, dated, and stamped by a California ***Certified Engineering Geologist (CEG)*** or by a California registered ***Professional Geologist*** with experience in engineering geology and should include the geologist's license number and license expiration date. Certain projects, including essential facilities and schools, need a California ***Certified Engineering Geologist***.

*Geotechnical Engineering Work*

In accordance with the California Business and Professions Code, foundation and geotechnical investigations and engineering reports should be prepared by either a registered ***Geotechnical Engineer*** or a registered ***Civil Engineer*** with experience in geotechnical engineering. All documents that include engineering data, interpretations, or recommendations should be signed, dated, and stamped by a California registered ***Geotechnical Engineer (GE)*** or a California registered ***Civil Engineer (CE)*** with experience in geotechnical engineering, and should include the engineer's license number and license expiration date. Certain projects, including essential facilities and schools, need a California registered ***Geotechnical Engineer***.

## 3.6 Definitions

### *3.6.1 Building Site*

The building site includes that portion of the lot or parcel of land upon which the building is located as well as the surrounding area that includes hardscape, clearances, proper site drainage improvements, and easements.

### *3.6.2 Restricted Use Area*

Any area that is within the lot or parcel of land and that has unmitigated geotechnical hazards or areas with slope stability factors below minimum guidelines may be designated as a "Restricted Use Area." It is important that Restricted Use Areas be shown on the geotechnical map. Buildings and swimming pools are not allowed in Restricted Use Areas. These areas may be modified, provided that the geotechnical hazard is mitigated.

### *3.6.3 Habitable Structure*

According to the California Code of Regulations Section 3601 (i.e., the Policies and Criteria of the State Mining and Geology Board, with Reference to the Alquist-Priolo Earthquake Fault Zoning Act), a "structure for human occupancy" is defined as "*any structure used or intended for supporting or sheltering any use or occupancy, which is expected to have a human occupancy rate of more than 2,000 person-hours per year*."

# 4 GUIDELINES FOR REPORTS

Reports consist of engineering geology reports, geotechnical engineering reports, or a combination of both geology and geotechnical disciplines in one report. This section provides specific guidelines related to report content for various aspects of these reports.

## 4.1 Geotechnical Reference Standards

It is expected that all engineering geology and geotechnical engineering reports comply with the most recent versions of appropriate standards, codes, and professional guidelines. The citations for some of the appropriate references are included in Appendix A.

### 4.2 Report Organization

The Project Geotechnical Consultants are responsible for providing geotechnical engineering reports that include the following items, as appropriate, for each project. Project Geotechnical Consultants may determine the specific report format.

- Purpose and Scope – Clearly identify the purpose and scope of the study report.
- Site Description – Describe the existing site conditions, including:
  - Site location, address, and cross streets.
  - Site topography.
  - Site drainage.
  - Existing structures and improvements.
  - Adjacent properties, in particular, closely located structures, OWTS, subterranean structures, and slopes that may affect or be affected by the proposed development.
- Proposed Development – Describe the proposed development and clearly show it on plans and cross-sections.
- Previous Geotechnical Data – Include:
  - Date and consultant's name for all previous reports prepared for the site.
  - Subsurface data for exploratory excavations by previous consultants.
  - Pertinent laboratory data.
- Field Exploration – Describe the field exploration, the methods of excavation, and the methods and types of sampling, provide exploration logs, and include dates of exploration.
- Materials Testing – Describe the laboratory testing procedures and test results, provide graphical laboratory test sheets, and reference current laboratory test procedures.
- Geotechnical Analyses and Findings – Describe the analyses performed and the technical findings, including a discussion to support the selection of shear strength parameters and the interpreted geologic structure. At a minimum, specifically address each of the following potential hazards. The purpose and results of each analysis should be described.
  - Seismic hazards (see Section 5.3 – Seismic Hazard Evaluation).
  - Expansive soil (see Section 6.10 – Expansive Soils).
  - Hydrocollapse potential (see Section 6.9 – Hydrocollapse).
  - Slope stability, including rotational and translational instability, mud and debris flows, and rockfall hazards (see Section 6.4 – Slope Stability Analyses and Section 6.5 – Seismically Induced Slope Instability).
- Conclusions concerning the geotechnical feasibility of the proposed project (see Section 5.4 – Engineering Geology Conclusions).
- Recommendations (see Section 5.5 – Engineering Geology Recommendations and Section 7 – GEOTECHNICAL ENGINEERING RECOMMENDATIONS).
- Figures – Include with each report:
  - Site location map.
  - Regional geologic hazard maps, USGS, Dibblee, etc.
  - Seismic hazard map (for sites near liquefaction or landslide hazard zones and fault zones).
  - Site geologic maps.
  - Site geotechnical map (at 40-scale or less).
  - Geotechnical cross-sections (per Section 4.3.4 – Geotechnical Cross-Sections).

- Signatures of Registered Professionals.
- References – Include as appropriate (see Section 4.4 – Technical Documentation).
- Appendices – Include as appropriate (see Section 4.4 – Technical Documentation).

## 4.3 Maps, Plans, and Cross-Sections

### *4.3.1 Site Location Map*

A site location map (to be provided for all projects) has a prominent north arrow and appropriate scale, indicates the subject site and surrounding area, and encompasses a large enough area to easily and accurately locate the site on regional maps.

### *4.3.2 Regional Geologic Hazard Maps*

Regional geologic hazard maps depict conditions that extend beyond the site geologic map. It is expected that the location of the subject site be shown on all regional maps and that scale and north direction be shown on all maps. Regional geologic hazard maps may be used to locate and generate geologic cross-sections that extend offsite, especially where sites encroach into hillside areas. For example, sites along the south side of Pacific Coast Highway need— in addition to a site-specific geologic map— a regional geologic map and cross-sections to depict slopes north of Pacific Coast Highway that may affect the site.

For all sites, it is the responsibility of the Project Geotechnical Consultants to provide copies of seismic hazard maps and Earthquake Fault Zone maps that show the site. The scale of the hazard map needs to clearly show the location of the site and the proximity to the hazard.

### *4.3.3 Site Geotechnical / Geologic Maps*

A site-specific geotechnical map (to be provided for all projects) depicts the site and surrounding area and includes:

- Existing on-site structures and closely located offsite structures that have the potential to interact with the proposed development.
- Proposed improvements.
- Limits (e.g., contacts) of earth units across the site.
- All exploratory borings and trenches/test pits known to exist on the site.
- All geologic cross-section lines.
- Geologic data from all subsurface excavations and surface mapping (where applicable).
- An explanation that clearly defines all contacts, symbols, lithologic units, and other relevant data shown on the map.
- On all oversize plans, bar scales.

For projects with significant grading, the site-specific geologic/geotechnical map should use an accurate topographic base map and should have a scale sufficient to clearly depict the details of the proposed development and geologic and soil conditions. The base map should clearly indicate the map scale, true north, and who prepared the map. All update reports should include a geologic map showing the current proposed construction and cross-sections.

#### *4.3.4 Geotechnical Cross-Sections*

Cross-sections depict interpreted geologic conditions underlying the site. Cross-sections should be drawn for all beach-front properties and where natural, cut, or fill slope heights, basement walls, retaining walls, or temporary / permanent excavations exceed 10 feet, or when an excavation extends below a 1:1(horizontal:vertical) plane from adjacent foundations. Cross-sections are to clearly show site boundaries, all existing and proposed structures, all exploratory excavations, contacts between earth units, intersections with other cross-sections, and the extent of proposed grading and overexcavations.

The Project Geotechnical Consultants are responsible for ensuring that geologic data— including the measured and the highest anticipated groundwater conditions across sites both in flat, alluvial areas and in hillside areas— is reasonably interpreted throughout the length of the section. The most adverse conditions that can reasonably be expected given the field conditions and site history need to be illustrated. Additionally, cross-sections should show historical high groundwater levels as well as current groundwater levels.

It is important that geologic cross-sections extend from the top to the bottom of slopes (and further as needed to depict critical geologic conditions), without regard for property lines. When offsite geologic conditions could influence a site, cross-sections should be drawn to illustrate those conditions. (This is common for properties located along the south side of Pacific Coast Highway as well as many hillside properties.) Cross-sections should be constructed across the site to depict the proposed seepage pits or leach fields, drip dispersal fields, anticipated paths of effluent, recommended capping depths for seepage pits (if applicable), areas where mounded groundwater would occur, and underlying geologic and groundwater conditions. In addition, cross-sections should depict changes in seepage pit elevations and capping depths. Those changes should consider proposed or future changes in site grades.

Bar scales should be used on all oversize cross-section sheets to facilitate the review process.

### 4.4 Technical Documentation

For all submitted technical documentation, the Project Geotechnical Consultants are responsible for:

- Substantiating all findings, conclusions, and recommendations by data included within the report.
- Reviewing and referencing in the report any applicable regionally published (and unpublished, if available) geology reports, maps, aerial photographs, and other technical documents (e.g., geotechnical engineering reports on file with the City) for the immediate area or subject property.
- At a minimum, researching all public files for the surrounding area, including update reports when prepared. For the purposes of this document, “surrounding area” includes all areas within 300 feet of the property boundaries, or extending further if needed to encompass adjacent slope areas underlain by geologic conditions that may influence the subject site development. In preparing update reports or responses where the time between the last research of all public files and the current report is more than two years, the research needs to be updated.

- Substantiating all recommendations and conclusions with site-specific field and/or laboratory data and appropriate analyses.
- Where professional judgment is employed to augment the data and analyses, clearly and thoroughly discussing the technical rationale to support the judgment.
- Disclosing any potentially hazardous geotechnical processes and site conditions.

### *4.4.1 Previous Geotechnical Data*

The Project Geotechnical Consultants are responsible for incorporating within the report all geotechnical data previously collected for the subject site and adjacent sites if that data is used to support engineering geology and geotechnical engineering interpretations. Such data may include geologic maps and appropriate cross-sections and should be referenced in the geotechnical engineering report. Project Geotechnical Consultants should perform a diligent search for previous data, should discuss known geotechnical investigations for the site, and should include copies of previous reports and data as appropriate.

### *4.4.2 Identification and Mitigation of Risks*

The Project Geotechnical Consultants are responsible for describing, discussing, and evaluating all potential geotechnical hazards (e.g., seismic shaking, fault and ground rupture, liquefaction, lateral spreading and surface manifestation associated with liquefaction, seismically induced settlement, tsunami, seiche, slope stability, expansive soils, and hydrocollapse) and for either stating that each hazard is not present or providing appropriate mitigation measures for all projects (e.g., new construction, remodels, additions, swimming pools/spas, and repairs) regardless of size or scope. Discussions and evaluations of each potential geotechnical hazard and any proposed mitigation measures need to be adequately and clearly supported with engineering geology and geotechnical engineering data and appropriate analyses. Reviewers expect that the Project Geotechnical Consultants use the discussions and evaluations to demonstrate that they have given adequate consideration to each potential geotechnical hazard. Additionally, it is expected that the discussions and evaluations provide information to the property owner as to which hazards are present at the subject site, which hazards are not present, and the mitigation measures that are being implemented (see Sections 4, 5, 6, and 7). The lack of discussion and evaluation of a particular hazard is not interpreted by the reviewers as a presumption that such hazard does not exist, even if in the opinion of the reviewer a particular hazard is not present at a site. It is neither the intent nor responsibility of the reviewer to infer conclusions that a particular hazard is not present. The Project Geotechnical Consultants should provide appropriate statements for each of the typical geotechnical hazards. Reports submitted without an evaluation of and comments on all potential hazards may be deemed incomplete and a response may be requested in the review.

Although the risks associated with some hazards cannot be totally eliminated, the Project Geotechnical Consultants should recommend risk mitigation methods that minimize the effects of the hazards (e.g., preventing structural collapse, injury, loss of life, etc.), ***and should identify in the report for the property owner the level of risk that remains for all risks that are not mitigated.*** Acceptable mitigation methods may include recommendations related to site improvement, site drainage, maintenance practices, structural design, and obtaining appropriate insurance.

In situations where such hazards are not identified at the site, the report should include statements to that effect and should provide support for making such statements. For example, CGS seismic hazard maps may be cited for certain exempt projects, as identified in Section 6.1, to support statements that liquefaction or seismically induced landslides are low risks, provided that the Project Geotechnical Consultants concur and specify the engineering geology or geotechnical engineering data that supports such concurrence. Another example is using consolidation data from nearby sites to support statements that foundation settlement due to hydrocollapse potential is low risk for small projects where extensive laboratory testing is deemed unwarranted and there is no history of hydrocollapse problems in the area, provided the Project Geotechnical Consultant is of the opinion that such data is representative of the subject site and the risk is appropriately discussed (see comments below concerning hydrocollapse). Soil classification data (e.g., dry density, moisture content, degree of saturation, and soil type) may also be useful to support such statements.

#### *4.4.3 References*

Reports should include a statement referring to the standards and specifications used for all field and laboratory procedures. Referenced materials may include:

- Literature and records cited and reviewed.
- Aerial photographs or images interpreted, listing the type, date, scale, source, index numbers, etc.
- Compiled data, maps, or plates included or referenced.
- Other sources of information, including well records, personal communications, procedures, or other data sources.

#### *4.4.4 Geotechnical Exploration Logs*

Logs for all geotechnical explorations performed on site need to be provided with the report. The following information needs to be depicted on the exploration logs or otherwise incorporated into the report:

- Names of the responsible field personnel.
- Dates of exploration.
- Exploration method/drill rig type (e.g., hollow-stem auger, bucket auger, or wet rotary).
- Groundwater observations (indicating time of measurement).
- Sample depths.
- Hammer (e.g., safety hammer), sampler details (e.g., SPT with or without liners, or modified California sampler), method of hammer drop (e.g., automatic or rope-cathead, with number of wraps), and method used to convert measured sampler blow counts to an equivalent blow count associated with SPT with a delivered energy of 60 percent ($N_{60}$). SPT data derived from wire-line hammers cannot be used to perform quantitative analyses that use blow-count data, as such systems deliver inconsistent energy to the sampler.
- Detail of Kelly bar weight and drop height (if applicable).
- Field (unmodified) sampler blow counts.
- Description of excavation backfill.
- Cone penetrometer (CPT) data.
- Results of field tests (e.g., pocket penetrometer and vane shear).
- Results of soil density and moisture tests and percentage fines.

- Results of other in situ testing.

#### *4.4.5 Cone Penetrometer Data*

Cone penetrometer (CPT) data, when obtained, should include profiles of cone tip resistance, either sleeve resistance or friction ratio, and porewater pressure, when available. Interpreted results, such as soil type, estimated relative density, friction angle, or undrained shear strength of the soil, and equivalent sample blow counts also need to be included in the report. It is the responsibility of the Project Geotechnical Consultants to cite the methodology for interpreting the CPT data and to document the type and size of the cone and its penetration rate.

CPT data needs to be substantiated by at least one adjacent soil boring, with samples analyzed for sampler blow counts and grain-size distribution and compared to interpreted CPT results.

#### *4.4.6 Plans and Cross-Sections*

When grading is proposed, the Project Geotechnical Consultants should provide a grading plan that shows existing and proposed contours, with a sufficient number of cross-sections to clearly depict the proposed landform alterations.

#### *4.4.7 Computer-Assisted Analyses*

Engineering analyses assisted by computer programs need to include reference information regarding the software used. Reports should include printouts of applicable input and output files. When spreadsheets are used to perform the analyses, sufficient cells demonstrating the results of intermediate calculations need to be shown such that the reviewer can confirm the results for these intermediate steps. All equations for the calculations should be documented in sufficient detail that the analyses can be verified.

