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December 2, 2016

VIA HAND DELIVERY AND EMAIL

Angela Calvillo, Clerk of the Board
San Francisco Board of Supervisors
1 Dr. Carlton B. Goodlett Place
City Hall, Room 244
San Francisco, CA 941 02
bos.legislation@sfgov.org

Re: Appeal of CEQA Categorical Exemption Determination
Planning Case No. 2013.1383ENV
Building Permit Application Nos. 2013.12.16.4318 & 2013.12.16.4322
3516-3526 Folsom Street ("Project Site")

Dear Ms. Calvillo:

Enclosed, please find the following:

1. Report from Rune Storesund, D.Eng., P.E., G.E. regarding public safety risk;
2. Report from retired SFFD Captain Mario Ballard regarding emergency vehicle access;
3. Letter from Robert Bea, Professor Emeritus, Center for Catastrophic Risk Management;
4. Letter from the Sierra Club, San Francisco;
5. Letter from the Bernal Heights Democratic Club;
6. Letter from the Bernal Heights Neighborhood Center;
7. Emails from Bradford Street neighbors regarding steep-street unusability;
8. Report from Patrick Buscovich, S.E.;
9. Documentation and information regarding gas-pipeline damage due to tree roots; and
10. Seismic guidelines and earthquake hazard maps.

Please kindly include these items with the appeal file.

Thank you.

Very truly yours,

ZACKS, FREEDMAN & PATTERSON, PC

Ryan J. Patterson
Encl.

Exhibit 1

Storesund Consulting
154 Lawson Road, Kensington, CA 94707
510-225-5389 (cell) email: rune@storesundconsulting.com

December 1, 2016

SF Board of Supervisors
San Francisco City Hall
1 Dr Carlton B Goodlett Pl #244
San Francisco, CA 94102

Subject: Independent Project Review
3516 & 3526 Folsom Street
San Francisco, California

Dear President Breed and Honorable Members of the Board of Supervisors,

This letter is in response to a request for an independent assessment of the proposed 3516 & 3526 Folsom Street development. My qualifications are presented in the attached resume. I am a practicing Geotechnical Engineer (CA License Number 2855), I provide gas pipeline risk reviews for the State of California Department of Education, and have participated in forensic engineering projects over the last 10 years with damage claims in excess of $2 billion and more than 8,000 hour of direct forensic analyses. My most recent engagement was a geotechnical forensic evaluation of the March 2014 Oso Landslide in Washington State, which resulted in the tragic loss of 43 individuals. In addition to private consulting, I am the Executive Director of the Center for Catastrophic Risk Management at UC Berkeley.

This geotechnical review is the requested independent assessment and is based on documents included in the Discretionary Review, Full Analysis by San Francisco Planning Department (dated October 4, 2016) as well as a set of geotechnical reports prepared by Mr. H. Allen Gruen (dated August 3, 2013).

The proposed projects are located immediately adjacent to a major PG&E transmission natural gas pipeline (Figure 1, Figure 2, Figure 3). This major pipeline is located immediately below the primary access road for the construction (Figure 4, Figure 5), immediately adjacent to significant proposed new utility work (e.g. gas service, water supply, sewer) as well as removal of existing pipeline soil cover (Figure 6, Figure 7), and immediately adjacent to significant proposed bedrock excavation (depths on the order of 6 to 10 feet per the submitted architectural elevations (such as sheet A-3), as seen in .

Construction-related stressing, as well as accidental 3rd party damage, has the potential to degrade the integrity of the PG&E natural gas transmission line, exposing the surrounding neighbors to increased risk of death and injury from the potential of construction-induced puncture or degradation of pipeline integrity.

Unlike lots further west and further east (Gates Street, Banks Street) that are not immediately adjacent to a transmission line, these specific parcels are unique in their proximity to a significant hazard.

Major items of concern include at this particular project site:

- Geotechnical borings do not extend to the proposed depth of excavation, providing information on competence of bedrock and anticipated level of effort to excavate;
- No explicit discussion about induced ground vibrations during rock excavation and associated potential degradation of the PG&E transmission line integrity;
- No explicit discussion about negative impacts of construction traffic to the PG&E transmission line integrity; and
- Significant construction operations immediately adjacent to the active PG&E transmission pipeline.

Given the uncertainties of actual pipe integrity, strong consideration should be given to replacing the segment of pipeline to ensure maximum integrity and minimal exposure of residents to undue injury or death as a result of the anticipated heavy excavation and ground disturbance activities.

Overview of parcel locations relative to transmission line.

Figure 1: Overview of parcels with proposed development. Note that the PG&E transmission line is directly under the primary access.

Site Photo

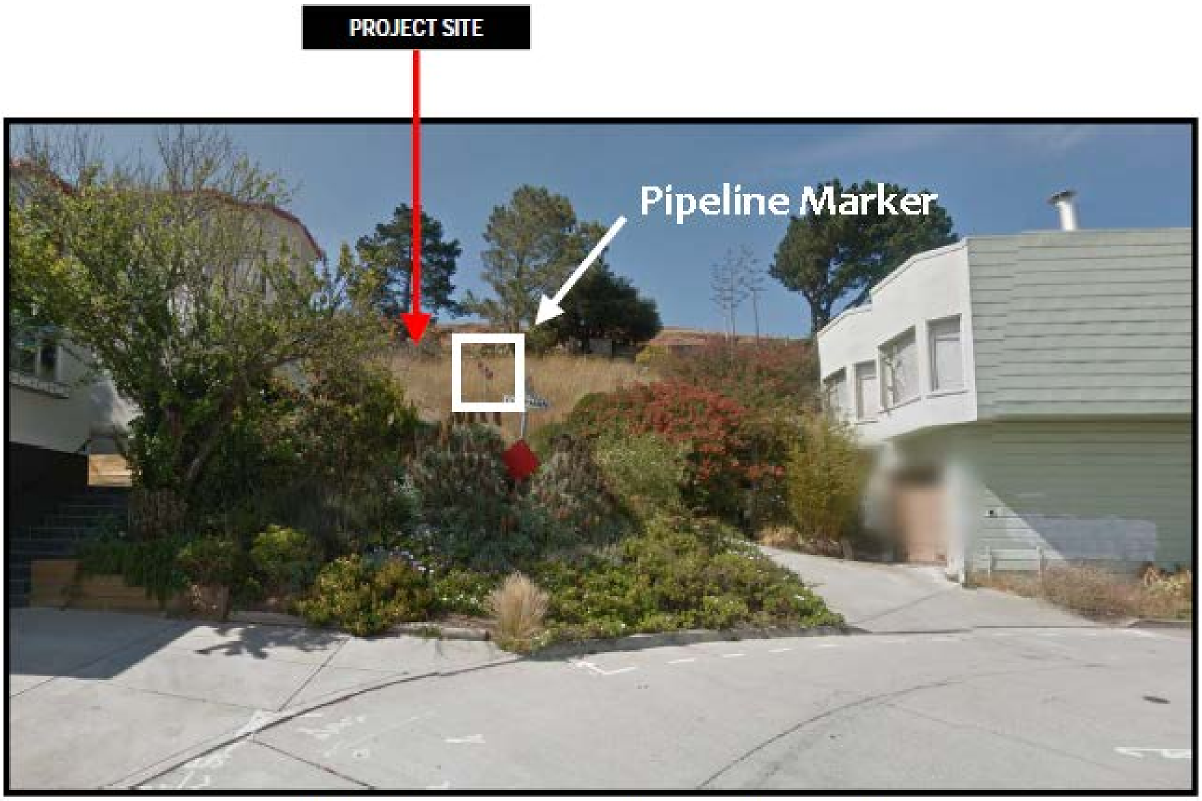
View from Bernal Heights Boulevard, near intersection with Folsom Street
(Source: Google Maps, July 2015; Accessed March 23, 2016)