## 5 ENGINEERING GEOLOGY GUIDELINES

### 5.1 Excavation Permits

Prior to the commencement of any excavation of borings or trenches on a site, it is the responsibility of the project applicant to complete a permit application and to obtain an Excavation Permit from the Environmental Sustainability Department. The permit application is available at the building safety public counter at City Hall and is also available on the City's website.

The Project Geotechnical Consultants or applicant should call Dig Alert at (800) 227-2600 at least two working days before the work is to begin so that the site can be properly marked for utilities. The Dig Alert reference number must be included on the excavation permit application.

The application is to be completed by the project applicant and submitted along with a map depicting all proposed exploratory excavations on the site. The completed permit application and map may be faxed to the Environmental Sustainability Department at (310) 456-7650. When approved, the form is returned to the applicant via fax. To obtain an Excavation Permit, bring the signed application to the City Hall Public counter. Payment of a fee is necessary prior to permit issuance.

## 5.2 Field Exploration Program

The Project Geotechnical Consultants are responsible for ensuring that exploration programs are sufficient in number and depth to evaluate site conditions, and for acquiring data to justify all conclusions and recommendations. Explorations need to extend to a depth greater than the proposed foundations. Where applicable, the exploration program should be coordinated between the project geotechnical engineer and the project engineering geologist. Subsurface exploration should be performed in areas most likely to reveal adverse geologic and soils conditions that may impact the proposed development and/or offsite properties as a consequence of the development on the subject site. Conditions to be evaluated and documented include:

- All geomorphic features that suggest the presence of landslides, mud and debris flows, faults, near-surface groundwater, and other possible adverse conditions.
- Geologic conditions, including bedding, joints, shears, clay seams, fractures, and physical properties of all soils, alluvial deposits, terrace deposits, colluvial deposits, weathered bedrock, bedrock, and other earth materials encountered.
- Locations of springs, artesian conditions, seeps, perched zones of groundwater, aquicludes, aquitards, and confined and unconfined aquifers.

At a minimum, the exploration program should include:

- In flat, alluvial areas, borings need to extend below a zone where increases in stress due to imposed loads do not negatively impact the performance of the site improvements, and borings need to be sufficiently deep to evaluate the hydrocollapse potential, the liquefaction potential, and the potential of seismically induced settlement of the site.
- In hillside areas, the depth of borings needs to be sufficient to locate the upper and lower limits of weak zones potentially controlling slope stability. It is important that the factor of safety of a potential slip surface passing beneath the maximum boring depth be equal to or exceed 1.5. In hillside areas, more than one boring is necessary to adequately evaluate the site for geologic conditions and slope stability. The ASCE-LA guidelines (Blake, et al., 2002) for mitigating landslide hazards provide additional information that may be used when establishing the scope of the field exploration program.
- Sampling intervals need to be at 2- to 3-foot intervals in the upper 10 feet and at 2- to 3-foot intervals in the upper 10 feet below cuts. The sampling frequency may increase to 5-foot intervals more than 10 feet below cuts. Sampling should occur at changes in material types when changes occur more frequently than the above sampling intervals.
- Qualified personnel under the direct supervision of a registered geotechnical professional need to log in detail all subsurface excavations. In reports, the Project Geotechnical Consultants should present geotechnical logs that include descriptions of earth units, intervals sampled with field (unmodified) blow counts, laboratory test results (where appropriate), and logs of the soils and/or geology. For the detailed evaluation of geologic conditions under the site, the project engineering geologist should provide downhole logging of geologic borings in hillside areas, unless safety issues preclude downhole logging. If downhole logging is not performed, then appropriately conservative

assumptions regarding geologic structure and lithology need to be incorporated in the project. The report should describe the method of sidewall preparation for downhole or trench logging.

Geotechnical engineering reports need to contain sufficient field exploration data to adequately characterize the subsurface materials and substantiate any conclusions or recommendations that are derived from that data. The field exploration program should at a minimum conform to 2010 CBC Section 1803.5 and present complete data, representative of the site conditions during and after site development, from a geotechnical engineering perspective.

### 5.3 Seismic Hazard Evaluation

Geotechnical engineering reports should address all potential seismically induced hazards that may affect the subject property and proposed development, and should provide adequate mitigation measures as necessary.

The Project Geotechnical Consultants are responsible for evaluating seismic hazards in full conformance with:

- The *Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117 and 117A* (CDMG, 1997, updated 2008),
- The SCEC document entitled *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California* (Martin and Lew, 1999), and
- The guidelines published by SCEC entitled *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California, Organized through the American Society of Civil Engineers, Los Angeles Section (ASCE-LA)* (Blake, et al., 2002).

In accordance with the Seismic Hazards Mapping Act of 1990 (see Sections 2690 through 2699 of the Public Resources Code), the City of Malibu has been included in the Seismic Hazards Maps for the Topanga, Malibu Beach, Pt. Dume, and Triunfo Pass Quadrangles. These maps are available for review at City Hall and on the CGS website. The maps delineate zones within the City that may be subject to liquefaction and earthquake-induced landslide hazards. Seismic hazard evaluation reports to accompany many of the maps have been prepared by the CGS and are available for review at City Hall or on the CGS website.

The Malibu Coast fault system is mapped east-west through the City of Malibu. Although all fault branches are not well defined throughout the area, the fault system has clearly been active during the Holocene (i.e., the last 11,000 years). Splays of the fault in the Latigo Canyon and Solstice Canyon areas within the City have been classified by the State Geologist as active, and zoned in accordance with the Alquist-Priolo Earthquake Fault Zoning Act of 1972. This fault system defines the southern boundary of the Transverse Range Provence in which historical earthquakes have had moment magnitudes of approximately 6.5. Consequently, all engineering geology and geotechnical engineering reports need to contain, at a minimum, a site-specific description of the following:

- Location of major fault traces and known faults near the site based on a review of surrounding geology files, regional maps, FER-229 by the CGS (Treiman, 1994), and stereo pairs of aerial

photographs. Distances from the site to faults within 2 miles of the site should be based on appropriate geologic maps and not on fault locations determined by computer programs using the CGS fault database.
- Fault rupture and ground rupture hazard evaluation.
- Significant historical earthquakes including epicenter distances, earthquake magnitudes, and estimated intensity at the site.
- Evaluation of ground shaking potential.
- Potential for liquefaction.
- Potential for lurching and topographic-related site effects.
- Potential for lateral spreading.
- Potential for surface manifestations.
- Potential for seismically induced settlement.
- Potential for earthquake-induced landsliding in hillside areas.
- Tsunami potential, for sites located within the zone of the tsunami inundation map published by the California Emergency Management Agency (2010). For the Malibu area this is needed for all sites that have finish floor levels below an elevation of 35 feet MSL.
- Seiche potential.

#### *5.3.1 Fault Rupture Evaluation*

The project engineering geologist should excavate trenches in a north-south direction across habitable building sites including single-family residences, multi-family residential units, guest houses, studios, and commercial buildings, to evaluate fault rupture hazards if the site lies:
- Within an Alquist-Priolo Fault Zone as defined by the CGS maps. These maps are available for review at City Hall; or
- Within 500 feet of faults mapped in the CGS 1994 Fault Evaluation Report, FER-229 (Treiman, 1994) for the Malibu Coast Fault Zone. A copy of this report is available for review at City Hall.

Project engineering geologists may utilize existing nearby trench data and exposures on adjacent properties east and west of the subject site in lieu of trenching. The City Geologist reviews these on a case-by-case basis with the project engineering geologist.

Where bedrock is available in the near-surface, trenches should be excavated to a minimum depth of 2 feet into layered, unweathered bedrock where the consultant is able to recognize continuity of stratigraphy. If bedrock lies below thicker sequences of surficial materials such as colluvium, alluvium, or terrace deposits, trenches should be excavated a minimum of 2 feet into layered, unweathered surficial materials where the consultant is able to recognize continuity of stratigraphy. Detailed, illustrated logs of the trenches (at a scale of 1 inch equals 5 feet) need to be provided, along with descriptions of all earth units and geologic conditions encountered. A discussion of the findings should be provided, including conclusions regarding activity of all faults exposed in the excavations. While trenching in beach areas and areas of ancient landslides is impractical and unsafe, the project engineering geologist is responsible for providing a detailed discussion of faulting in the area, utilizing pertinent references and stereographic pairs of aerial photographs. Other forms of site-specific fault rupture hazard investigations may be acceptable provided they are conclusive (based on mutual agreement in writing between the project engineering geologist and the City Geologist.) Acceptable methods may include CPT investigations and large-diameter borings. Seismic reflection surveys may be utilized to determine areas to trench on a property.

When fault trenching is needed, it is the responsibility of the project engineering geologist to sign the fault hazard evaluation and to ensure that the signed document includes a discussion of:

- Site location relative to the various mapped locations of the fault and geomorphic scarps and/or lineaments.
- Recency of activity on the fault.
- Relative risk and consequences of fault or ground rupture at the site.
- Mitigation measures.

When fault trenching, or alternative means, demonstrates that a fault is present, new construction is not permitted over the trace of the fault and an adequate setback needs to be established as mitigation. The project engineering geologist is responsible for establishing minimum setback distances in accordance with the requirements of CGS Special Publication 42, Fault-Rupture Hazard Zones in California (2007).

Exceptions: Fault trench explorations are not required for the following project types:

- Remodels and additions.
- Nonhabitable structures.
- Swimming pools and spas.

For the above exempted projects, for projects within the Fault Hazard Zone, and for projects within 500 feet of faults mapped in the CGS 1994 Fault Evaluation Report, FER-229 (Treiman, 1994) for the Malibu Coast Fault Zone, geotechnical engineering reports need to discuss the risk, provide a conclusion regarding the relative hazard at the site, describe to the property owner measures that may be taken to assess the likelihood of a fault traversing the property, and describe the potential consequences if a fault extends below the proposed development.

#### *5.3.2 Ground Shaking*

Reports should discuss the potential hazard from strong seismic ground shaking. For completeness, the discussion should include an analysis of both probabilistic and historical ground motions. Where appropriate for quantitative hazard analyses (e.g., liquefaction, seismically induced settlement, and slope stability), ground acceleration values are to be represented by the ground acceleration having a 10 percent probability of exceedance in 50 years, derived using either: (1) an unweighted magnitude analysis (with associated deaggregated mean magnitude), or (2) a magnitude-weighted analysis (weighted to M = 7.5). Design accelerations, methods of analysis, and the probability of exceedance need to be discussed and justified in the report. Analyses should be based on earthquake events generated by faults that may affect the site (including faults within at least 65 miles of the site) using the current CGS/USGS fault database. Any deviations from the CGS/USGS fault database should be described and justified. The analyses should be performed using the Next Generation Attenuation (NGA) relations published in 2008.

Earlier versions of SP 117 and the CBC allowed ground accelerations to be based on CGS seismic hazard evaluation report maps, in lieu of a site-specific study. This is no longer permissible (per SP 117A – CGS, 2008), because the fault database and attenuation curves have been updated (Petersen, Frankel, Harmsen, Mueller, Haller, Wheeler, Wesson, Zeng, Boyd, Perkins, Luco, Field, Wills, and Rukstales; 2008). Ground accelerations are to be based on current versions of SP 117 and the CBC. Section 1803.5.12 of the 2010 CBC states that ground acceleration can be taken as the short-period design spectral acceleration ($S_{DS}$) divided by 2.5 in lieu of a site-specific study. Unfortunately this does not provide guidance for selecting the earthquake magnitude, which is necessary for liquefaction evaluations, seismically induced settlement, and lateral spreading evaluation. Site-specific peak ground acceleration associated with a 10 percent probability of exceedance in 50 years and an unweighted magnitude can be determined from the USGS web site or from an equivalent program. (FRISKSP is no longer an acceptable program as the fault database and attenuation curves have not been updated.) It is important that the associated magnitude of the peak ground acceleration be the mean and not the modal value because the modal value, being the most frequently occurring number, may not be giving sufficient weight to the more severe conditions that can occur.

Analyses of previous earthquakes have shown that the 1994 Northridge earthquake has been one of the most significant historical earthquakes in the Malibu area. During the Northridge earthquake, seismographs in or near the City of Malibu recorded peak ground accelerations varying from 0.128g to 0.333g, based on data from CGS. To estimate the historical seismic shaking that has occurred at a subject site, a map of peak ground accelerations recorded from the Northridge earthquake should be included in the report. To facilitate that process, a contour map of ground motion from the Northridge earthquake is provided for the Project Geotechnical Consultants' use (http://www.malibucity.org/index.aspx?nid=258). The consultants should include a copy of that ground motion map in their report, with the subject site plotted on the map. On the basis of that map, the consultants should interpolate the ground acceleration at the subject site and state that value in their report.

### *5.3.3 CBC Seismic Design Factors*

Seismic design factors should be provided in accordance with the current editions of the California Building Code (CBC) and of the County of Los Angeles Building Code with Amendments, including seismic amendments. The CBC static-force procedure calls for the following seismic parameters to generate the response spectrum: the maximum spectral accelerations for 0.2 second ($S_S$) and 1 second ($S_1$), which are influenced by the location, seismicity of the area, and the site class; and the two site coefficients ($F_a$, $F_v$), which depend on the spectral response accelerations and the site class. Knowing the site coordinates and the site class, the remaining items are readily determined by the structural or civil engineer using the USGS, Earthquakes Hazards Program web site. Thus, the site class is the only information the Project Geotechnical Consultant needs to provide.

CBC Section 1613.5.2 states, "*When the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the Building Official or geotechnical data determines that Site Class E or F soil is likely to be present at the site.*" If a site class other than D is recommended, the Project Geotechnical Consultants should discuss and support the recommendation with site-specific data. Without site-specific measurements, it is not acceptable to use a classification inconsistent with the site-condition classification based on correlations between geologic units and the average S-wave velocity ($V_s$) in the upper 30 meters, developed by the CGS and published in the SCEC Phase III report (Wills and others, 2000). The SCEC report is available on the SCEC website.

If the structural design is based on CBC dynamic lateral-force procedures, the Project Geotechnical Consultants should provide an appropriate response spectrum curve and recommendations for vertical as well as horizontal acceleration. The vertical component is often taken as two-thirds of the horizontal component. Studies have shown, however, that the ratio of vertical-to-horizontal components is strongly dependent on oscillator period, source-to-site distance, and local site conditions (Bozorgnia, Campbell, and Niazi, 1999). Project Geotechnical Consultants should include in the geotechnical engineering report a discussion of the rationale for selecting accelerations when developing the response spectra.

## 5.4 Engineering Geology Conclusions

The scope of conclusions provided in engineering geology reports depends upon the site-specific soils and geologic conditions and the proposed development. Conclusions are to be based on a geologic model substantiated by appropriate geology data and analyses. The project engineering geologist should disclose and address any geologic hazards that are on or adjacent to the site and that affect the site, and should discuss and address the effects of the proposed development on adjacent properties. The project engineering geologist should provide conclusions regarding the following in engineering geology reports:

- Presence or absence of active faulting across the building site.
- Effects on the site from ground shaking.
- Potential for secondary effects from earthquakes, such as ground rupture, slope instability, liquefaction, seismically induced settlement, and lateral spreading.
- Presence of slumping, landsliding, or other slope instabilities that are on or adjacent to the site and that may affect the site.
- The potential for mud and debris flows and rock falls to adversely affect the proposed development (e.g., for hillside sites).
- Soil and bedrock conditions, including swelling or collapsible soils, that could affect the building site.
- Groundwater conditions and highest anticipated groundwater conditions.
- Feasibility of using an on-site wastewater treatment system on the site (see Section 5.8).
- Subsidence, settlement, and hydrocollapse potential of the soils on the site.
- Expansive soils.
- Excavation methods.
- Potential for earthquake-induced flooding (e.g., tsunamis).
- Presence of man-imposed conditions encountered at the site (e.g., undocumented fill, abandoned water wells, swimming pools, septic systems, etc.).