SAN FRANCISCO
PLANNING DEPARTMENT

Discretionary Review Hearing
Case Numbers:
2013.1383DRP-10 & 2013.1768DRP-09
3516 & 3526 Folsom Street

Figure 2: Pipeline marker at Bernal Heights Boulevard.

Site Photo

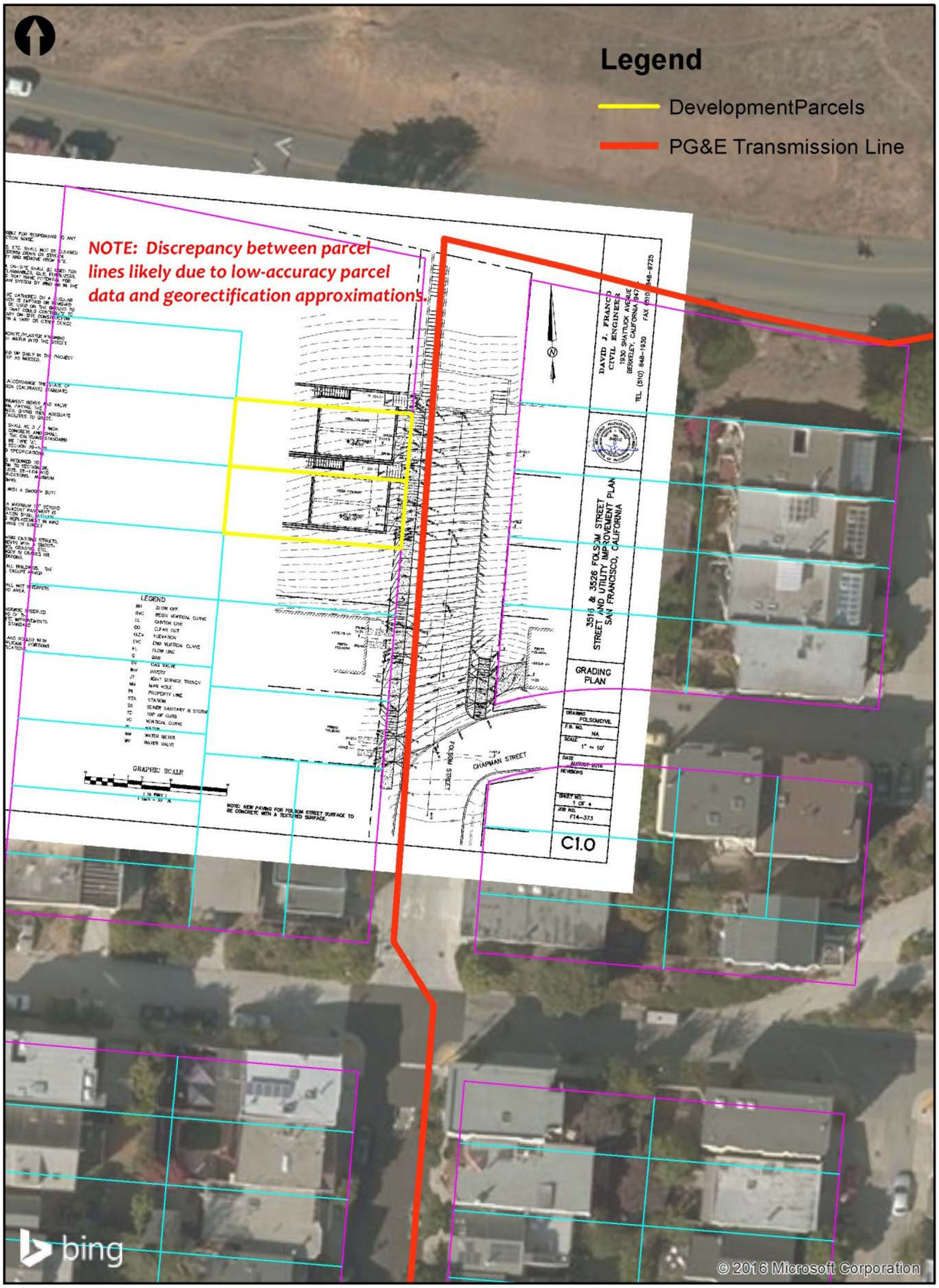


View of Folsom Street (looking up to Project Site)
(Source: Google Maps, July 2015; Accessed March 18, 2016)

SAN FRANCISCO
PLANNING DEPARTMENT

Discretionary Review Hearing
Case Numbers:
2013.1383DRP-10 & 2013.1768DRP-09
3516 & 3526 Folsom Street

Figure 3: Pipeline marker at corner of Folsom & Chapman.



Site plan relative to transmission line.

50 25 0 50 Feet

Figure 4: PG&E transmission line relative to proposed site plan.

CAMERA 5: View from Chapman Street at Folsom Street looking North-West

Figure 5: Approximate PG&E transmission gas line alignment relative to proposed structures.

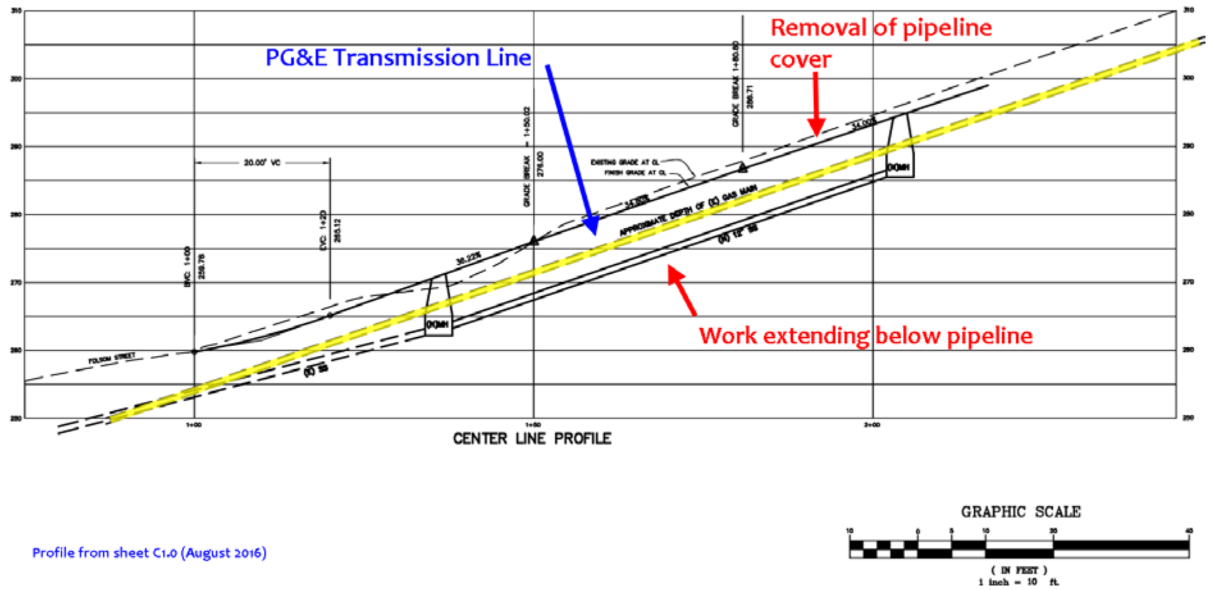


Figure 6: Plans call for removal of pipeline cover as well as construction work <u>below</u> the existing pipeline.

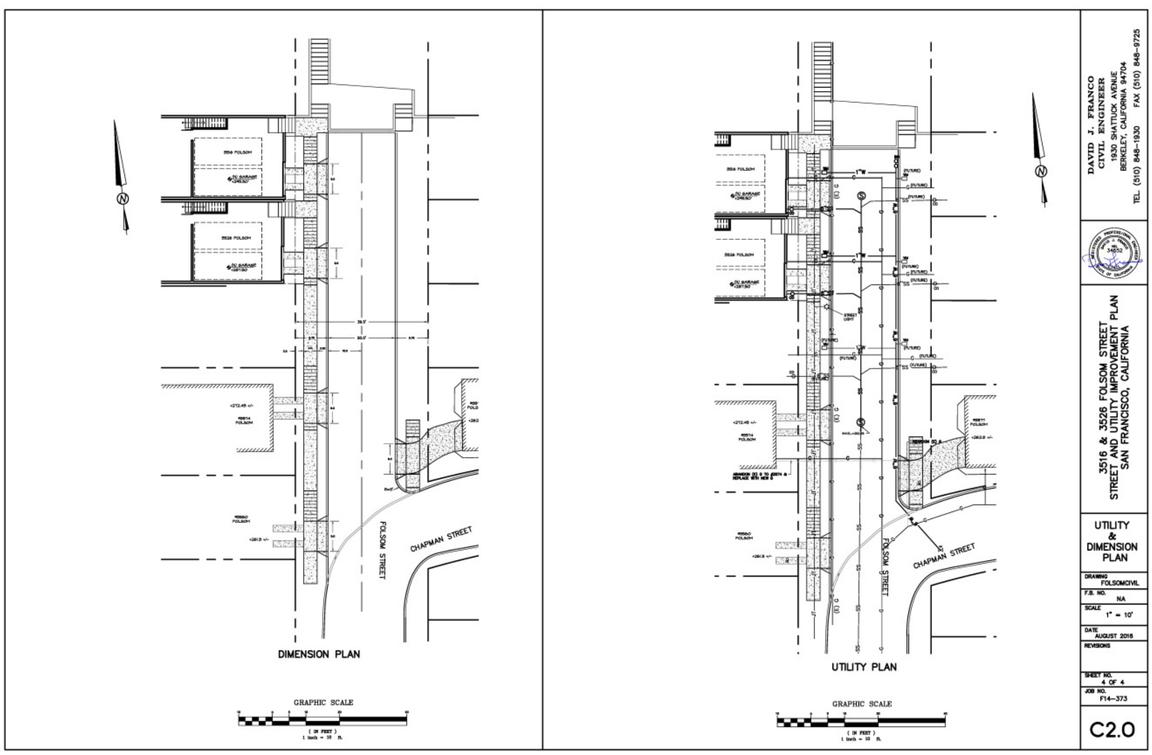


Figure 7: Proposed utilities <u>immediately adjacent</u> to the PG&E transmission line.