## 5.5 Engineering Geology Recommendations

As a minimum, the project engineering geologist is to:

- Provide setbacks from faults to habitable structures in accordance with the requirements of CGS Special Publication 42, Fault-Rupture Hazard Zones in California.
- Designate restricted use areas on the property due to the presence of unmitigated geologic/geotechnical hazards on the site or adjacent properties. Delineate the areas on the site plan and geologic maps.
- Locate structures on the site based on existing geologic/geotechnical hazards and/or adverse geologic conditions.

- Provide measures to mitigate geologic hazards on the property. Where an existing geologic hazard on the property affects off-site property, but the existing conditions are not changed, worsened, or otherwise affected by the proposed development, and the hazard does not affect on-site or off-site building areas, the hazard may not need mitigation. The conditions need to be clearly identified and level of risk discussed in the geotechnical engineering report.
- Provide methodology for excavating and moving earth materials and, as necessary the expected rippability of rock materials.
- Provide recommendations for the dispersal of effluent on the subject site based on the analyses and evaluations performed as discussed in Section 5.8.2 of these guidelines.

### 5.6 Subdividing Geologic / Geotechnical Hazards

Landslides exhibiting factors of safety below the City of Malibu's minimum standards, along with the landslides' possible affected areas, are considered geologic hazards and may not be subdivided. Lot lines should be located such that the landslide is located entirely within one lot, or the landslide hazard needs to be properly mitigated. The hazard may not pose a threat to any building areas on the lot containing the hazard or to the adjacent lots. Each lot needs to have a building site suitable for development as determined by City Geotechnical Staff.

### 5.7 Mandatory Building Code Statements

Project Geotechnical Consultants are responsible for providing a complete finding in accordance with Section 111 of the Malibu Building Code for all proposed developments, including on-site wastewater treatment systems. The complete finding should be included with update reports. Section 111 of the Malibu Building Code states that the geotechnical engineering report "*shall contain a finding regarding the safety of the building site for the proposed structure against hazard from landslide, settlement or slippage and a finding regarding the effect that the proposed building or grading construction will have on the geotechnical stability of property outside of the building site.*" See Section 3.6.1 for definitions of building sites. The Project Geotechnical Consultants should provide recommendations to mitigate the hazard(s) to comply with the standards outlined in the City's guidelines.

In areas of active landslides where stabilization systems are needed to satisfy minimum computed safety factors for the construction site, it should be clearly demonstrated that implementation of the proposed stabilization measures on the property will not adversely affect adjacent properties. The direction of landslide movement relative to the orientation of the stabilization system may influence the stabilizing forces and movement of the landslide, including tearing or extension and continued movement of the landslide along the edges of the stabilizing measures (e.g., soldier pile walls) on adjacent building sites. Stabilization of a portion of a landslide on one property could potentially impact adjacent properties by altering the stresses of landslide movement on those properties.

The amount, quality, and uncertainty of geotechnical/geology data are important to defining and analyzing the limits, direction, stabilizing forces, and extent of future movements, as well as providing confidence in the conclusions developed by the Project Geotechnical Consultants. Cut-off walls consisting of bentonite-filled excavations may be recommended for the purpose of reducing the possibility that tearing could affect adjacent properties by forcing failure to occur along weak boundaries.

The stability of the landslide on adjacent properties may be reduced due to the loss of the benefit of the edge effects with the installation of the weak zones along the property lines. Most of the relative, differential deformation in landslides occurs along or near their boundaries. If the location of the landslide edge is altered by creating weak boundaries, damage may occur to an adjacent structure if it is sufficiently close to the new landslide boundary. It should be clearly demonstrated with data and analyses that implementation of the stabilization measures on the property will not adversely affect adjacent properties in accordance with Section 111 of the Malibu Building Code.

Under certain conditions, full mitigation may not be required for all geotechnical hazards (see Section 110.2 of the Malibu Building Code). In these cases, an unqualified Section 111 statement may not be possible. If on-site or off-site geologic or geotechnical hazards are not mitigated and if the project falls into a category of Section 110.2 and therefore does not require full mitigation, the Project Geotechnical Consultants and property owners are to provide the following:

- When all hazards of the building site cannot be fully mitigated to a level that meets code requirements, but the proposed structure is not at risk, then the Section 111 statement needs to be qualified to indicate the level of mitigation that is provided and the remaining level of risk (e.g., the potential extent of damage that may occur).
- The property owners need to sign and record at the County of Los Angeles recorder's office an "Assumption of Risk and Release" (ARR) for the hazard, and to provide a certified copy to City Geotechnical Staff. ARR forms are normally not allowed for new habitable construction (new residences, guest houses, studios, commercial/industrial projects, and multi-family projects). Copies of the ARR form are available at the City Geotechnical Staff counter at City Hall.
- The Project Geotechnical Consultants need to ensure that the property owner is made fully aware of the extent of damage that may occur to the structure or property if the hazard is not mitigated, such that the owners are fully aware of the risks when they sign an Assumption of Risk and Release.

It is critical that the Project Geotechnical Consultants provide specific recommendations regarding foundations, utility lines, wastewater disposal, surface and subsurface drainage, and fills that meet or exceed this objective, and that they clearly explain how each of these recommendations complies with the objective. Recommendations should include mitigation measures to be implemented to repair future distress on the property. Special consideration should be given to utilities and foundations that trend across any cracks or landslide boundaries mapped across the property. Recommendations should also be made as to the types of foundation systems that would be best in an active landslide.

The Project Geotechnical Consultants should provide minimum thresholds that warrant site observations based on observed surface distress. On the basis of the site observations, the consultants should investigate movements and the owner needs to implement the necessary repairs due to continued landslide movement. As appropriate, repairs should be implemented for utility lines, foundation systems, drainage facilities, flatwork, retaining walls, and the on-site wastewater treatment system. The thresholds, as well as specific procedures for investigating distress and evaluating and implementing the necessary repairs, are to be included in a Quality Control Maintenance Manual prepared by the project civil, geotechnical, and structural engineers.

## 5.8 On-Site Wastewater Treatment Systems (OWTS)

The majority of new developments on properties in the City of Malibu are serviced by on-site wastewater treatment systems where grey water and black water (i.e., effluent) generated from the developments are disposed into the subsurface on the property. Three primary designs of effluent dispersal are used within the City by environmental health specialists retained by property owners: seepage pits, leach lines or fields, and drip irrigation dispersal zones. Project Geotechnical Consultants retained by property owners who need to install new OWTS or enlarge existing OWTS need to evaluate how the dispersal of effluent into the subsurface may affect groundwater levels, slope stability, adjacent developments and their OWTS, and whether or not daylighting of effluent may occur on the property or adjacent sites over time.

The following discussions are guidelines for consultants to use when evaluating OWTS from a geotechnical perspective. Project Geotechnical Consultants need to demonstrate that the effluent from the proposed OWTS (including leach fields, seepage pits, or drip irrigation systems) will not adversely affect the stability of the subject site or adjacent properties in accordance with Section 111 of the Malibu Building Code. That is, it should be demonstrated that the dispersal of effluent into the subsurface on the property will not contribute to landsliding, settlement, or slippage and that the disposal of effluent will not adversely affect adjacent properties. Project Geotechnical Consultants are expected to provide a written statement in accordance with Section 111 of the Malibu Building Code regarding the OWTS.

### *5.8.1 Submittal Requirements*

Applicants are responsible for submitting the following information regarding the OWTS to City Geotechnical Staff:

- One copy of the percolation/infiltration test report(s).
- One copy of supporting geotechnical engineering report(s). (See discussion below.)
- One copy of the OWTS plans (minimum scale to fit on an 11 x 17 page).

All reports should be submitted as hard copy and as a searchable PDF file on a CD to both the City Geotechnical Staff and the Environmental Health Staff for review.

### *5.8.2 Effluent Dispersal (seepage pits, leach lines, and leach fields*)

Project Geotechnical Consultants should incorporate the following in project reports, depending on the site conditions:

- The current static and highest anticipated groundwater levels under the subject site, citing appropriate references/data. It should be clearly demonstrated that the bottom of the disposal areas maintains the minimum separation distance from groundwater.
- Interpretations of geologic structure and/or stratigraphy (specifically, lithologic changes across the site that could affect hydraulic conductivities across the site) and discontinuities such as fractures, faults, clay seams, and joint systems (for bedrock sites) or variations in sandy, clayey, and silty soils across the site (for terrace or alluvial sites).

- Geotechnical cross-section(s) that depict the proposed development, existing and proposed grades, proposed OWTS, anticipated paths of effluent, current and highest anticipated groundwater levels, and capping depths of seepage pits.
- Surficial, static, and pseudo-static slope stability analyses that include the highest anticipated groundwater levels, as appropriate considering the proposed grades across the property. Saturated zones from effluent disposal need to be evaluated as part of the analyses.
- Sufficient data to substantiate report conclusions regarding the effects of effluent dispersal on groundwater levels under the site, the potential for mounding of groundwater, and the potential for effluent to daylight on the site or adjacent properties (such as on nearby slopes, cut banks, sloping terraces, or on the beach). Provide sufficient data to substantiate the conclusions.

#### *5.8.3 Drip Irrigation Dispersal Zones*

Drip irrigation systems are alternative OWTS that evapotranspirate effluent through vegetation to reduce or minimize percolation of effluent into the deeper subsurface. Treated effluent is dosed into drip lines typically 6 to 12 inches below the ground surface and is used to irrigate plants and trees. Project Geotechnical Consultants should incorporate the following in supporting geotechnical engineering reports, depending on the site conditions:

- The current static and highest anticipated groundwater levels under the subject site, citing appropriate references/data. It should be clearly demonstrated that the bottom of the disposal areas maintains the minimum setbacks from groundwater.
- The feasibility of dosing water into the subsurface. Evaluate the amount of infiltration into the subsurface during the rainy months using the dosing rate, peak daily effluent discharge from the development, maximum daily average rainfall during the winter months, and lowest evapotranspiration rates during the winter months. The consultant needs to address the potential for mounding of groundwater to occur from the drip irrigation dispersal system and needs to provide recommendations to prohibit migration and infiltration where such potential exists.
- Geotechnical cross-section(s) that depict the proposed development, existing and proposed grades, proposed OWTS, anticipated paths of effluent, and current and highest anticipated groundwater levels.
- Surficial, static, and pseudo-static slope stability analyses that include the highest anticipated groundwater levels on hillside sites. Saturated zones from effluent disposal need to be evaluated as part of the analyses.

The landscape plans need to be approved by the City Biologist. Deeper rooting plants may be necessary to ensure that the surface of the slope that supports the drip dispersal field remains stable.

#### *5.8.4 Reduced Setbacks*

If septic/treatment tanks and/or effluent dispersal on a property are to be sited within the plumbing code's minimum allowable horizontal setback to structures, then the project geotechnical, architectural, and structural consultants should provide an evaluation and recommendations to mitigate any adverse affects that effluent may have on the adjacent improvements. Such recommendations may include, but are not limited to, deepening foundations, shoring, and waterproofing.

Any subdrains or backdrains that are proposed to remove groundwater adjacent to structures, including but not limited to retaining wall and basement back drains, need to meet the setbacks from effluent disposal areas in accordance with the plumbing code (which are currently 15 feet). If these setbacks cannot be maintained, the Project Geotechnical Consultants, architect, and structural engineer need to evaluate the geotechnical conditions as to whether or not effluent could migrate into the subdrains, and the consultants need to provide recommendations to prohibit such migration and infiltration where such potential exists.

All reports should be submitted as hard copy and as a searchable PDF file on a CD to both the City Geotechnical Staff and the Environmental Health Staff for review.

# 6 GEOTECHNICAL ENGINEERING GUIDELINES

## 6.1 Exemptions and Requirements for Small Additions and Remodels

In lieu of retaining Project Geotechnical Consultants, conservatively assumed values may be used in designing foundation systems for small additions and remodels (as defined in Section 3.2) that are supported on shallow foundations and located in flat areas outside of liquefaction zones, landslides, and fault zones. Such sites may use minimum Malibu Building Code values for soil bearing capacity and for lateral resistance, assuming a footing embedment depth below lowest adjacent grade of at least 24 inches and assuming that the slab and foundation are structurally designed in accordance with 2010 CBC Section 1808.6. All foundations need to be continuous. The new footings should be doweled into the existing footings. Dowels should be placed across cold joints and slabs should be doweled into foundations. Continuity of existing grade beams needs to be maintained at the garage door and at crawl holes. Footings should be supported on a minimum of 2 feet of certified compacted fill using the same type of soil materials that support existing footings, or footings should be supported on competent older alluvium or bedrock.

The current Malibu Building Code requires that a minimum 5 percent positive drainage away from foundations for a minimum distance of 10 feet be established and maintained. All roofs should be guttered and the run-off conducted to a drainage system or natural drainage course in nonerosive devices. Landscaping adjacent to foundations should be limited to native plants that need a minimum of hand watering. Planters adjacent to the foundation should have waterproof sides and bottoms and should have a drainage system that conducts water away from the foundation. A French drain system adjacent to the foundation is highly recommended. Trees may not be planted closer than 15 feet from the foundation.

## 6.2 Laboratory and / or In Situ Test Data

Geotechnical engineering reports need to contain sufficient in situ and/or laboratory testing data to characterize the subsurface material(s) and to substantiate calculations from which any conclusions or recommendations are derived. The report should include descriptions of the sample preparation and testing procedures, and should reference applicable ASTM procedures. Laboratory procedures should be selected that are representative of the site conditions during and after site development from a geotechnical engineering perspective.

Numerical data is to be presented for all laboratory testing, plots, or illustrations of laboratory data. Data plots should be submitted as necessary to substantiate the Project Geotechnical Consultants' conclusions and recommendations. Typical examples of numerical and graphical presentations of laboratory data include dry density and moisture content of "undisturbed" samples, compaction curves showing maximum dry density and optimum moisture content, and grain-size analyses (e.g., sieve and hydrometer) for representative samples.

#### *6.2.1 Direct Shear*

The determination of shear strength of undisturbed and recompacted samples is extremely important. For all analyses including shear strength, data should be obtained using on-site samples. The report of shear strength data should include plots of normal stress versus shear resistance and plots of shear resistance versus displacement. If the normal stress is not constant during the shear test, plots of normal stress versus shearing resistance should also be provided. Shear strength test results are to be demonstrated for both peak and ultimate conditions. The number of shear tests should be appropriate to evaluate the variability of the strength for a given material and variability between material types encountered for the project. Refer to
Appendix B for further discussion and guidelines on direct shear testing.

#### *6.2.2 Consolidation*

If newly placed compacted fill is intended to support loads, then consolidation tests should be performed on representative undisturbed samples and remolded samples to represent fill materials. Hydroconsolidation tests are to be performed on undisturbed and remolded samples as needed to characterize the foundation soils. An adequate number of consolidation tests need to be performed to evaluate hydrocollapse potential as well as soil compressibility. It is expected that laboratory testing include both: (1) odometer tests in which hydrocollapse is simulated, and (2) appropriate soil index testing (e.g., grain-size, Atterberg Limits, dry density, and moisture content). The evaluation of hydrocollapse potential should extend to a depth of about 50 feet or to the depth of groundwater, whichever is less. Tests should be performed at pressures typical of the magnitude to be encountered under design conditions at the depth of the sample. A discussion regarding potential risks for hydrocollapse is provided in Section 6.9. When soft to firm clayey or silty soils are present and/or anticipated, adequate time-rate consolidation testing should be performed.

#### *6.2.3 Index Tests*

An adequate number of soil index tests need to be performed to characterize the expansive nature of the material. At a minimum, the near-surface soils or the material at the basement level should be characterized with expansion index tests or with a weighted plasticity index.