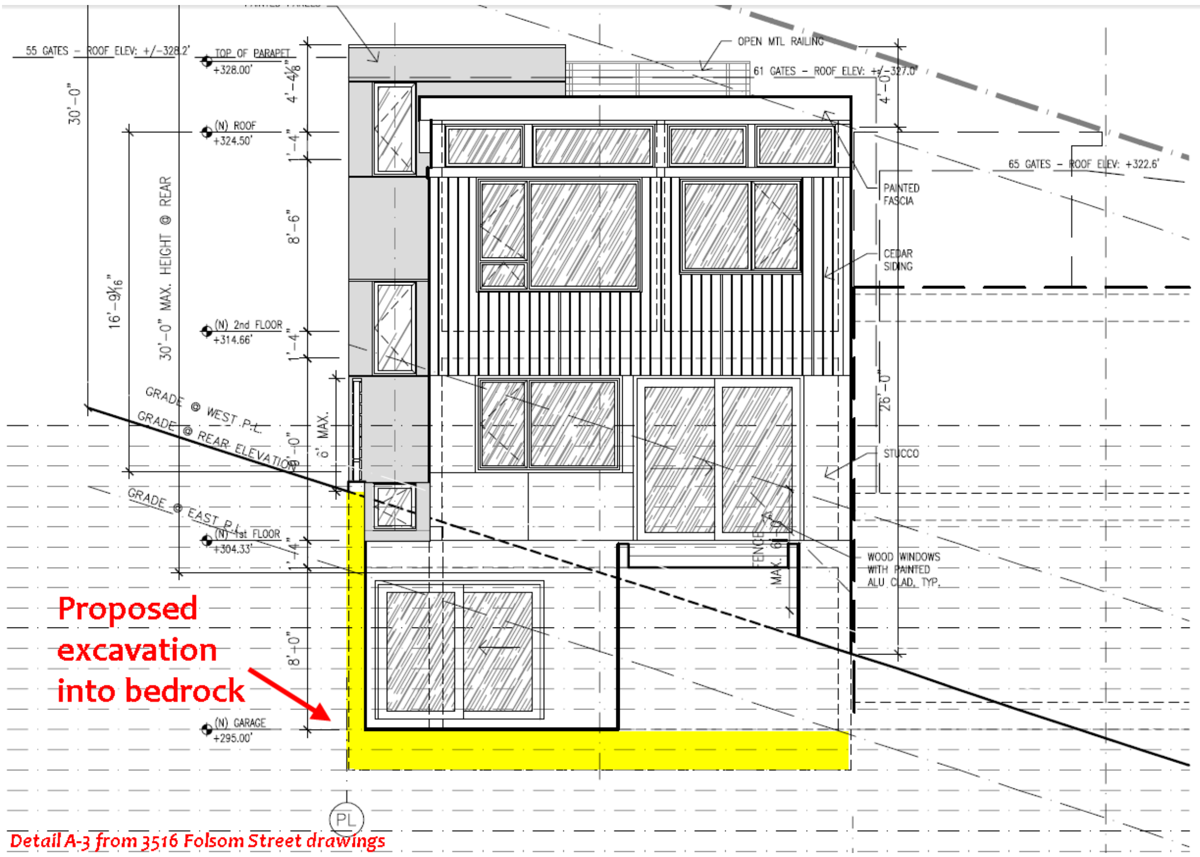


Figure 8: Significant cuts into bedrock resulting in ground vibrations.

No payments for services have been received and no future promises of compensation have been offered.

I reserve the right to update my independent review based on new information.

Please contact me with any questions or comments by phone at (510) 225-5389 or via email at rune@storesundconsulting.com.

REGISTERED PROFESSIONAL ENGINEER
RUNE STORESUND
GE 2855
6/30/17
GEOTECHNICAL
STATE OF CALIFORNIA

Sincerely,

STORESUND CONSULTING

Rune Storesund, D.Eng., P.E., G.E.
Consulting Engineer

UC Berkeley Center for Catastrophic Risk Management
Executive Director

Attachment Dr. Rune Storesund Resume

EDUCATION:

D. Eng Civil Engineering, University of California, Berkeley, 2004-2009
(*Dissertation: Life-Cycle Reliability-Based River Restoration*)

Management of Technology Certificate Program, HAAS, UC Berkeley, 2007

M.S. Civil Engineering, University of California, Berkeley, 2002 (Geotechnical Engineering)

B.S. Civil Engineering, University of California, Berkeley, 2000
B.A. Anthropology, University of California, Santa Cruz, 2000

QUALIFICATIONS:

- California, Civil Engineer, RCE 64473
- California, Geotechnical Engineer, GE 2855
- Louisiana, Civil Engineer, RCE 35034
- Hawaii, Civil Engineer PE-15439
- Washington, Civil Engineer PE 52924
- California Safety Assessment Program Disaster Service Worker
- NAUI Scuba Diver Openwater I (1994)
- Offshore Survival Certification

EXPERIENCE:

Dr. Storesund has 16 years of planning, design, engineering, and construction experience and has worked on a variety of projects throughout California, the United States, and internationally. Dr. Storesund provides consulting services in all aspects of civil, geotechnical, water resources, ecological, restoration, and sustainability engineering projects. His expertise is on the application of reliability and risk-based approaches to engineering projects (with a specialization in environmental restoration and flood control projects) in order to effectively manage project uncertainties. Dr. Storesund has participated in all aspects of engineering projects; from preliminary reviews to detailed analyses to construction observations and post-project monitoring. He provides expert forensic engineering services for geotechnical and civil infrastructure systems. In addition to traditional engineering services, he provides consultations on field instrumentation and monitoring programs as well as Terrestrial LiDAR field survey services. His doctoral research was on life-cycle, reliability-based river restoration.

Dr. Storesund is the Executive Director of UC Berkeley's Center for Catastrophic Risk Management (risk.berkeley.edu). The Center for Catastrophic Risk Management (CCRM) is a group of academic researchers and practitioners who recognize the need for interdisciplinary solutions to avoid and mitigate tragic events. This group of internationally recognized experts in the fields of engineering, social science, medicine, public health, public policy, and law was formed following the tragic consequences of Hurricane Katrina to formulate ways for researchers and experts to share their lifesaving knowledge and experience with industry and government. CCRM's international membership provides experience across cultures and industries that demonstrate widespread susceptibility to pervasive threats and the inadequacy of popular, checklist-based remedies that are unlikely to serve in the face of truly challenging problems.

Dr. Storesund serves as an on-call expert Geotechnical Engineer to the State of California's Department of Consumer Affairs for their annual examination.

PROJECTS: Projects Dr. Storesund has worked on are listed below:

Environmental Restoration

Louisiana Coastal Protection and Restoration (LACPR): Working with Environmental Defense, Dr. Storesund provided consultation services on proposed coastal restoration efforts in Louisiana, submitted by the United States Army Corps of Engineers (USACE). Dr. Storesund developed planning and design evaluation metrics by which to evaluate the adequacy of the proposed restoration alternatives. Additionally, Dr. Storesund is perfored a technical review of the risk-based design prepared by the USACE.

Yosemite Slough Restoration: Dr. Storesund served as a project engineer, providing geotechnical recommendations during design. Project specifications were developed for this restoration project in San Francisco, California. The USACE SPECSINTACT program was used to develop the specifications.

Hamilton Wetland Restoration Project Shaping Contract, Novato, California: Dr. Storesund served as the geotechnical engineer of record for this earthwork project to shape dredge spoils into habitat features. Four areas (North Seasonal Wetland, Wildlife Corridor, Tidal Panne, and South Seasonal Wetland), each having different habitat requirements, were configured as part of the restoration project. A special low-permeability bottom was developed to minimize water infiltration and maximize salt retention in the seasonal tidal areas (habitat feature).

Redwood Creek, Napa County, California: Dr. Storesund provided topographic as-built and photographic documentation for this in-stream habitat enhancement project. Boulder features were added to provide channel roughness and resting pools for migrating fish.

Upper Napa River Restoration Project, Napa County, California: Dr. Storesund served as the lead engineer providing civil, geotechnical, environmental, hydrological engineering and topographic mapping services for a four-mile stretch of the Napa River south of Calistoga, California. The project was sponsored by the California Land Stewardship Institute.

Sulphur Creek Monitoring, Hayward, California: Dr. Storesund is conducting annual geomorphic monitoring (for a total of 10 years) of this completed restoration project in Hayward, California. The project included slope stabilization and installation of habitat features (rock boulders). The monitoring includes surveys (cross-sectional, thalweg) and photo monitoring.