#### *6.2.4 Corrosivity*

Elementary laboratory testing of the on-site materials should be performed to provide a preliminary evaluation of soil corrosivity. The chemical properties of soils can have a deleterious effect on building materials resulting from chemical reactions and electro-chemical processes. Tests that should be

performed to provide a preliminary evaluation of these potential hazards include pH, chloride and sulfate contents, and resistivity. Refer to
Appendix C for further discussion and guidelines.

#### *6.2.5 R Value*

Tests to determine the R value of potential subgrade materials should be performed when providing pavement sections. When pavement sections are based on presumed R values, confirmation tests need to be performed during grading. Adequate notes should be included on grading plans to ensure that these tests are performed.

#### *6.2.6 Other Tests*

Other tests such as unconfined consolidation or triaxial testing should be considered to supplement the above test schedule.

### 6.3 Groundwater

The Project Geotechnical Consultants are responsible for addressing the potential variations in groundwater conditions underlying the site and how the groundwater conditions affect the existing site and proposed development. The term groundwater, as used in this document, refers to all subsurface water (e.g., seepage, perched water, etc.). The Project Geotechnical Consultants should address how the proposed development may affect future groundwater conditions and how these changes may affect the development. For all analyses, the Project Geotechnical Consultants should use the highest anticipated groundwater levels that affect the strength of the materials under the site. At a minimum, the following items should be addressed and incorporated in the groundwater assessment:

- Groundwater data such as: current water level or piezometric head, seasonal changes, and historical-high and historical-low water tables. For new construction projects, subsurface exploration is used to determine current groundwater levels underlying the site.
- The effects of effluent, from existing or proposed on-site wastewater treatment systems, on groundwater levels.
- The effects of irrigation on groundwater levels.
- The effects of potential heavy rainfall (such as the 1969, 1978, 1980, 1983, 1986, 1993, 1995, 1998, 2001, 2005, and 2011 rainfall years in California).
- The potential for geotechnical hazards associated with groundwater (such as seepage, shallow groundwater, springs, and artesian conditions).

### 6.4 Slope Stability Analyses

Reports should address the stability of slopes that may affect the site or that the proposed development may affect. Stability should be analyzed along all critical cross-sections where development includes or is adjacent to slopes with a gradient steeper than 4:1 (horizontal:vertical). Stability analyses are also necessary for slopes that have a 4:1 gradient or flatter if the slope includes a geologic/geotechnical hazard such as a landslide. The critical cross-section is defined as the slope with the most adverse combination of conditions, such as the steepest gradient, highest slope, most adverse geologic conditions, groundwater conditions, weakest soils and bedrock, etc. More than one cross-section may need to be evaluated on a geotechnically more complex site. The critical failure surface should be identified by search techniques and evaluated on each cross-section. Slope stability evaluations are to conform with the guidelines published by SCEC entitled *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California, Organized through the American Society of Civil Engineers, Los Angeles Section (ASCE-LA)* (Blake, et al., 2002) and with SP 117A (CGS, 2008). Subsurface geologic and groundwater conditions should be sufficiently evaluated and illustrated on geologic cross-sections. The subsurface geologic and groundwater data should be utilized by the Project Geotechnical Consultants for the slope stability analyses.

If on-site wastewater or storm water disposal as part of an urban mitigation plan exists or is proposed, the slope stability analyses need to include the effects of the effluent plume or infiltrating storm water on slope stability. The limits of the effluent plume define a phreatic surface. For a steady-state condition, equipotential lines exist within the effluent plume, and the magnitude of the pore water pressure is not necessarily zero within the effluent plume. If a potential failure surface is within the effluent plume then the effective stress is affected. The anisotropy of the material, as well as fracturing and other features of the material through which the effluent flows, affects the size of the effluent plume. The Project Geotechnical Consultants should evaluate the bedrock materials encountered in the subsurface exploratory excavations and provide reasonable conclusions regarding the ability of the bedrock materials to transmit effluent and regarding the anticipated paths of effluent from the disposal areas into the subsurface.

Reports for sites in hillside areas need to address the potential for surficial instability, debris/mudflow, rockfalls, and soil creep on all slopes that may affect the proposed development or be affected by the proposed development. Stability of slopes along access roads needs to be addressed, and it should be demonstrated that access roads have safety factors of at least 1.25 for static conditions. Where access roads have minimum safety factors between 1.25 and 1.5 for static and acceptable displacements for seismic conditions, an Assumption of Risk and Release for geotechnical hazards needs to be signed and recorded along with the identification of restricted use areas.

Mitigation measures should be provided for utility lines if differential movements associated with potential sliding masses could damage such lines.

For flatter slopes, the critical failure surface may not be through the toe of slope. Therefore, search limits near the toe of slope need to be large enough to confirm that the minimum computed safety factor has been found. The Project Geotechnical Consultants need to expand critical failure surfaces that are near the search limits, or the consultants need to provide a detailed discussion in the report to justify where the failure envelopes are crowding the initiation limits.

### *6.4.1 Interpretation of Geologic Conditions*

The project engineering geologist is responsible for interpretation of geologic conditions, for clearly explaining the interpretations, and for supporting them with adequate subsurface exploration and laboratory data. Part of the process in developing an interpretation is explaining plausible models and eliminating models that may seem to be plausible but are flawed. Therefore, when the project engineering geologist explains the interpretation, comments and discussion should be provided to rule out other seemingly plausible interpretations of the data.

Geologic cross-sections should clearly show an interpretation of the site stratigraphy across the section and should not be limited to the interpretation only at exploration points. It is expected that interpretations include bedding, faults, material types, folds, pervasive joint sets or fracture patterns, shallow failures or slumps, and landslides.

#### *6.4.2 Shear Strength Selection for Slope Stability Evaluation*

The Project Geotechnical Consultants should describe their selection of shear strength parameters for the various materials to be used in the slope stability analyses, including a discussion explaining the selection of strength parameters for site characterization and how the shear strength testing methods used are appropriate in modeling field conditions and long-term performance of the subject slope.

The Project Geotechnical Consultants are referred to
Appendix B for further discussion on shear strength selection for slope stability evaluation and on the impact of defects on shear strength selection.

#### *6.4.3 Soil Creep*

The Project Geotechnical Consultants are responsible for addressing the potential effects of soil creep where any proposed structure is planned in close proximity to an existing fill slope or natural slope. The potential effects on the proposed development should be evaluated and mitigation measures proposed, as appropriate, including appropriate setback recommendations.

#### *6.4.4 Surficial Stability*

Surficial slope stability refers to slumping and sliding of near-surface sediments and is generally most critical during the rainy season or with excessive landscape watering. The assessment of surficial slope stability should be based on analysis procedures for stability of an infinite slope with seepage parallel to the slope surface or an alternate failure mode that would produce the minimum factor of safety. The minimum acceptable depth of saturation for surficial stability evaluation is 4 feet. Project Geotechnical Consultants may use shallower depths of saturation for the analysis if the consultant can justify a thinner zone of saturated materials on the basis of the on-site data. All conclusions should be substantiated by appropriate analyses and data. Shear strengths should be based on fully (i.e., 100 percent) saturated samples tested at effective overburden pressures representative of the upper 4 feet of material. Residual shear strengths should be used unless the Project Geotechnical Consultants can justify the use of higher shear strengths. Additional comments concerning shear strengths and safety factors are provided in the ASCE-LA guidelines on slope stability (Blake, et al., 2002).

Surficial stability analyses should be performed under rapid draw-down conditions where appropriate (e.g., for debris and detention basins).

### *6.4.5 Gross or Deep-Seated Stability*

Gross stability includes rotational and translational deep-seated failures of slopes or portions of slopes existing within or outside of the proposed development. Project Geotechnical Consultants are responsible for conforming to the following guidelines, in addition to those in the ASCE-LA and CGS (2008) documents, when evaluating slope stability:

- Stability should be analyzed along cross-sections depicting the most adverse conditions (e.g., highest slope, adverse bedding planes, and steepest slope). Analyses are needed for different conditions and may need more than one cross-section to demonstrate which condition is most adverse. The critical potential failure surfaces (i.e., those with the lowest computed safety factor) on each cross-section and for each mode of potential failure need to be identified, evaluated, and plotted on the large-scale cross-section. The coordinates of the potential slide planes within the lowest factors of safety for each mode of failure (e.g., rotational, block, translational, etc.) should be depicted on the cross-sections used in the analysis.
- The minimum long-term factor of safety is 1.50 for all new construction of habitable structures, including single-family residences, guest houses, studios, multi-family residential projects, commercial projects, and swimming pools. Pseudostatic factors of safety should be assessed under the guidelines of Section 6.5.
- When areas are adjacent to or near existing landslides, the Project Geotechnical Consultants need to discuss why the subject site has an acceptable computed safety factor against sliding while adjacent or nearby areas are potentially unstable or do not exhibit satisfactory safety.
- The minimum static factor of safety is 1.25 for temporary excavations (during construction) and for slopes associated with access roads (if the slope does not affect the stability of the building pad).
- If the computed factors of safety are less than the above minimums, mitigation measures are necessary to bring the factor of safety up to at least the minimum level. Alternatively, the project may be redesigned so as to achieve the minimum factors of safety for static and pseudostatic conditions.
- Long-term stability should be analyzed using the highest known or anticipated groundwater level based upon a groundwater assessment performed under the guidelines of Section 6.3.
- The stability analysis models need to consider and incorporate all adverse geologic conditions such as joints, fractures, shears, faults, bedding planes, folds, clay seams, gouge zones, clay beds, and landslide rupture surfaces.
- Circular and noncircular potential slip surfaces should be used, as appropriate.
- If units exhibit anisotropic strength conditions or planes of weakness that are inclined at angles ranging from nearly parallel to the slope to horizontal, safety factors for translational failure surfaces should be calculated.
- Tension cracks and anticipated external loading should be modeled, as appropriate.
- The most critical potential failure surface needs to be well within the search limits. Crowding of the search limits needs to be justified by the Project Geotechnical Consultants.
- For the block-sliding mode, consideration should be given to the potential failure mode passing through the toe of the slope. See Figures 9.d through 9.1f of the ASCE-LA guidelines (Blake, et al., 2002).

## 6.5 Seismically Induced Slope Instability

Seismically induced slope stability analyses are necessary in all cases, including remedial repairs, where static or gross slope stability analyses are performed. Pseudo-static analyses may be performed if a calibrated seismic coefficient of 0.35 is used and a factor of safety that equals or exceeds 1.0 is demonstrated. If those pseudo-static analyses do not result in factors of safety that equal or exceed 1.0, then seismic displacement analyses are to be performed. The seismic slope stability evaluations need to conform with the guidelines published by ASCE-LA entitled *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California, organized through the American Society of Civil Engineers, Los Angeles Section (ASCE-LA)* (Blake, et al., 2002) and with SP 117A (CGS, 2008), except that the displacement analyses need to utilize the updated Bray and Travasarou (2007; 2009) and Rathje and Antonokos (2011) methods. If direct integration of seismic records is to be performed (e.g., using a fully coupled equivalent-linear computer code like SLAMMER), a significant number of ground motions (i.e., over 30) that are consistent with the entire design spectrum for the site — as well as duration and fault type — need to be selected, and the selection process needs to be thoroughly documented. The results from utilization of the Bray and Travasarou (2007) and Rathje and Antonokos (2011) methods should be averaged when estimating empirical seismic displacement for design. If the computed seismic displacements are less than 5 cm, the slopes may be considered acceptable. If the computed seismic displacements are greater than 15 cm, then remedial measures are needed to reduce the estimated displacements to acceptable levels. Computed seismic displacements between 5 and 15 cm are considered acceptable for nonstructural areas or areas where structural improvements are designed to resist those estimated displacements.

Although the high strain rates commonly encountered in earthquake loading may result in higher shear strengths, the effects of cyclic degradation, particularly in brittle materials, can be expected to reduce those shear strengths. Consequently, seismic analyses of previously undisturbed materials may be performed using peak shear strength, but the seismic analyses should also be checked for adequate post-peak performance using fully softened/ultimate shear strengths. Residual shear strengths should be used when performing seismic slope stability analyses of bedded materials that may have experienced flexural slip due to folding or for previous landslide slip surfaces.

## 6.6 Construction Stability

The Project Geotechnical Consultants are responsible for evaluating the construction stability (i.e., temporary stability) during grading, foundation construction, and retaining wall excavations. See Section 7.6 for a discussion of minimum factors of safety, shoring, and temporary excavations.

## 6.7 Liquefaction

Reports should address the potential for liquefaction at the site (including settlement, lateral spreading, and surface manifestations) and should identify whether the site is within a Liquefaction Hazard Zone based upon the current Seismic Hazards Maps published by the CGS. The Project Geotechnical Consultants should evaluate the liquefaction potential in general accordance with the SCEC document entitled *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California* (Martin and Lew, 1999), incorporating recent modifications (including current SP 117A – CGS, 2008; Youd et al, 2001; Seed, Cetin, Moss, Kammerer, Wu, Pestana, Riemer, Sancio, Bray, Kayen, and Faris, 2003; Idriss and Boulanger, 2004; Boulanger and Idriss, 2006; Boulanger and Idriss, 2007). Deviations from the guidelines and modifications need to be described and justified. These methods do allow for screening.

When an adequate factor of safety against liquefaction cannot be demonstrated (where the factor of safety against liquefaction should exceed 1.25) and it is determined that the effects of liquefaction exceed tolerable levels, the Project Geotechnical Consultants should recommend mitigation measures that minimize the effects (e.g., preventing structural collapse, injury, loss of life).

In the case of one- and two-story single-family residences not within a Liquefaction Hazard Zone, if the Project Geotechnical Consultants do not consider liquefaction to be a hazard at the site, then the consultants are responsible for providing a rational basis for that conclusion. A rational basis may consist of the Project Geotechnical Consultants being of and stating the opinion that the depth to groundwater, density, and age of underlying materials, or other factors (which are all appropriately referenced) are sufficient to preclude the risk of liquefaction.

Comprehensive liquefaction studies are not necessary for swimming pools, spas, remedial repair projects, additions, or remodel projects, but the potential for liquefaction should be discussed. If the site is subject to liquefaction or is within a Liquefaction Hazard Zone, it is the responsibility of the Project Geotechnical Consultants to ensure that the report clearly informs the property owner of the risk and of the potential consequences to the proposed improvements if the liquefaction hazard is not mitigated.

The City guidelines require geotechnical studies for all new habitable structures as defined in earlier sections. The Seismic Hazards Mapping Act defines "residential project" subject to the Act as developments of four or more dwellings, however the Act does not prohibit the City from establishing guidelines that are more restrictive than those established by the Act (Chapter 7.5 Section 2624 of the Public Resources Code).

Remodels and additions to existing structures, as well as accessory nonhabitable structures, may be exempt when the homeowner signs and records an "Assumption of Risk and Release" (ARR) for liquefaction hazard. For those exempted projects, it is expected that the Project Geotechnical Consultants provide a discussion of the risk in the report.

### 6.8 Seismically Induced Settlement

Portions of the City of Malibu are underlain by dense Pleistocene-age and younger deposits. For a magnitude 7.5 earthquake with a peak ground acceleration of 0.5g, the computed seismically induced settlement in a 50-foot-thick, dense, dry, granular deposit with an SPT blow count ($N_{60}$) of 30 is less than ½ inch. The computed settlement reduces to about ¼ inch if the blow counts are 40. If, however, the groundwater is at the surface, the computed seismically induced settlement is about 4½ inches in a deposit with an SPT blow count of 30 to a depth of 50 feet, decreasing to about ½ inch when the blow counts increase to 35 and to a negligible amount if the blow counts are 40. If the groundwater is at a depth of 25 feet, the computed seismically induced settlement is about ½ inch in a deposit with an SPT blow count of 30 to a depth of 50 feet.

When selecting the depths of borings to quantify seismically induced settlement to a depth of 50 feet, consideration can be given to geologic conditions at the site. It is necessary to extend the borings only to a depth where the deeper soils are expected to be sufficiently dense— based on geology, exploratory data in the area, and experience— that the estimated seismically induced settlement to the depth explored plus the amount anticipated for materials between the depth explored and 50 feet, assuming the highest anticipated groundwater, is within tolerable amounts or is adequately mitigated. The presentation of results within the geotechnical engineering report should clearly present the rationale and supporting data when site-specific data is not determined to a depth of 50 feet.

The fact that a site has been subjected to previous significant earthquakes does not preclude additional seismically induced settlement from occurring. The argument that seismically induced settlement will not occur due to having experienced previous shaking is not acceptable, by itself, as a reason for no risk due to seismically induced settlement.

### 6.9 Hydrocollapse

Hydrocollapse of subsurface materials is a decrease in volume (i.e., collapse) when water is added to those materials at a constant load. Materials prone to hydrocollapse include man-made fills, wind-laid deposits, and alluvial fan and mudflow sediments deposited during flash floods. The composition of materials most susceptible to hydrocollapse potential include silty to clayey sands that exhibit a degree of cementation. Some of the major causes of infiltrating water are pipe breaks, excessive landscape watering, poor drainage, and rising groundwater levels. Acceptable measures to mitigate hydrocollapse risk include removal and recompaction of the more susceptible material, checking utility lines for leaks and promptly repairing such leaks, maintaining site drainage and drainage devices, and properly managing landscape watering to reduce the likelihood of water infiltrating deeper materials.

Because of their cemented and contractive nature, collapsible soils are not usually as susceptible to disturbances caused by using samplers with large area ratios or by the vibrations of driven samplers. Although sample disturbance may impact the measured hydrocollapse in some materials, sample disturbance is not acceptable, without supporting data and discussion, as a reason to dismiss data that shows significant hydrocollapse potential.

The need for mitigation of hydrocollapse risk should be based on the magnitude of potential total and differential settlements, not on a specified magnitude of strain. Laboratory evaluation of hydrocollapse potential needs to be performed at pressures typical of the magnitude to be encountered in the field. Geotechnical engineering reports need to recognize the potential for risk of hydrocollapse, inform property owners of such risks, and recommend mitigating measures.

### 6.10 Expansive Soils

Soils with an expansion index (EI) of more than 20 are considered expansive and may be subject to large volume changes with changes to the moisture content of the soil, causing foundation and slab uplift with increasing moisture and settlement with decreasing moisture. It is the responsibility of the Project Geotechnical Consultants to provide mitigation measures for conditions with an expansion index of more than 20.

### 6.11 Settlement / Heave

Foundation and slab movements may result from settlement induced by seismic shaking and/or compression of supporting materials caused by live and dead loads of the foundations, settlement of compacted fill and underlying materials due to the weight of compacted fill, and swell or hydrocollapse of supporting materials if moisture infiltrates these materials. The Project Geotechnical Consultants are responsible for analyzing and estimating future total and differential movements of all structures such as foundations, slabs, and pipelines, as well as any engineered fills that will be supporting those structures. The subsurface profiles used for settlement analysis should be shown in cross-section and be substantiated by subsurface data. Settlement analysis calculations should be submitted. If professional judgment is used in addition to the calculated settlement or to modify the calculated settlement, then the Project Geotechnical Consultants should provide the justification or rationale upon which the judgment is made.

It is expected that the magnitude of total and differential settlement be provided, along with the computations. Where significant settlement is anticipated, the report should include an estimate of the time for settlement to be 90 percent complete, along with related computations. When estimating vertical movement, Project Geotechnical Consultants should consider, at a minimum:

- Seismically induced settlement. (See Section 6.8.)
- Compression of the fill materials due to their own weight.
- Compression/consolidation of subsurface materials underlying fill.
- Secondary consolidation, if it exists, of both fill and underlying subsurface materials.
- Hydrocollapse of fill and underlying subsurface materials. (See Sections 4.4.2, 6.2.2, and 6.9.)
- Settlement of foundations due to dead and live loads.
- Potential movement due to swelling (i.e., expansive) soils (where EI > 20).

A settlement-monitoring program should be implemented during and after construction in situations where the anticipated settlement of fill and underlying materials, due to the added weight of fill, exceeds 1 inch (e.g., thick fills or fills overlying soft materials). Settlement monitoring should consist of surface monuments and subsurface settlement plates.

For additions, the Project Geotechnical Consultants should discuss the potential for and the potential impacts of differential settlement between the existing structure and the addition.

## 7 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

The following comments are intended to serve as a guide to the Project Geotechnical Consultants. These are items that reviewers address when reviewing geotechnical recommendations. The list, however, is not intended to be exhaustive. A number of additional issues are identified in Sections 5 and 6 of these guidelines. The Project Geotechnical Consultants should address each of the issues with supporting information. The reviewers do not assume that unmentioned items are unimportant or do not need mitigation, even if, in the opinion of the reviewer, such is the case. The Project Geotechnical Consultants are responsible for identifying and discussing each issue and for providing mitigation measures, as necessary.

## 7.1 Shallow Foundations (e.g., wall and spread footings)

Design of shallow foundations should include the following recommendations as applicable:

- Allowable bearing pressure. When the allowable bearing pressure exceeds 3000 psf, provide computations to demonstrate that the computed safety factor equals or exceeds 3 for static loading conditions. Increases greater than one-third in bearing pressure for temporary (e.g., wind or seismic) loading are not acceptable unless it can be demonstrated that foundation movements under such loads do not cause unacceptable distress.
- Minimum footing embedment depth below lowest adjacent grade of at least 12 inches for one-story wood-frame structures and at least 18 inches for two-story wood-frame structures. A slab-on-grade floor with a roof above is considered a one-story structure. A slab-on-grade floor with a second floor and roof above is considered a two-story structure.
- Minimum slope setback (2010 CBC Section 1808.7).
- Estimated total and differential settlement.
- Resistance to lateral loads (including passive soil resistance and/or base friction) specified as ultimate or allowable with recommended safety factors that equal or exceed 1.5. A one-third increase in resistance for temporary (e.g., wind or seismic) loading is not acceptable for passive and base friction resistances, unless the safety factors for static conditions exceed 2. If the recommended passive or sliding soil resistance relies on a cohesive strength component, the shear strength parameters are to be based on drained tests at overburden pressures representative of the application (e.g., less than 250 psf for shallow footings) and on samples that have been soaked and have a degree of saturation of 100 percent. Cohesions measured on partially saturated (i.e., <100 percent) samples are not acceptable to compute lateral resistances for shallow footings.
- Recommendations for compacted fill pads or over-excavation and recompaction.
- If existing footings may be subjected to additional loads: recommendations for underpinning existing foundations, or geotechnical criteria for accepting the existing foundations to carry additional loads.

## 7.2 Deep Foundations

### *7.2.1 General Comments*

Design of deep foundations should address each of the following as applicable:

- Allowable vertical capacities (including compression and uplift) as a function of foundation size (i.e., width, diameter, and depth). Specify skin friction or end bearing, and safety factors used. When computing allowable vertical capacities, geotechnical safety factors must equal or exceed 3 for driven piles and the end bearing component of drilled shafts, and geotechnical safety factors must equal or exceed 2 for shaft resistance of drilled shafts. Where load tests or pile driving analyses are performed to verify the results, lower safety factors may be used at the discretion of the Project Geotechnical Consultants (ASCE, 1993; O'Neil and Reese, 1999).
- Pile or caisson-tip elevations corresponding to minimum depths of embedment.
- Feasible pile and/or caisson types.
- Recommendations for installation of deep foundations.
- Potential for negative skin friction and effects on allowable vertical loads.
- Lateral resistance from earth pressures. The lateral resistance for piles is generally taken as twice that for a wall to account for three-dimensional effects. It is expected that recommended allowable lateral

loads be in accordance with Section 1810.2 of the CBC. Recommended lateral resistance of pile groups and the minimum pile spacing for the recommendations need to be supported by analyses and recent references (e.g., Reese and Van Impe, 2001). Pile spacing of three diameters (as measured center-to-center) may be necessary before the piles act independently (e.g., Reese and Van Impe, 2001). The lateral resistance provided in the direction of the descending slope may be less than that for a level area. Project Geotechnical Consultants are responsible for providing calculations that support the recommendations for lateral resistance of piles. At shallow depths, the cohesive component has a strong influence on the computed equivalent fluid unit weight for lateral resistance. For piles that extend to much greater depths than shallow foundations, the relative influence of the cohesive component decreases. Therefore, the computed lateral resistance of piles needs to be based on representative depths appropriate for the conditions.

- Forces acting on the piles and pile caps resulting from external loads, including lateral spreading, soil creep, and surcharge from adjacent structures, or lateral load to achieve the appropriate factor of safety against slope failure.

When the Project Geotechnical Consultants do not compute deflections of laterally loaded piles under design loads, the geotechnical engineering report should include recommendations for such computations by the structural/civil engineer. Adequate consideration needs to be given to the potential effects of a cracked section on the lateral behavior of concrete piles.

### *7.2.2 Considerations for Computations of Axial Capacity*

Methods of analyses to evaluate axial capacity should be appropriate for the type of pile being considered. For caissons, adhesion factors should be no greater than one and adequately supported with references (O'Neil and Reese, 1999; Kulhawy, 1991). O'Neil and Reese (1999) have divided materials into five categories for purposes of computing shaft resistance of drilled shafts:

- Cohesive soils are fine-grained materials with undrained shear strengths, $s_u$, < 5 ksf.
- Granular soils are coarse-grained materials with standard penetration sampler blow counts, N, < 50.
- Cohesive (i.e., fine-grained) materials with $s_u$ > 5 ksf and < 50 ksf are intermediate geomaterials.
- Cohesionless (i.e., coarse-grained) with N > 50 are intermediate geomaterials.
- Rock are materials with $s_u$ > 50 ksf.

Methods are described by O'Neil and Reese (1999) for computing the shaft resistance, as well as end bearing, for each material type. Other methodologies may be used, but the shaft resistance for any submerged coarse-grained materials should be based on the submerged unit weight of the material. In the case of cohesive soil or rock when the submerged unit weight is not used, undrained shear strength parameters need to be used. The Project Geotechnical Consultants needs to clearly state and provide supporting data (not opinions) regarding the category of the material that is supporting the piles, the strength and other properties used to characterize the materials for purposes of computing the axial capacity, and the methods of analyses.

### *7.2.3 Soil-Pile-Structure Interaction during Seismic Events*

The soil-pile-structure interaction during a seismic event is a complex phenomenon, even when the supporting soil is not subject to liquefaction. The Project Geotechnical Consultants are referred to Appendix D of these guidelines for further discussion of this topic.

### *7.2.4 Foundation Loading - Caltrans Guidelines*

Caltrans issued a publication in early 2011 entitled *Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading*. Project Geotechnical Consultants may use this method when evaluating sites subject to lateral spreading. Caution should be exercised and the Project Geotechnical Consultants need to justify all variables used in the evaluation. The Caltrans document may be downloaded from the Caltrans website.

### *7.2.5 Micro-Piles in Wave Uprush Zone*

Micro-piles proposed on the beach, specifically in the wave uprush zone, encounter conditions and concerns not normally found landward of the wave uprush zone, including but not limited to:

- Wave erosion of the sand in the uprush zone, thus exposing the shaft of the micro-pile and increasing the micro-pile's laterally unsupported length. Scour of the surrounding beach sand changes the column action of the micro-pile and reduces load-bearing capacity of the micro-pile.
- Wave uprush forces on the micro-piles and the supported structure.
- Abrasion of the relatively small diameter piles due to sand and cobbles carried in the wave uprush.
- Impact loads caused by floating storm debris such as floating timbers, floating timber piles, or other debris that may break off of other coastal structures during storms.
- Corrosion concerns due to the marine environment.
- Littoral drift to the adjoining properties, which may occur during storms or during scoured conditions resulting from storms, and which may be caused under certain circumstances by the density of the micro-pile group.

Per the City of Malibu LCP, new structures or ancillary structures are to be designed to be free-standing and not rely on the protection of a seawall or revetment. This creates a concern where micro-piles are used for those structures in the wave uprush zone for the above reasons. If micro-piles are to be used specifically in the wave uprush zone where they are subjected to beach scour and wave action, then the Project Geotechnical Consultants need to adequately address the above concerns and to show how the above conditions may be remedied for a structure during storm conditions.

## 7.3 Pool Foundations

The Project Geotechnical Consultants are responsible for providing specific recommendations for the design of pool foundations to mitigate adverse settlement or expansion that may lead to leakage. Recommendations for swimming pools and spas should include lateral soil pressures acting on the walls, type of supporting materials, and recommendations to address hydrostatic pressures on the pool shell. Subdrains or hydrostatic relief values are typically acceptable to the City for hydrostatic pressure relief.

***Note:*** Sections 7.3.1 and 7.3.2 apply to all swimming pools and spas, which include any body of water over 24 inches deep.

### *7.3.1 Geology and Geotechnical Engineering*

The Project Geotechnical Consultants need to provide current, applicable, soils and geology reports that address the construction of any proposed pool and/or spa at the project site. The type of report depends on the proposed location of the pool/spa. When the pool or spa is to be located in a geologically stable, flat area, a basic foundation investigation for the proposed pool/spa is acceptable. When the proposed pool or spa is to be located on a hillside area, it is expected that the consultants provide, in addition to the basic foundation investigation, a more comprehensive engineering geology and geotechnical engineering report that evaluates slope stability. In each case the Project Geotechnical Consultants should provide a discussion of all potential geotechnical hazards in accordance with Section 4.4.2 of these guidelines. To obtain more information or to verify what type of report is to be submitted, the project can be discussed with any member of the City Geotechnical Staff during public counter hours on Tuesdays and Thursdays from 8:00 AM to 11:00 AM or by calling (310) 456-2489 extension 306. The two different types of reports are:

- Basic Foundation Investigation (for a flat area), which should contain:
    - Applicable and adequate subsurface information
    - Discussions of hazards
    - Expansive soil details and recommendations
    - Recommendations regarding bearing materials
    - Pool wall design pressures, including seismic pressures
    - Retaining wall design pressures, including seismic pressures
- Engineering Geology and Geotechnical Engineering Report (for a hillside area), which should contain:
    - Applicable and adequate subsurface information
    - Discussions of hazards
    - Analysis of slope stability in accordance with the City's guidelines for geotechnical engineering reports
    - Recommendations for the most appropriate foundation system and specific geotechnical foundation recommendations
    - Expansive soil details and recommendations
    - Pool wall design pressures, including seismic pressures
    - Retaining wall design pressures, including seismic pressures
    - Subdrain and subdrain outlet recommendations and details

#### *7.3.2 Structural Calculation / Standard Plans*

"Standard Plans" may be used (and revised with calculations as necessary) in conjunction with other engineered plans. If "Standard Plans" are used, the following should be provided:

- Wet stamp and wet signature of the engineer.
- Indication or highlighting of the specific applicable details and deletion of those that do not apply.
- Structural calculations to support the standard design.

Structural calculations and details as well as soils and/or geology recommendations should be provided for all conditions not covered by the "Standard Plan." Calculations and details are also needed for decks, other structures, and retaining walls.

The pool design needs to designate the pool walls that are designed as "free standing." Free-standing walls are considered structural concrete and 2011 Malibu Building Code requires a "special inspection" for the shotcrete/gunite process.

The site plan, structural plans, and calculations need to be stamped and signed by the appropriate licensed professionals.

### 7.4 Slab-on-Grade Construction

It is expected that all slab-on-grade construction conform, as a minimum, to 2010 CBC requirements.