Kirby Canyon Landfill Mitigation, Santa Clara County, California: Dr. Storesund provided geotechnical engineering recommendations for this dam removal and creek restoration project. The site is located in a very steep canyon, with high gradients. In addition, the dam had been overtopped during previous storms, resulting in very deeply incised ravines forming (which needed to be backfilled).

Waldo Point Wetland Restoration, Marin County, California: This project is a wetland restoration project. Dr. Storesund provided topographic survey and piezometer monitoring services to establish connectivity parameters between San Francisco Bay and the proposed wetland mitigation site.

Huichica Creek Fish Passage: A fish-friendly culvert was designed as part of Caltran's Highway 36 widening project in Sonoma County, California. Dr. Storesund developed the conceptual and final designs, project specifications, and project cost estimate.

Great Valley Grasslands, Merced County, California: Dr. Storesund served as the project manager and project engineer for this floodplain reconnection project at the Great Valley Grasslands State Park. His evaluations consisted of a site reconnaissance, erosion/scour susceptibility screening, and hydraulic analysis of inundation through a series of existing culverts.

Pond 1 Restoration, Mountain View, California: Storesund Consulting performed a topographic survey of existing conditions to develop a base map for grading to alter onsite flood discharge to minimize inundation times (and prevent die-off of vegetation due to temporary storm water retainage). We developed grading plans, specifications, performed construction staking and performed an as-built survey using Terrestrial LiDAR methods.

ECCC Souzal, Antioch, California: Storesund Consulting performed a high-resolution RTK GPS survey of this wildlife area in order to generate a detailed topo to evaluate micro-watersheds for vernal pool development.

Hess Creek Restoration, Clayton, California: Storesund Consulting performed a high-resolution RTK GPS survey of this incised creek stretch to be restored. The survey results were integrated with available aerial LiDAR topography. We also provided geotechnical recommendations for the restoration plans.

Rancho San Vicente, New Almaden, California: Storesund Consulting provided geotechnical recommendations for this restoration project which involved the removal/stabilization of 16,000 CY of earthen fill dumped into a ravine on County Park Land. The recommendations involved environmental contamination, grading operations, temporary haul roads, slope stability, and earthwork.

Port of Richmond, Operable Unit 2: Dr. Storesund provided geotechnical design on this environmental remediation and restoration project within the Port of Richmond. The mitigation consisted of a subaqueous cap (comprised of Bay Mud) in the inlet, installation of rip-rap along the shoreline revetment zone, and installation of a concrete facing and asphalt concrete cap to isolate in place sediments.

Port of Oakland, Operable Unit 2: Dr. Storesund provided geotechnical design support services to Land Marine Geotechnics on this reclamation and restoration project within the Port of Oakland. Dredged spoils were used to abandon a deep-draft U.S. Navy pier at the Port of Oakland.

Storm Water Pollution Prevention Plans

Oakley Civic Center Frontage Improvements, State Route 4, Oakley, California: A SWPPP was prepared for this widening project in Oakley. The existing Main Street in the project limits has two westbound lanes and one lane eastbound. The project added pavement, roadway entries/exits, curb, gutter and sidewalks on the south side of Main Street, as well as street lights along both sides of Main Street.

Brentwood Boulevard Widening and Reconstruction From Woodfield Lane to Central Boulevard, Brentwood, California: A SWPPP was prepared for this project which widens the current Brentwood Boulevard (State Route 4) between Woodfield Lane and Central Boulevard from the existing geometry of a three-lane with two way left turn lanes to a four-lane roadway with a raised landscape median and turn pockets at intersections. Project demolition included removal of curb and gutter, sidewalk sections, damaged pavement sections, and removal of select trees.

Mainstreet Roadway Improvement Plans for Subdivision 8916, Oakley, California: A SWPPP was prepared for this roadway improvement project in Oakley, California. The project added pavement curb & gutter and sidewalk to the west side of the existing roadway in order to facilitate future addition of a second eastbound lane.

Sand Creek Road Intersection Improvement Project, Brentwood, California: A SWPPP was prepared for this project which expands an existing intersection and widens the roadway. The project added pavement, curb & gutter, and sidewalks.

Sausalito Yacht Harbor, Sausalito, California: Dr. Storesund developed a design for treatment of storm water runoff in the large parking lot adjacent to the Sausalito Yacht Harbor as part of a bulkhead wall replacement project. The design involved the installation of a permeable rock infiltration zone under a walkway area. This infiltration area was designed to treat storm water runoff before it enters Richardson Bay.

Flood Control

California Rural Levee Repair Criteria Committee: This advisory committee was charged with developing rural levee repair and improvement criteria to be applied for planned or emergency work. The group worked in conjunction with DWR, interested stakeholders, and USACE. Dr. Storesund provided engineering (seismic, geotechnical marine, ecological, water resources) and risk-based decision making input to this group. This committee was active between 2012 and 2014.

USACE West Sacramento Flood Control Project, West Sacramento, California: Dr. Storesund served as a field engineer responsible for field construction quality control program, which consisted of sand cone density testing, nuclear gauge density testing, associated geotechnical laboratory testing, and issuing a final services during construction report.

Warm Springs Dam Control Structure Study, Sonoma County, California: Dr. Storesund served as the project manager and project engineer for this crack evaluation study for the San Francisco US Army Corps of Engineers. The study was performed in conjunction with PB. The vertical control structure for Warm Springs Dam suffered from water infiltration due to cracking of the concrete control structure. A LiDAR imaging and visual observation mapping was conducted of the cracks. Repair recommendations and cost estimate were provided to the US Army Corps of Engineers.

Las Gallinas Coastal Inundation Study, Marin County, California: Dr. Storesund served as a project engineer for this study (for the San Francisco US Army Corps of Engineers) that evaluated overtopping conditions during storm events for an existing flood protection system. Dr. Storesund developed a GIS terrain and inundation maps based on overtopping analyses.

Upper Penitencia Creek, Subsurface Geotechnical Exploration, Santa Clara County, California: Dr. Storesund served as the project engineer for this United States Corps of Engineers project which consists of on-land, subsurface geotechnical exploration along a portion of Upper Penitencia Creek. The requested services include drilling, sampling, field classification, laboratory testing, and Unified Soil Classification System (USCS) for soil borings at select locations along the creek alignment. The purpose of the soil borings was to provide subsurface data for the preliminary design of flood control structures, such as levees, floodwalls, culverts, and weirs along Upper Penitencia Creek. Dr. Storesund coordinated and managed Fugro's field operation exploration program that consisted of 22 soil test borings. Following the field exploration, Dr. Storesund managed the QA/QC review of all field and laboratory data. Dr. Storesund also managed the data report preparation.