To provide a more competent foundation system for single-family residences supported with a slab-on-grade, the concrete for the slab and footings should be poured as a monolithic unit and fiber-reinforced concrete should be used to augment (not to replace) steel reinforcement. Fiber reinforcement improves the tensile strength of concrete and reduces the likelihood of developing shrinkage cracks. The Project Geotechnical Consultants may want to consider supplementing the design recommendations to incorporate the above.

Recommendations for vapor barriers should conform to 2010 CBC requirements and should specify that vapor barriers be a minimum of 10 mils thick. The Project Geotechnical Consultants are responsible for providing specific recommendations to mitigate the effect of expansive soils for all slab-on-grade construction on soils with an expansion index value over 20.

### 7.5 Retaining Structures

#### *7.5.1 Standard Retaining Walls*

Standard retaining walls are those gravity walls consisting of reinforced concrete or masonry block. Geotechnical engineering reports need to provide recommended earth pressures for proposed retaining structures. The design pressures should consider and/or incorporate:

- Type of backfill (e.g., sand, silty sand, sandy clay, or clay) within the wedge defined by a 45-degree line from the heel of the retaining wall footing to the surface. Recommended lateral pressures are to be compatible with the type of backfill within this zone, and with higher pressures associated with soils having a higher fines content. Using stability analyses to estimate lateral pressures can be misleading when nonzero cohesion values are used. The effective cohesion can reduce with time as the materials become wet, resulting in increases in lateral pressures. Care should be exercised when selecting shear strength parameters for computing lateral pressures behind retaining structures, especially those backfilled with fine-grained soils. The Project Geotechnical Consultants are responsible for providing a discussion regarding the selection of shear strength parameters to support their recommendations.

- Existing and proposed surcharges.
- Slopes, adversely oriented geologic features (e.g., bedding, joints, fractures, etc.), and any other factors that may affect the lateral loads.
- Wall restraining conditions. Higher lateral pressures and forces are expected for restrained retaining walls (e.g., basement walls) than retaining walls that are free to deflect.
- Backfill placement recommendations, including temporary excessive equipment loading, if any.
- Appropriate shear strength for backfill materials, in-place materials, and structure support materials.
- Effects and pressures from expansive soils.
- Effects of creep-prone materials.
- Seismic loading as appropriate.

The geotechnical engineering report should contain the following design parameters/items:

- Coefficient of friction against sliding.
- Back drainage design, including size of piping, volume and specification of gravel, and height and width of gravel and earthen backfill.
- Surface drainage recommendations.
- Recommendations to waterproof or damp-proof subterranean walls and floors.
- Seismic loading recommendations.

For walls that are retaining slopes, the Project Geotechnical Consultants need to ascertain the amount of freeboard to prevent sloughing over the wall. For walls that are retaining slopes that are subject to surficial failure, debris flows, and/or mudflow, appropriate design criteria to compensate for the effects of impact from debris or mud flows should be incorporated into the design. Catchments for potential earth flows should also be considered. Supporting calculations should be provided.

In accordance with Section 1803.5.12 of the CBC, the Project Geotechnical Consultants are responsible for providing recommendations for lateral pressures on all basement walls and free-standing retaining walls due to earthquake motions. Commercial and private swimming pool walls also need seismic design recommendations from the Project Geotechnical Consultants. The Project Geotechnical Consultants should give consideration to both the CBC/LA County code and current up-to-date references, should provide specific recommendations for lateral earth pressures during seismic events, and should provide an explanation for the basis of their recommendations.

### *7.5.2 Cribwalls / Reinforced Earth Walls*

In addition to the aforementioned guidelines for standard retaining walls, the following items are applicable for cribwalls and reinforced earth walls.

- Adequate stability analyses are to be performed to show that both the internal and external stability of the wall is maintained.
- All pertinent manufacturer's specifications and recommendations are to be included in the report.
- All walls are to provide appropriate back-drainage for the entire height of the wall.
- Walls are to be backfilled with free-draining clean sand or gravel, including backfill within the cells of the cribwalls, unless it is demonstrated that alternatives perform acceptably.
- Structures are not to derive any support from nonstandard retaining walls unless it can be demonstrated that the vertical and lateral movements are tolerable.
- The zone or area behind walls containing reinforcement is to be clearly shown on as-built plans, and the area is to be marked with warning tape to reduce the likelihood of the reinforcement being compromised by any future excavation.

### *7.5.3 Other Nonstandard Retaining Walls*

A sufficient number of case histories are needed to substantiate the performance of any nonstandard retaining walls proposed under similar loading conditions.

### *7.5.4 Surcharge Behind Retaining Walls*

The Project Geotechnical Consultants are responsible for evaluating the potential for vertical and lateral surcharges on retaining walls due to adjacent structures, footings, traffic load, etc. A surcharge source, such as a wall footing of an adjacent structure, located below a 1:1 (horizontal:vertical) plane may result in a sufficiently large lateral force on a retaining wall that needs to be accounted for in the design of the wall. Using the 1:1 (horizontal:vertical) criterion to preclude the potential for lateral surcharge of retaining walls is not acceptable unless substantiated by appropriate analyses (e.g., methods of analysis presented in NAVFAC DM7.2). Acceptable methods to compute the lateral surcharge force or pressures include methods of elasticity, and not limit equilibrium methods unless joints, adverse bedding, or fractures exist within the soil/bedrock mass being retained and would result in larger lateral surcharge loading.

## 7.6 Shoring and Temporary Excavations

Shoring systems are usually temporary supporting structures used to retain earth until the structure is completed. It is the responsibility of the Project Geotechnical Consultants to provide shoring design parameters for determining the load(s) that the retained soil is likely to impose on the shoring units. The Project Geotechnical Consultants should evaluate the construction stability (i.e., temporary stability) during grading, foundation construction, and retaining wall excavations. All shoring should be designed in accordance with the following criteria at a minimum:

- A stability analysis model, considering and incorporating all applicable geologic discontinuities such as joints, shears, fractures, bedding planes, and faults.

- Shear strengths representing worst-case conditions anticipated at the time of excavation. Soil peak shear strength parameters may be used to compute the shoring loads.
- Modeling of tension cracks and anticipated external loading, as appropriate.
- Analysis of construction stability on all potential critical cross-sections. The critical failure surface on all cross-sections is to be identified, evaluated, and considered in the design of the shoring system. All potential failure modes of anchored walls are to be discussed and evaluated (Abating, Pass, and Bacchus, 1999).
- Analysis of construction stability using worst-case groundwater levels anticipated at the time of excavation.
- If the factor of safety for temporary excavations is less than 1.25: recommendations for remediation/mitigation measures to bring the safety factor up to 1.25.
- If shoring is recommended: a geotechnical design including, but not limited to, active, passive, and at-rest pressure magnitudes and lateral pressure distributions, the type of shoring, the location and magnitude of any external loads that may affect the design and/or performance of the shoring systems, and minimum embedment for the restraint system.
- If a slot-cut type system is proposed: analysis demonstrating the stability of excavated slots. The resistance on the sides of the wedge needs to be taken in a direction parallel to the critical failure plane (not in a horizontal direction). Slot cutting should be the A/B/C method and not the A/B method. The factor of safety for slot-cut calculations decreases and then increases with increasing excavation height. Therefore, the calculations are to demonstrate that a shallower excavation does not have a lower safety factor than the plan excavation height.
- Discussion of applicable requirements of CAL-OSHA that are incorporated into the excavation stability assessment. All trench shoring is subject to the provisions of the California Labor Code/State Construction Safety Orders. These regulations can be obtained from CAL-OSHA.
- When tiebacks are proposed: recommendations for performance and proof testing.

The Project Geotechnical Consultants are responsible for addressing whether any construction dewatering is necessary with respect to the proposed excavations and for evaluating the effects of the dewatering on adjacent existing structures/properties.

The Project Geotechnical Consultants need to address the amount of anticipated deformation during construction and its effect on existing adjacent structures. The need for a preconstruction survey to document existing conditions, including those of adjacent structures, and for deformation monitoring during construction should also be addressed (if applicable).

If an excavation affects the stability of existing structures and/or off-site property, shoring needs to be designed and installed to eliminate the hazardous condition. The design is to be in accordance with all standards in this guideline and is to consider all factors such as slope stability, settlement, creep, etc. The soil strength parameters are to be in accordance with the applicable criteria and are not to exceed the test values noted in the geotechnical engineering report.

## 7.7 Grading Recommendations

The report should contain sufficient and appropriate grading recommendations for the proposed grading in accordance with the City's Local Coastal Program and the Malibu Building Code. Grading recommendations should specify the depth and extent of the materials underlying the proposed foundations. If removal and recompaction is recommended, minimum removal depths referenced to the bottom elevation of the proposed foundations need to be specified and be consistent with the settlement estimates.

It is the responsibility of the Project Geotechnical Consultants to provide in the report recommendations (i.e., specifications) for compacted fill and to address:

- Minimum relative compaction.
- Moisture conditioning.
- Maximum particle size limits.
- Lift thickness.
- Mixing.

Compacted fill should be moisture conditioned to at or above optimum moisture content. The minimum relative compaction requirement for structural fills, including slopes, is 90 percent of the laboratory maximum dry density as determined by ASTM D1557.

### *7.7.1 Removal and Recompaction*

It is the responsibility of the Project Geotechnical Consultants to provide grading recommendations that incorporate recommendations regarding clearing and grubbing, removal of old fill, removal of debris, removal of abandoned tanks and wells, and removal of on-site wastewater treatment systems. The Project Geotechnical Consultants are also responsible for providing recommendations as to the minimum depth and the extent of materials that are to be removed and recompacted for support of proposed foundations and slab-on-grade construction. It is expected that the report also specify the minimum distance for removal and recompaction beyond the outside edge of shallow foundations, as determined by the project geotechnical engineer, and that the report provide recommendations for a foundation system that mitigates or reduces the effects of excessive settlement or heave (e.g., to a level in which service-related problems such as nonfunctioning doors and windows or excessively sloping slabs do not occur). Minimum removal depths (referenced to the bottom elevation of the proposed foundations) are to be specified and to be consistent with the settlement estimates.

### *7.7.2 Subdrains*

The Project Geotechnical Consultants should include in the geotechnical engineering report the locations of and design specifications for all subdrains and back drains. The report should also include, but not be limited to, outlet location, size, gravel pack, flow gradient, filter fabric, proposed cut-off walls, glued joints, vertical and horizontal drains, and design specifications. Subdrains should be installed beneath all pools, and under all other water amenities (e.g., spas, fountains, and ponds) when located in critical areas sensitive to subsurface water.

#### *7.7.3 Cut / Fill Transition Areas*

Foundations and utilities located in cut/fill transition areas and over variable thicknesses of fill may be subject to differential movements due to different stiffness characteristics and different hydrocollapse potential of the different supporting materials. Structures on cut/fill lots have not performed well during seismic events. Consideration should be given to potential differential foundation movements in such cases, and recommendations should be provided to mitigate the risk of differential movements. It is important that building pads located in cut/fill transition areas be over-excavated to provide a relatively uniform thickness of fill below the bottom of the proposed footings. At a minimum, fill thickness beneath foundations in cut/fill lots should be at least 3 feet, unless an alternative recommendation is justified on a site-specific basis. The Project Geotechnical Consultants need to provide recommendations for structural mitigation in the form of extra structural reinforcement of slabs and footings, as necessary.

#### *7.7.4 Organic Content in Fills and Backfills*

All certified fills and backfills are subject to the provisions of the current edition of the Malibu Building Code. Whenever the organic content percentage as calculated in accordance with ASTM D2974 Method C or D exceeds 2 percent, the material is considered detrimental and is not acceptable. Treated wood lagging is NOT considered organic content.

#### *7.7.5 Existing Fills*

It is the responsibility of the Project Geotechnical Consultants to ensure that grading plans show all existing fills on a site, classify these fills as certified or uncertified, and identify all buttress fills. For any grading involving cutting into an existing fill slope, the Project Geotechnical Consultants need to characterize the fill slope and to provide slope stability analyses as to the proposed and as-built conditions.

#### *7.7.6 Fill Slopes*

The Project Geotechnical Consultants are responsible for recommending keyways, benching, and drainage details that conform to the City's grading codes.

### 7.8 Drainage

The geotechnical engineering report should specify the need for and reasons why drainage and maintenance practices are needed for satisfactory performance of foundations and slabs. Proper drainage and irrigation are important to reduce the potential for damaging ground/foundation movements due to hydrocollapse and soil expansion or shrinkage. Proper drainage and irrigation are also important to mitigate adverse effects due to erosion that may endanger the integrity of the graded site, foundations, or flatwork. It is important that all surface runoff be carefully controlled and remain a crucial element of site maintenance.

The geotechnical engineering report should discuss and incorporate, as appropriate, recommendations for:

1. Minimum slope gradients and distance for drainage away from foundations,
2. Installing roof drains, areas drains, catch basins, and connecting lines,
3. Drainage beneath raised floors,
4. Managing landscape watering and maintenance of drainage devices,
5. Inclusion of waterproofing or damp-proofing systems for walls and floors when dealing with subterranean space or when landscaping mounds are constructed against buildings, and
6. Maintenance guidelines for property owners.

Planter boxes located adjacent to foundations should have water-tight bases and be connected to an acceptable drainage system. Subdrains should be installed below ponds and fountains.

### 7.9 Construction Observation and Testing

The report should contain sufficient and appropriate construction observation and testing recommendations for the proposed construction in accordance with the City's grading codes. At a minimum, the report needs to address the following:

- All fill placement and compaction are to be under observation and testing by the Project Geotechnical Consultants.
- The project engineering geologist is to observe all excavations in bedrock formational materials.
- Grain-size analyses and compaction curves are to be provided for all compaction curve samples.
- One duplicate sand cone test is to be performed for every four nuclear-gauge tests.
- The project geotechnical engineer is to observe the foundation excavations during construction and to verify the design assumptions.
- Geotechnical observation is to be performed for the installation of drilled deep-pile foundations, including verification of pile tip depth and clean-out of pile drill-holes.
- When driven piles are used, the Project Geotechnical Consultants are to confirm that field driving records are consistent with the engineer's design assumptions.
- The Project Geotechnical Consultants are to provide recommendations when shoring or underpinning is proposed adjacent to public right of ways or private existing developments. Such recommendations should include provisions to monitor ground deformation so as to adequately protect and inspect the conditions of infrastructure, buildings, streets, and walkways.
- When tiebacks are used, the tieback contractor is to perform an adequate number of proof tests and performance tests to confirm that anticipated tieback performance is being satisfied. It is the responsibility of the Project Geotechnical Consultants to observe the proof and performance testing, to document the results, and to submit the observations to the City for review.

# Appendix A – References

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# Appendix B - Shear Strength

*Direct Shear*

Shear test samples are commonly soaked prior to testing. The degrees of saturation of these test specimens are typically on the order of 80 to 90 percent. Therefore, specimens should not routinely be described as being saturated, unless the consultant provides substantiating data that supports that description. Direct shear tests on partially saturated samples may grossly overestimate the cohesion that can be mobilized when the material becomes saturated in the field. This potential overestimation of the cohesion needs to be considered when selecting shear strength parameters. The as-tested moisture content should be reported for all strength testing.

Direct shear tests need to be performed in accordance with ASTM procedures. When the Project Geotechnical Consultants use a rate of shear displacement exceeding 0.005 inch per minute, the consultants are responsible for providing adequate data to demonstrate that the rate is sufficiently slow for drained conditions (e.g., the time to failure exceeds 50 times the time for 50 percent consolidation). Such data may also be necessary when testing fine-grained soils, regardless of the rate of shear displacement used in the test. The rate of 0.005 inch per minute is not and should not be taken as a code or City requirement for performing direct shear tests. This rate is only a threshold rate that is used in the review process to determine in most, but not all, cases when the Project Geotechnical Consultants should demonstrate that the rate of deformation is sufficiently slow for drained conditions.

ASTM standards for direct shear tests limit the particle size to 10 percent of the diameter of the direct shear test specimen. When descriptions of samples or results of grain-size analyses indicate that particle sizes exceed 10 percent of the diameter of the direct shear box, the measured shear test results may be impacted by larger particle sizes. The Project Geotechnical Consultants either need to provide results of grain-size analyses or visual descriptions of the tested samples split along the failure surface to demonstrate that the particle sizes meet ASTM requirements, or need to discuss the impact that oversized particles may have had on the test results. The Project Geotechnical Consultants need to address this issue and provide an appropriate discussion of the selection of shear strength parameters for the project.

Studies demonstrate significant variability of soil strength within a site for a given material classification— whether the material is natural or compacted fill. Therefore, the number of shear tests needs to be appropriate to evaluate the variability of the strength for a given material and between material types encountered for the project.

*Shear Strength Selection for Slope Stability Evaluation*

Shear strengths occasionally used in stability analyses can be categorized as peak, ultimate, fully softened, and residual. The peak strength represents the maximum shear resistance on the stress-deformation (or strain) curve. The residual shear resistance is the shear strength at large displacements or strains when the material particles are aligned in the direction of shearing. The ultimate shear resistance for some materials may be the same as the peak strength, but for other materials— generally those that experience dilative behavior— the ultimate strength is less than the peak strength but greater than the residual strength and is identified as the point on the stress-displacement (or strain) diagram where there is a reverse in its curvature or an inflection point (Skempton, 1985). The reverse in curvature is associated with zero volume change in a drained test or zero change in porewater pressure in an undrained test on a saturated sample with increasing displacement or strain. According to Sabatini, Bachus, Mayne, Schneider, and Zettler (2002), "*The fully softened strength is intermediate between the peak strength and the residual strength and there are no specific procedures to identify the fully softened strength. Conceptually, the fully softened strength is close in value to the peak strength of the same soil in a normally-consolidated condition.*" The ultimate strength and the fully softened strength, in general, are not the same. The fully softened shear strength is associated with a failure envelope having a zero or near-zero cohesion. Therefore, the use of the term "fully softened" shear strength should not be used unless it can be demonstrated that the shear strength represents that of a normally consolidated sample when the volume change is zero with increasing deformation.

The design shear strength values are to be justified with laboratory test data, geologic descriptions, and history, along with past performance history, if known, of similar materials. The justification should include the rationale as to why the selected strength parameters are appropriate for the site. Some of the items that need to be included or considered in the justifications are:

- Strengths used for design are to be no higher than the lowest computed strength using back calculation. Assumptions regarding presliding topography and groundwater conditions at failure need to be discussed and justified. If the calculated factor of safety for a landslide mass is above the value that existed at the time of failure, it should be shown what changes have taken place to result in the safety factor increase.
- The literature is full of data on presumptive strength parameters for different material types. Great care, however, should be exercised when attempting to justify a selection of strength parameters by referring to such data. For example, presumptive strength parameters given for siltstone are generally for hard siltstone and may not be appropriate for softer siltstones encountered in this region. If such presumptive parameters are used to supplement on-site data, Project Geotechnical Consultants need to provide a convincing argument as to why such parameters may be appropriate for the subject site before such parameters are accepted.
- Multiple shear tests should be performed for each project. The number of shear tests needs to be appropriate to evaluate the variability of the strength for a given material and between material types encountered for the project. Multiple shear strength tests should be provided for each material type. When limited strength data is obtained, appropriate conservatism needs to be used to select shear strength parameters for slope stability (see page 50 of the *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California, Organized through the American Society of Civil Engineers, Los Angeles Section (ASCE-LA)* – Blake, et al., 2002). Ample site-specific data is necessary. The Project Geotechnical Consultants need to provide a discussion to support their selection of shear strength parameters. The use of composite graphs of shear strength data of similar materials is useful when providing justification for the selected shear strength parameters for slope stability.
- Reliance is sometimes made on shear strength data in the Seismic Hazards Zone Reports. That strength data was obtained from a number of sources, with the quality of sampling and testing varying between the sources. Today's standards differ from those at the time when most of the data contained in the Seismic Zone Hazard Reports was obtained. Using data from those reports is not considered acceptable for a specific site.

- Shear strength values higher than those obtained through site-specific laboratory testing are not acceptable.
- The material near bluffs may be more highly weathered and may have lower shear strengths than material more remote from the bluff. Evaluations of the stability of bluffs are to be based on shear strength parameters near the bluff and not remote from the bluff. Descriptions of the materials and mapping of the bluff need to be provided. If available, shear tests performed on samples taken adjacent to or on the bluff face from adjacent sites should be reviewed and discussed in addition to the mapping. The Project Geotechnical Consultants are responsible for including within the report a clear discussion and evaluation of the selected shear strength parameters, based on appropriate data.
- Direct shear tests do not always provide realistic strength values. Watry and Lade (2000) demonstrate the magnitude of scatter that can result when using the direct shear device to measure residual shear strength. Furthermore, bedding planes may not be parallel to the direction of shear in a direct shear device. The residual shear strength measured in the laboratory parallel to bedding is not necessarily the same as measured perpendicular to bedding (Mesri and Shahien, 2003). Correlations between liquid limit, percent clay fraction, and strength (fully softened and residual) by Stark, Choi, and McCone (2005) (recently updated in Stark and Hussain, 2013) are often used during the review process to evaluate strength parameters used by Project Geotechnical Consultants. Consultants need to provide justification if the strength values used in analyses exceed those obtained by these correlations. Therefore, results of grain-size analyses and Atterberg limits should be submitted for samples with shear-strength test results. If the percent clay and liquid limit are determined on samples that are air-dried or on samples that are well indurated, then the ball-milling corrections need to be applied to the measured percent clay and liquid limit.
- The ASCE/SCEC guidelines (Blake, et al., 2002) state, "*It is the judgment of the Committee that, based on the current state of knowledge, the residual strength friction angle from a drained test conducted at 'normal' strain rates can be used as a first-order approximation of the residual strength friction angle under undrained and rapid loading conditions.*" For seismic loading conditions, the City generally accepts the use of residual strengths for estimating the shear resistance on previously sheared surfaces, such as slickensides and landslide failure surfaces, under seismic conditions.
- Shear strengths can and do vary within a site. For example, at one site in Malibu shear strengths of a siltstone measured on samples in two borings about 190 feet apart varied by about 40 percent. This is only one example out of many that can be found by examining geotechnical engineering reports on file in Malibu. To account for such potential variability, the number of shear tests needs to be appropriate to evaluate the variability of the strength for a given material and between material types encountered for the project.
- The shear strength along bedding is typically less than the strength across bedding. To obtain the shear strength along bedding in a direct shear box, the technician needs to align the bedding plane in the direction of shear, and needs to locate a weak bedding plane in the zone of shear. This is not an easy task, and thus it is difficult in many cases to obtain good quality measurements of strength along bedding. Because residual strength is used to model along-bedding failures, orienting samples close to the bedding orientation and repeatedly shearing until a residual strength is achieved is a common procedure. The Project Geotechnical Consultants needs to address this issue in their discussion of site shear strength characterization.
- The criteria for strength selection, as described in the ASCE-LA guideline, rely in some cases on the plasticity as defined by the liquid limit. Although the ASCE-LA guideline allows visual

classifications to distinguish between levels of plasticity, laboratory test data is necessary for all submittals.

- Shear strengths for proposed fill slopes should be evaluated using samples mixed and remolded to represent anticipated field conditions. Strength testing may need to be confirmed during grading.
- Design shear strengths for fill slopes should be consistent with anticipated long-term movements and obtained from samples that have been soaked in an effort to reach saturated conditions.
- If direct shear or triaxial shear testing is used to model the strength of jointed and fractured bedrock masses, the design strengths are to be cross-checked with shear strengths obtained from the overall bedrock mass quality and are to be consistent with rock mechanics practice. When a material contains fractured bedrock with tectonic shears, either continuous or discontinuous, or discontinuities such as slickensides and fissures, the in situ strength depends on the frequency and orientation of the discontinuities. The Project Geotechnical Consultants should provide sufficient data to characterize joint patterns likely to be present below the subject site, and at least a qualitative analysis of the following:
  - Any well-defined joint sets that exist and that could either individually, collectively, or through their intersections act as planes of weakness along which translational, quasi-rotational, or wedge failures could occur. Finite element slope stability analysis programs such as Phase2 (Rocscience, 2013) are useful for evaluating the effects of joint discontinuities.
  - How these joint sets either individually or through interaction with each other impact developments proposed in the shallow subsurface.

  These basic geometric considerations need to be defined before an appropriate mode of failure can be defined for analysis and are best defined by a detailed discussion of observed joint patterns based on examination of many joints and joint sets both on- and off-site.

  The choice of appropriate shear strengths should consider the continuity of joints, the morphology of joints and asperities (e.g., planar, irregular, smooth, rough), and the presence and nature of any joint linings. Direct shear tests need to be completed that represent the materials likely to be present along potential failure surfaces. For example, it may be most appropriate to complete direct shear tests on samples that have been pre-cut to represent the strength along fractures or joints.

  In short, the Project Geotechnical Consultants need to provide a detailed evaluation of the measured shear strengths and discuss how the impact of fractures and joints was taken into account when selecting the shear strength parameters for use in slope stability analyses, in assessing the mode of potential failure, and in making design recommendations.

<u>*Impact of Defects on Shear Strength Selection*</u>

The soil properties measured in the laboratory (e.g., failure envelope, as defined by cohesion, c’, and friction angle, $\phi$’, or the modulus, E) may differ from the corresponding in situ or operational properties. Factors contributing to inaccuracies or shortcomings of laboratory measurements include:

- Disturbance due to stress release during sampling,
- Mechanical disturbances during sampling, handling, storing, and specimen preparation,
- Soil anisotropy,
- Relative magnitude of intermediate principal stress,
- Rotation of principal planes during shear,
- Rate of shearing,

- Cyclic loading,
- Strain softening,
- Initial in situ stresses,
- Specimen size,
- Calibration errors, and
- Limited testing of a statistically heterogeneous material.

All these factors need to be considered when selecting shear strength parameters to characterize the site. The influence of these factors on the uncertainty in the material properties has been discussed in a number of papers (Lumb, 1966; Singh and Lee, 1970; Wu and Kraft, 1970; Schultze, 1971; Ray and Krizek, 1971; Fredlund and Dahlman, 1971; Yucemen, 1973; Kraft and Murff, 1975). One item that often is not fully addressed by Project Geotechnical Consultants but is of significant importance is the influence of defects, such as fissures, slickensides, and joints, on the shear strength that can be mobilized in the field compared to what is measured on small-diameter test specimens.

When the size of a specimen with a system of fissures randomly distributed is increased, the number of fissures included in the sample increases, the probability of having fissures critically orientated to the applied stress system increases, the probability of having larger fissures increases, the probability of having large fissures critically oriented increases, and the probability of coalescing adjacent defects in the proximity of the potential failure plane increases. All these factors tend to decrease the applied stress necessary to rupture the specimen. The apparent strength therefore decreases as the size of the sample increases and, in the limit, approaches the strength of the soil mass. It is difficult to conceive that failure in the soil mass can take place along a single continuous plane of weakness of considerable extent, although a composite failure path along multiple discontinuities is likely. Consequently, the operational strength is likely to be higher than the individual-fissure strength of the material. However, exceptions are cases where failure is along a pre-existing slide surface in which the strength is reduced to, or close to, the residual strength of the material, where failure is along predominant bedding planes or weak layers that are sandwiched between stronger strata, or where progressive failure occurs in which strengths of portions of the potential sliding surface are successively reduced from the peak value to the residual state.

Specimens typically used for direct shear testing are only 1 inch high and about 2.4 inches in diameter. Also, the specimens are confined in a relatively rigid ring when undergoing a shear test. The likelihood of a defect such as a fissure, regardless of its tightness or continuity, negatively impacting the measured strength is very small compared to the behavior in the field where there is less displacement constraint as would be provided by the ring in a laboratory test. Unconfined compressive strength on samples of siltstone and claystone larger than direct shear tests (from a Malibu site tested by Bing Yen and Associates) ranged between 800 to 4130 psf. Similarly large variations in unconfined compressive strengths were also found by Bing Yen and Associates in sandstone (from 570 to 2980 psf). Some of that variation in unconfined compressive strength may be due to variations in moisture content, density, and grain-size composition, but it is likely that defects such as shears, fissures, and slickensides played a major role in the variation. The number or length of defects along a potential failure surface in the field, as a percentage, likely exceeds that in a small laboratory specimen, such as that of a direct shear specimen. Sampling of materials may result in the development of fractures, but that does not necessarily mean that such are present in the small specimens tested and therefore their impact on in situ strengths may not have been taken into account in a laboratory direct shear test. The fractures, fissures, and slickensides do not have to coincide with the orientation of the potential failure plane in situ to negatively impact the strength that can be mobilized in situ, although the orientation quantitatively affects the impact.

Also, the number or length of defects along a potential failure surface in the field, as a percentage, likely exceeds that in a small laboratory specimen. The shear strength measured in the laboratory on small and potentially intact samples may not be reflective of the strength that is mobilized in situ for materials that are fractured, even if not continuous. When fissures, fractures, and joints of the bedrock are of sufficient size and abundance for migration of precipitation, irrigation, and effluent into the bedrock, they certainly have an impact on the shear strength that can be mobilized. One would expect that a fracture system that provides for adequate effluent disposal would not have a negative impact on the shear strength. Although guidelines provide for judgments in the selection of shear strength, all judgments need to be supported with appropriate references and not be contrary to the project data.

Much of the research on shear strength of drained direct shear tests performed at university and national research laboratories, such as the Building Research Station or the Waterways Experiment Station, have been done at slower rates of deformation corresponding to test durations of one-half to more than one day compared to less than one hour in most commercial laboratories. This results in lower measured shear strengths as well as reducing some of the displacement constraints in the direct shear test.

The intact and fissure strengths reflect the upper and lower bound values, respectively, of the strength that can be measured on a sample. Fractures, joints, and other defects impact the strength of the bedrock that can be mobilized in the field and result in a strength that is below that of a truly massive material. Laboratory specimens may not contain fractures, joints, or other defects due to their small size. Studies have shown that when a material contains fractures, either continuous or discontinuous, or discontinuities such as slickensides and joints, the in situ strength depends on the frequency and orientation of the discontinuities. From the engineering point of view, however, the strength in the soil or bedrock mass in the field is the critical issue. Lo (1970) refers to this strength as the operational strength. Most of the work on the influence of defects on the measured shear strength as it varies with test specimen size has been done for undrained conditions, as the test times get very large for large-scale direct shear tests and drained conditions. Nevertheless, information gained for undrained conditions is useful for gaining insight into the effects for drained conditions. The operational strength is influenced by the location, orientation, spacing, frequency, or density of defects (Ward, Marsland, and Samuels, 1965; Lo, 1970; Wu, Williams, Lynch, and Kulatilake, 1987; Jade and Sitharam, 2003).

Regardless of whether the material is hard rock or softer soil, the studies show that the operational shear strength decreases as the number of defects increases. Wu, Williams, Lynch, and Kulatilake (1987) and Jade and Sitharam (2003) provide a means to quantify such defects. The ratio of the shear strength of an intact sample to the shear strength of a larger sample decreases with sample size in an exponential manner. Lo (1970) expresses the relationship in terms of the size of the failure surface, while Jade and Sitharam (2003) use a term called joint factor, which includes the density of the joints or defects, the inclination of the defect, and the strength of the defect.