Geotechnical Study Northern Borrow Area, Bulge And Pacheco Pond Levees, Hamilton Wetlands Restoration Area, Novato, California: Dr. Storesund served as the project engineer for this project which consisted of a geotechnical study for the Bulge and Pacheco Levees located in the Hamilton Wetlands Restoration Area. The project site is situated at the former Hamilton Army Air Field in Novato, California. The purpose of the geotechnical field exploration and laboratory testing program was to obtain information on subsurface conditions in the Northern Borrow Area in order to estimate the amount and nature of potential borrow material. The scope of services performed included:

- Conducting a field exploration program consisting of 18 test pits to determine the subsurface profile in the Northern Borrow Area;
- Conducting a laboratory testing program to obtain soil properties of the samples collected during our field exploration; and
- Preparing this geotechnical report presenting the results of our geotechnical field exploration, laboratory testing program, and a discussion of the exploration results.
- Specified development / review

USACE San Lorenzo Flood Control, Santa Cruz, California: Dr. Storesund served as a field engineer responsible for field density testing, performing associated geotechnical laboratory testing, and issuing a final services during construction report for this levee project in Santa Cruz.

USACE Napa River Flood Protection, Napa, California: Dr. Storesund served as a field engineer responsible for field density testing, performing associated geotechnical laboratory testing, and issuing a final services during construction report for this levee project in Napa.

Codornices Creek Restoration Project, Between Fifth and Eighth Streets, Albany and Berkeley, California: Dr. Storesund served as the project engineer for this geotechnical study. The purpose of this project is to restore the existing Codornices Creek, located between the City of Albany and the City of Berkeley, to a more natural setting using bioengineering and biotechnical methods. Dr. Storesund was responsible for the geotechnical field exploration and laboratory-testing program. The scope of our services included: Compiling and reviewing available geotechnical and geologic data; conducting a field exploration and laboratory-testing program; evaluation of slope stability and erosion susceptibility; development of embankment fill recommendations and general construction considerations; and preparing a final geotechnical report that included the results of our geotechnical field exploration and laboratory testing program, discussion of geotechnical issues, and geotechnical recommendations

Water Storage Reservoirs

Napa, Sonoma, and Lake Counties, California: Provided engineering design recommendations and construction observations services for water storage reservoirs for various agricultural clients. Reservoirs are off-stream, agricultural purpose reservoirs or are on-stream reservoirs with embankment heights less than 25 feet and store less than 50 acre-feet. Thus, the reservoirs are not within the jurisdiction of the California Department of Dam Safety (DSOD). Projects include construction of earth embankments and placement of either low permeability compacted soil liners or installation of geosynthetic liner systems.

- **Brooks Reservoir, Napa County, California:** 2.5 acre-foot, off-stream water storage reservoir formed by constructing three earthen embankments and lined with a geosynthetic liner.
- **Platt Reservoir, Sonoma County, California:** An off-stream reservoir formed by constructing a compacted earthen embankment with on-site soils. The reservoir was lined with a geosynthetic liner. The project included installation of an underdrain system to preclude the “floating” of the synthetic liner if the reservoir is drained during periods of high groundwater as well as a cut slope drain to intercept hillside groundwater flows. Dr. Storesund was also responsible for issuing a final services during construction report for the project.

- **Mondavi Dutra Dairy Reservoir, Napa County, California:** Dr. Storesund served as a field engineer responsible for embankment keyway inspections, field density testing, and concrete placement quality control during the enlargement of this reservoir in Napa County. Dr. Storesund was also responsible for issuing a final geotechnical services during construction report for the project.
- **Amber Knolls Reservoir, Lake County, California:** Dr. Storesund served as a field engineer responsible for embankment keyway inspections, field density testing, and concrete placement quality control during the construction of this reservoir in Lake County. Dr. Storesund was also responsible for issuing a final geotechnical services during construction report for the project.
- **Red Hills Reservoir, Lake County, California:** Dr. Storesund served as a field engineer responsible for embankment keyway inspections, field density testing, and concrete placement quality control during the construction of this reservoir in Lake County. Dr. Storesund was also responsible for issuing a final geotechnical services during construction report for the project.
- **Chimney Rock Vineyard, Napa County, California:** Dr. Storesund served as a field engineer responsible for embankment keyway inspections and field density testing during the construction of this reservoir in Napa County.
- **Hershey Vineyard Reservoir, Sonoma County, California:** Dr. Storesund served as a staff engineer responsible for generating design recommentions and issuing of a final geotechnical design report for this reservoir project in Sonoma County.
- **BV Reservoir No. 10 Rehabilitiation, St. Helena, California:** Dr. Storesund served as a field engineer responsible for the execution of the field investigation program and issuance of a final geotechnical design report for this reservoir rehabilitation project in St. Helena.

Off-Stream Storage Projects (Sonoma and Santa Clara Counties, California): Dr. Storesund worked in close conjunction with the Center for Ecosystem Management and Restoration (CEMAR) and Trout Unlimited (TU) on a number of off-stream water storage reservoir projects, designed to help landowners manage water resources in a manner that balances water use with habitat and minimum required in-stream flows for listed coho salmon and steelhead trout. These projects include:

- **Grape Creek Streamflow Stewardship Project, Healdsburg, California:** Dr. Storesund served as the project manager and project engineer for this off-stream reservoir storage project, providing all aspects of engineering planning (permit assistance, conceptual layouts), design (site geotechnical exploration and survey, analyses, development of plans, specifications, and estimates), and construction oversight during construction. The Grape Creek Streamflow Stewardship Project (GCSSP) is a cooperative project designed to help landowners manage water resources in a manner that balances water use with habitat and minimum required in-stream flows for listed coho salmon and steelhead trout. An existing flashboard dam and containment berm was replaced with a new reservoir adjacent to the creek to allow passage of river flows while providing the farmer with an agricultural water supply.
- **Little Arthur Creek Streamflow Stewardship, Healdsburg, California:** Dr. Storesund served as the project manager and project engineer for this off-stream reservoir storage project, providing all aspects of engineering planning (permit assistance, conceptual layouts), design (site geotechnical exploration and survey, analyses, development of plans, specifications, and estimates), and construction oversight during construction. The Little Arthur Creek Streamflow Stewardship Project (LACSSP) is a cooperative project designed to help landowners develop water supply security in a manner that improves in stream flows and habitat for listed steelhead trout.
- **Pescadero Creek Streamflow Stewardship, Healdsburg, California:** Dr. Storesund served as the project manager and project engineer for this off-stream reservoir storage project, providing all aspects of engineering planning (permit assistance, conceptual layouts), design (site geotechnical exploration and survey, analyses, development of plans, specifications, and estimates), and construction oversight during construction. The Pescadero Creek Streamflow Stewardship Project is a cooperative project designed to help landowners develop water supply security in a manner that improves in stream flows and habitat.