Hoek and Brown (1988) and Hoek, Carranza-Torres, and Corkum (2002) also provide a quantitative method to evaluate the operational strength. If the Hoek-Brown model is used to determine shear strength or to provide a comparison with laboratory measured strength on small samples, it should be recognized that one of the critical parameters is the compressive strength of the intact material. Thus, if the Hoek-Brown model is used, multiple unconfined compressive tests need to be obtained to provide a basis for the appropriate compressive strength. Sometimes Project Geotechnical Consultants estimate a compressive strength that approaches or exceeds that of concrete in materials that were drilled without the need of coring. A bucket auger is not likely to be able to excavate a material with a compressive strength of concrete. Thus, a much lower compressive strength than that for concrete would be appropriate for most materials if the Hoek-Brown criterion is used to represent the site. Even though some contend that the Hoek-Brown model is for hard rock, it can be an appropriate model to estimate the operational strength of softer fissured materials, if the input parameters are properly selected. In the case of blue London clay for which adequate data are available, reasonable estimates of the operational strength parameters, representative of the curved failure envelope over a normal stress of 0.5 to 2.5 ksf, are obtained using the average measured intact compressive strength. There are three other parameters in addition to the unconfined compressive strength of the intact material that impact the results of the Hoek-Brown estimates of the operational strength. The selection of these three parameters, however, is somewhat subjective and open to debate. Thus, the confidence level of using the Hoek-Brown model may be less than desirable when used for softer materials.

Wu, Williams, Lynch, and Kulatilake (1987) found that measured peak and softened strengths, in terms of c' (effective cohesion) and $\phi$' (effective friction angle), for 25-cm samples fell within one standard deviation of the strengths on 5-cm samples. The relatively small difference in this case may be due to the close spacing of the defects. Marsland and Butler (1967) compared drained cohesions and friction angles measured on triaxial samples with a length-to-diameter ratio of 2 and diameters varying from 1.5 inches, to 3 inches, to 5 inches. The friction angles for all specimens were very close (a difference of 0.5 degree less for the larger samples), but the cohesion of the two larger samples was 65 percent of that for the smaller sample. A drained test on a 2-foot-by-2-foot square shear box provided a cohesion that was slightly larger than measured on the two larger triaxial specimens and a friction angle that was a few degrees higher. A direct shear test may provide different strength parameters than a triaxial test, even of the same size, due to differences in the principal stress direction between the two test types as well as soil anisotropy and other factors, so it makes the comparison of the influence of test size a little less meaningful when results from different types of tests are compared in addition to the effect caused by heterogeneity differences between two samples. Nevertheless, most data show that, for either undrained or drained tests, shear strength decreases with an increase in specimen size for materials that contain defects. The rate of decrease, however, is affected by the orientation of the defects as well as the density or spacing and size of the defects. Variability in material or between samples of like material makes any comparison based on very few tests of limited value.

Bishop (1967) found for a 1.5-inch-diameter triaxial specimen that the drained cohesion was 45 percent of the intact cohesion and the drained friction angle was 80 percent of that for an intact sample. Using Lo's (1970) model and the parameters for undrained tests would predict a strength for the 1.5-inch-diameter sample of 63 percent of the intact strength. If the drained intact cohesion and intact friction angle are used with the strength parameters along the fissure to obtain the ratio of the operational strength to the intact strength (keeping the other parameters as determined from the undrained tests), the computed strength for the 1.5-inch-diameter specimen is 58 percent (compared to the measured 45 percent) for the cohesion and 84 percent (compared to the measured 80 percent) for the friction angle. This is only one comparison, but it suggests that, for drained tests, the drained cohesion is reduced more than the drained friction angle, and the drained cohesion may be impacted more by defects than that for undrained strength. The question remains whether the constraints of the direct shear tests on small samples prevent the full impact of defects from being measured.

Wu, Williams, Lynch, and Kulatilake (1987) used Skempton's residual factor (R) to determine the operational strength to cause failure. An R of 1 corresponds to failure occurring at the residual strength, and an R of less than 1 corresponds to failure occurring at an operational strength between residual and peak strength. Slopes tended to fail with R values between 0.8 and 0.9 when the defect intensity was high and a local zone of stress concentration was present. For the materials they studied, the maximum effect of defects was to reduce the operational shear strength to an R of 0.8. Thus, failure can occur before residual strength is reached. When failure occurs at an R value of less than 1, the computed safety factor using the residual strength would be less than 1. It is sometimes concluded by some Project Geotechnical Consultants that a computed safety factor of less than 1 with residual strength for a slope that is currently stable implies that residual strength is higher than what is being used. This may not be the case. Failure can occur before the residual strength occurs. With additional slope movement, the residual strength may be reached, but geometry changes result in a safety factor stabilizing at 1.

Skempton (1977) found for first-time slides in London clay that the operational effective strength envelope lies between the lower bound envelope for the strength on fissures (which is generally above the residual envelope) and the post-rupture strength for initially intact samples, which may be close to the ultimate strength. Stark and Eid (1997) found in a study of 14 first-time slides through stiff fissured clay with a liquid limit between 50 and 130 percent that the mobilized shear strength along the failure surface in first-time slides through stiff fissured clay can be lower than the fully softened shear strength and can be as low as the average between the fully softened and residual shear strengths. Geologic factors such as fissure spacing and bedding existence, loading conditions (such as changes in groundwater), and zones of stress concentrations undoubtedly affect the operational strength. Although the fully softened and ultimate shear strength are not necessarily the same, the findings of Skempton (1977), Wu, Williams, Lynch, and Kulatilake (1987), and Stark and Eid (1997) are in general agreement as to the operational strength in comparison to other laboratory measures of shear strength.

The Project Geotechnical Consultants need to provide a discussion, supported with adequate data and reasonable interpretations, to justify their selection of shear strength parameters and to specifically address how the presence of defects was taken into account in their selection of shear strength parameters used to characterize the site. The references used above as well as the paper by Mesri and Shahien (2003) are useful in developing a discussion to support how the presence of defects was taken into account in the consultants' selection of shear strength parameters used in the analyses.

## Appendix C - Soil Corrosivity

The following comments are taken, with some minor editing, from the Los Angeles County Manual for Preparation of Geotechnical Reports and are provided for general information only.

Soil corrosivity is influenced by the sulfate and chloride content of the soil or water and by the pH and resistivity of the soil. Soil corrosivity is often beyond the expertise of Project Geotechnical Consultants. Professionals that specialize in this subject are available and should be consulted. Nevertheless, the Project Geotechnical Consultants can provide a preliminary evaluation of potential concerns by testing the soil for sulfate and chloride content and for pH and resistivity. In evaluating potential soil chemistry effects on building sites, the Project Geotechnical Consultants should use a qualified laboratory to conduct the testing and should provide a copy of the laboratory data sheet. The test should be evaluated in light of the test method and compared with industry thresholds for chemical attack or corrosion.

The Project Geotechnical Consultants can, based on their experience and judgment, evaluate potential chemical and electro-chemical hazards in lieu of, and/or in addition to, those cited here. The Project Geotechnical Consultants may elect to use one of the representative test methods cited in these guidelines or another method of the Project Geotechnical Consultants' choosing, or may recommend that an expert in soil corrosivity be retained to further evaluate site conditions.

If field resistivity tests are performed, the Project Geotechnical Consultant is responsible for reporting the following:

- Map showing array locations.
- Data sheet showing measured resistance.
- Table or figure showing computed results.

*Sulfides –Sulfates*

Sulfide minerals are usually encountered in unweathered bedrock. When exposed to air and moisture, sulfides undergo a chemical reaction to become sulfates, which can lead to other problems as described below. During this chemical reaction the sulfide minerals may expand as much as eight times. Often this reaction is described as being soil expansion. The standard expansive soil test, however, does not detect this potential chemical reaction. Presently, little is known about the chemical reaction rate. In some areas, the chemical reaction is very rapid occurring within a few days after exposure. In other areas, this reaction is very slow, affecting structures years after construction. Sulfide minerals have been encountered in the Castaic Formation in the Santa Monica Mountains, from Topanga to Encino.
Certain sulfate minerals present in the soil, rock mass, or groundwater have a detrimental effect on concrete. Most prominent of these are sulfates of sodium, magnesium, and calcium. These sulfates react chemically with the hydrated lime and calcium aluminate of the hardened cement paste to form calcium sulfo-aluminate.

Disintegration of the concrete is due to a combination of chemical and physical forces. The effect of such an attack is minor in dense impermeable concrete on relatively dry natural materials, but results in disintegration of high water-cement ratio, permeable concrete bearing on saturated highly mineralized fill or natural materials.

Based on CBC requirements, when soluble sulfate concentrations are greater than 1,000 ppm in soil and 150 ppm in groundwater, mitigation measures are to be taken to protect any concrete structures in contact with the soils. If soil is not to be removed, an appropriate cement type should be used. The geotechnical engineering report should consider tests for sulfide-sulfate minerals in the soil, rock mass, and/or groundwater. Recommendations in the geotechnical engineering report are to include mitigation measures such as: the removal of the sulfide and sulfate materials to a depth below the concrete that the sulfates do not influence the proposed structure, treatment to remove the sulfates, and/or design of foundations to resist the effect of the sulfates.

*Chlorides*

Large concentrations of chlorides adversely affect ferrous materials such as iron and steel. When chloride concentrations exceed 10,000 parts per million, mitigation measures need to be taken to protect any steel reinforcing within concrete and any steel pipe or cast iron that serve the development (LA County, 2010). Mitigation measures generally consist of cathodic protection, isolation such as using very dense cement mixes around vulnerable material, or plastic wrap to prevent moisture contact between the soil and the material under protection.

*pH*

Mitigation is to be recommended when test results indicate that the on-site soils are corrosive. Typically, acidity does not cause trouble until the pH gets down around 4.0 (LA County, 2010).

*Resistivity*

The most common factor in determining soil corrosivity is electrical resistivity. As a soil's resistivity decreases, its corrosivity increases. Mitigation is to be recommended when test results indicate the soil to be moderately corrosive or worse per the following table:

| **Soil Resistivity, Ohm-Cm** | **Corrosivity Category** |
|---|---|
| 0 – 1,000 | Severely Corrosive |
| 1,000 – 2000 | Corrosive |
| 2,000 – 10,000 | Moderately Corrosive |
| over 10,000 | Mildly Corrosive |

## Appendix D - Soil-Pile-Structure Interaction during Seismic Events

The soil-pile-structure interaction during a seismic event is a complex phenomenon, even when the supporting soil is not subject to liquefaction. For example, the shear and moment in a pile increase with increasing pile stiffness, and amplification of the structural response is accentuated if the exciting frequency and the resonant frequencies of the soil deposit and structure are close to each other. As the pile stiffness increases, more of the seismic load is transferred to the structure above, which in turn increases the dynamic load on the pile. Adding liquefaction to the mix further compounds the complexity of the interaction. Numerical models have been developed to gain insight into the interactions, and centrifugal testing in recent years has provided additional insights. Computed results with finite element analyses are generally in good agreement with results from the simpler beam-on-Winkler foundation method if the soil-pile springs and dashpots used to represent the soil-pile interaction effects are appropriately selected.

A search of the ASCE database from 1975 to 2009 using "soil-pile-structure interaction" and "seismic" as the search terms yielded 47 papers beginning in 1980 with publications by Kagawa and Kraft (1980 and 1981). Most of the work is related to waterfront structures (e.g., piers and wharves) and bridges. The Kagawa and Kraft work was related to offshore, pile-supported structures. Nevertheless, these publications have application to pile-supported residential structures on beach-front properties. All of these studies demonstrate that the dynamic characteristics of the seismic event, local soil conditions, the structural characteristics of the piles, and the structural characteristics of the structure can have a significant impact on the results. Unfortunately software programs that have been developed for the numerical models are not readily available for commercial application, but a number of papers are available to provide insight. In addition, the results of centrifugal testing have confirmed the complexity of the interaction and also provide insight that can be used in developing design criteria.

Liquefied soil and nonliquefied soil within and above a zone of liquefaction could impose lateral loads on the piles and pile cap. Loads from laterally spreading ground have been a major cause of damages to pile foundations in past earthquakes, particularly when a nonliquefied crust layer spreads laterally over underlying liquefied layers (Dobry and Abdoun, 2001). In the case of piles in soils that experience lateral spreading, Dobry and Abdoun (2001) state, "*Both very rigid and more deformable foundation superstructure systems may be exposed to large lateral soil pressures, including especially passive pressures from nonliquefied shallow soil layer riding on top of the liquefied soil,*" and, "*More damage tends to occur to piles when lateral movement is forced by a strong nonliquefied shallow soil layer than when the foundation is more free to move laterally and the forces acting on them are limited by the strength of the liquefied soil.*"

The magnitude of lateral load that liquefied soil and nonliquefied soil in and above a zone of liquefaction impose on the piles and pile cap depends, among other factors, on the strength of the nonliquefied zone riding on top of the liquefied zone and the magnitude of lateral spread or the lateral movement of the soil relative to the pile and pile cap (Dobry, Abdoun, and O'Rourke, 1996; Dobry and Abdoun, 2001; Dobry, Abdoun, O'Rourke, and Goh, 2003; Abdoun, Dobry, O'Rourke, and Goh, 2003; Boulanger, Kutter, Brandenberg, Singh, and Chang, 2003; Brandenberg, Boulanger, and Kutter, 2005). The lateral force can vary from near zero (if no lateral spread occurs) to that sufficient to correspond to the passive force in the zone above the liquefied zone plus the lateral force in the liquefied zone for large lateral displacements (Brandenberg, Boulanger, Kutter, and Chang, 2007).

The 2010 CBC requirements (considering only inertial loading) for computing the lateral load that gets transferred from the structure to the piles under seismic loading are considered acceptable at this point in time if the soils do not experience lateral spreading. When, however, the soils are subject to lateral spreading, additional lateral load (such as kinematic loading) is transferred to the piles and should be accounted for in the pile design to satisfy 2010 CBC requirements.

The Project Geotechnical Consultants need to provide analyses to support a lateral force that the piles and pile cap may be subject to if the site liquefies and experiences lateral spreading. The lateral loads need to be consistent with Dobry's findings and as well as those at the University of California at Davis (Boulanger, Kutter, Brandenberg, Singh, and Chang, 2003; Brandenberg, Boulanger, Kutter, and Chang, 2005, 2007, and 2007a). The results of these studies have been based on single piles as well as small groups of piles. Although the number of piles in these studies may differ from that supporting a structure on the Malibu coast, the findings lend themselves to develop reasonable levels of lateral loads that piles may be subjected to if lateral spreading occurs. To avoid the need for a costly mitigation solution, it is critical that the scope of the geotechnical study be sufficient to provide a realistic assessment of not only liquefaction potential, but also the potential for lateral spreading. Piles may inhibit the amount of lateral spread, and analyses should be provided to demonstrate that the recommended lateral stabilizing force provided by the piles is consistent with the amount of predicted lateral spreading if lateral spreading occurs.

The 2010 CBC relies on ASCE 7-05 for design procedures. In particular, Chapter 11 of ASCE 7-05 deals with seismic design. ASCE 7-05 is undergoing modifications to clarify design requirements. Arulmoli, Johnson, Yin, Jaradat, and Mays (2008) describe the types of analyses that are needed and the geotechnical input necessary for seismic design of structures. Single-family structures in Malibu are considered of Moderate Importance with a performance level of Controlled and Repairable Damage.

It should be recognized that simplified structural analyses may not provide cost-effective pile design when considering the lateral loads that need to be supported under lateral spreading. Analyzing the foundation system as a bent where some of the moment is carried by the axial forces in piles may be more costly, but the resulting economies in the design, as reflected by construction costs, may more than offset the analyses cost. Ground modification may be a more economical alternative than using piles to mitigate lateral spreading.