Whitethorn Elementary School Auxiliary Water Storage System, Whitethorn, California: Dr. Storesund served as the principal engineer on this conservation project performed in collaboration with Trout Unlimited and Sanctuary Forest. The project entailed installation of sixteen 5,000 gallon water tanks so that the school could divert water during wet months. Dr. Storesund performed the permitting, planning, engineering, construction bid documentation, and review services.

Residential

MLK Plaza Homes, Oakland, California: Dr. Storesund provided field density testing services for this low income housing project in Oakland. The project consisted of constructing thirteen new two-story residential structures at the site as well as associated improvements.

Standard Pacific Homes' Dublin Ranch, Dublin, California: Dr. Storesund served as a field engineer for this residential development in Dublin, observing mass grading operations, performed field density tests on housing pads, roadways, utility trenches, special inspections on rebar placement, concrete placement, post-tensioning, and performed related geotechnical laboratory testing. Dr. Storesund was also responsible for inspection and evaluation of erosion control systems in place during mass grading operations.

Palomares Hills, San Anselmo, California: Dr. Storesund served as a field engineer providing construction observations and field density testing during construction of retaining walls for this residential development.

Lund Ranch Creek, Pleasanton, California: Dr. Storesund provided construction observation services during a creek restoration project located within the Lund Ranch Creek residential development in Pleasanton. The restoration project involved bank erosion mitigation through placement of rock rip rap.

University Avenue Housing, Berkeley, California: Dr. Storesund served as a field and project engineer for this multi-unit residential housing project. An existing Salvation Army structure and parking lot were demolished and replaced with the new housing structure. Dr. Storesund performed the field exploration, engineering analyses, foundation recommendations, and prepared the final geotechnical design report.

The Estates at Happy Valley, Sun City, Arizona: Dr. Storesund served as a field engineer responsible for the execution of a field investigation program, which involved hollow stem auger drilling and geotechnical sampling for this mass grading residential development project in Sun City.

Educational

Children's Hospital Oakland Upgrade, Oakland, California: Dr. Storesund served as a staff engineering providing pipeline thrust block design recommendations for this facility upgrade project in Oakland.

Bessie Carmichael School, San Francisco, California: Dr. Storesund served as a staff engineer providing drilled pier design recommendations for this new school situated between the existing Saint Michael Ukrainian Orthodox Church and the Vineyard Christian Fellowship Church in San Francisco. It is three-story structure with a total footprint area of approximately 24,000 square feet. The facility features a single-story gymnasium and multi-purpose room with an elevated roof, a central courtyard area, and an asphalt-paved playground adjacent to the school building.

Blue Oaks School, Napa, California: Dr. Storesund served as a field engineer for this school renovation project in Napa. The field services consisted of field density testing on pavement subgrades and base rock.

Vista College Facility, Berkeley, California: Dr. Storesund served as a field engineer responsible for logging test pits to identify the foundations for existing structures surrounding the project site. The facility upgrade consisted of a new six to eight-story building for Vista College on the south side of Center Street, between Shattuck Avenue and Milvia Street in Berkeley. Excavations on the order of 15 to 20 feet were required to construct the basement level. The new foundations consisted of 36-inch diameter drilled piers with lengths from 50 to 70 feet.

New Alameda Elementary School, Alameda, California: Dr. Storesund served field as a field engineer responsible for the execution of the field exploration for this project. The new school will consist of classroom buildings and multi-use buildings. The scope of work for this investigation included a site reconnaissance by a State of California Certified Engineering Geologist, subsurface exploration utilizing both exploratory borings and Cone Penetration Testing, laboratory testing, engineering analyses of the field and laboratory data, and preparation of this report. The data obtained and the analyses performed were for the purpose of providing design and construction criteria for site earthwork, building foundations, slab-on-grade floors, retaining walls and pavements.

Ocean Branch Library, San Francisco, California: Dr. Storesund served as a staff engineer responsible for generating foundation recommendations for this new library structure in San Francisco.

Commercial

Clear Channel Outdoor, Oakland, California: Dr. Storesund served as a staff engineer responsible for providing drilled pier design recommendations for this outdoor billboard structure. The proposed billboard structure was supported by four 24-inch diameter, 3/8-inch thick hollow steel pipe columns.

JB Radiator Complex, Sacramento, California: Dr. Storesund provided geotechnical recommendations for foundation grading for a new storage tank at a site with expansive soils.

Linde Processing Facility, Richmond, California: Dr. Storesund performed a field exploration program (CPT) to characterize onsite soil conditions and provided foundation design recommendations for new infrastructure developments at the property.

Moraga Country Club Landslide Mitigation, Moraga, California: Dr. Storesund served as a field engineer for three landslide mitigation projects at the Moraga Country Club. Dr. Storesund provided field density testing services and general construction observations. He was responsible for summarizing the field data and issuing a construction report.

Moss Landing Powerplant, Moss Landing, California: Dr. Storesund served as a field engineer for this power plant upgrade project in Moss Landing. Dr. Storesund provided construction observations auger cast pile installation for the main generating structure and piezometer monitoring during the construction and dewatering of the water cooling intake structure.

Coliseum Lexus Dealership, Oakland, California: Dr. Storesund served as a staff engineer responsible for generating foundation design recommendations and issuing the final geotechnical report for this dealership in Oakland.

Infiniti of Oakland Dealership, Oakland, California: Dr. Storesund served as a field engineer responsible for the implementation and execution of the field investigation program for this project which consisted of advancing three cone penetration tests (CPTs). In addition, he was also responsible for generating foundation design recommendations and issuing a final geotechnical design report.

Sho*Ka*Wah Casino Bridge, Hopland, California: Dr. Storesund served as a field engineer for this bridge and parking lot and suspension bridge project in Hopland. Dr. Storesund provided concrete sampling, keyway inspection, and field density testing services during construction.

Anthropologie – Berkeley, Berkeley, California: Dr. Storesund served as a field engineer responsible for executing the field exploration program for this structural upgrade project in Berkeley. Dr. Storesund was also responsible for the issuing of a final geotechnical design report

2150 Shattuck, Berkeley, California: Dr. Storesund served as a field engineer for this seismic retrofit project in Berkeley. Dr. Storesund was responsible for the monitoring of micropile installation and load testing. He was also responsible for quality control of the injected micropile grout.

Bayer Building 55, Berkeley, California: Dr. Storesund served as a field engineer responsible for field density testing services during construction for this new commercial facility in Berkeley.

Chino Bandito, Chandler, Arizona: Dr. Storesund served as a field engineer responsible for the execution of the field investigation program, which involved hollow stem auger drilling and geotechnical sampling for this 11,500 square foot commercial development project in Chandler.

150 Powell Street, San Francisco, California: Dr. Storesund served as the project manager and project engineer for this structural renovation project near Union Square. The historic building required the façade structure to be saved and incorporated into the new structure. Dr. Storesund developed and implemented an exploration program that involved test pits to expose and evaluate the condition of spread footings. Foundation design services were also provided for temporary construction features (tieback walls, support frame for façade) and permanent features (foundations) as well as support and observation services during construction.

390 Fremont Street, San Francisco, California: Dr. Storesund provided geotechnical engineering support to a property owner adjacent to a high-rise construction project that involved installation of a shoring system, excavation to a depth of 70 ft, excavation of soil and bedrock, and development and evaluation of a monitoring program during the excavation activities.

Waterfront and Offshore Facilities

California Tsunami Hazard Policy Committee: The California Tsunami Policy Working Group (CTPWG) is a voluntary advisory body operating under the California Natural Resources Agency (CNRA), Department of Conservation, and is composed of experts in earthquakes, tsunamis, flooding, structural and coastal engineering and natural hazard policy from government, industry, and non-profit natural hazard risk-reduction organizations. The working group serves a dual purpose as an advisor to State programs addressing tsunami hazards and as a consumer of insights from the SAFRR Tsunami Scenario project, raising awareness and facilitating transfer of policy concepts to other coastal states in the nation. CTPWG's role is to identify, evaluate and make recommendations to resolve issues that are preventing full and effective implementation of tsunami hazard mitigation and risk reduction throughout California's coastal communities. Dr. Storesund provided engineering (seismic, geotechnical marine, ecological, water resources) and risk-based decision making input to this group. This committee was active between 2011 and 2013.

Emeryville Shoreline Protection Project, Emeryville, California: Dr. Storesund was a project engineer overseeing the construction of this shoreline improvement project. Site grades were raised 2-4 feet above existing grade and an enlarged shoreline breakwater slope was constructed.

Alcatraz Hydrodynamic Evaluation, City and County of San Francisco, California: Dr. Storesund was the project manager and project engineer for this coastal hazard screening evaluation at Alcatraz. The purpose of the screening was to inform long-range planning activities, accounting for shoreline erosion and sea level rise. The recommendations were provided to the National Park Service, in association with Kleinfelder.

Emeryville Marina Breakwater, Emeryville, California: Dr. Storesund was a project engineer responsible for the planning and execution of a field exploration and geotechnical laboratory testing program for this breakwater and pier project in Emeryville. Dr. Storesund also completed the geotechnical design recommendations and issued the design report.

Nelson's Marine Shoreline Stabilization, Alameda, California: Dr. Storesund served as the project manager and project engineer for this shoreline stabilization and remediation project at an abandoned boat yard within the Oakland Estuary. The project required an alternatives analysis (approach and cost estimate), decision matrix, development of remediation plans, specifications, and estimates. Field efforts included site surveys (RTK GPS) and geotechnical exploration.

Seadrift Shoreline Study, Stinson Beach, California: Dr. Storesund served as a project engineer and performed a site characterization study (based on historical topographic maps and aerial photographs), conducted hydrodynamic characterization, and aided with the design of the extension of an existing sheet pile bulkhead system along Bolinas Lagoon.

Loch Lomond Breakwater Improvement Project, San Rafael, California: Dr. Storesund was the project manager and a project engineer for the improvement of an existing 1,500 foot long rip rap breakwater structure. He performed a hydrodynamic evaluation during the planning phase to establish design criteria, managed the project (preparation of project plans, specifications, and estimates), and provided civil and geotechnical engineering expertise.

Harbor Point Shoreline Stabilization Project, Tiburon, California: Dr. Storesund served as a project engineer and performed a site characterization study (based on historical topographic maps and aerial photographs), conducted hydrodynamic characterization, and aided with the design of a shoreline stabilization solution.

Martin Luther King Jr. Drive Shoreline Study, Bay farm Island, California: Dr. Storesund served as the project manager and project engineer for this Bay Trail feasibility study for the East Bay Regional Park District (teamed with Creegan D'Angelo Engineers). Dr. Storesund prepared a screening-level coastal engineering guidance document and technical review of alternative plan elements.

Richmond Marina Breakwater Improvements, Richmond, California: Dr. Storesund served as a support staff engineer for this breakwater improvement project in Richmond. The project entailed wave and tide surveys, wind pattern evaluations, and preliminary foundation recommendations to upgrade an existing breakwater structure.

Third Street Boat Ramp, Lakeport, California: Dr. Storesund was a staff engineer responsible for organizing and performing the geotechnical exploration for this public boat ramp improvement project in Lakeport.

Dow Chemical Wharf, Pittsburg, California: Dr. Storesund was the project manager and a project engineer for the evaluation of an existing wharf to evaluate its ability to accommodate larger supply ships. After the initial review, Dr. Storesund was responsible for the development of alternatives, preparation of project permits, design of a new mooring system (including specifications and cost estimate), and construction observations and load testing.

Alviso Marina County Park, Alviso, California: Dr. Storesund served as a field engineer responsible for the implementation of Fugro's geotechnical exploration for the Alviso Marina County Park, Phase 1 Master Plan Implementation Project in Alviso. The geotechnical exploration consisted of two test borings, two Cone Penetration Tests (CPTs). Fugro evaluated the geotechnical conditions for the design and construction of the new parking area, a planted mound area (which includes the placement and compaction of up to 5 feet of engineered fill), and a 24-inch high by 18-inch wide flood control wall.

Brooklyn Basin Dredging Study, Oakland, California: Dr. Storesund served as the project manager for this maintenance dredging study commissioned by the San Francisco US Army Corps of Engineers to URS Corporation.

Pipelines and Water tanks

NCFCWCD South Segment Sewer Replacement, Napa, California: Dr. Storesund served as a field engineer, observing construction of a 54-inch to 66-inch diameter sanitary sewer line in Napa. The project, separated into two segments, realigned and replaced approximately 4,500 lineal feet of mainline sewer outside the river flood plain as part of the Napa River Project. Construction observations pertained to pressure grouting ground improvement, pipeline subgrade inspections, pipe bedding and backfill observations, trench backfill density testing, AC pavement density testing, concrete sampling, pipe segment seal testing, and observations of lightweight concrete backfill of old sewer line.

PG&E Line 131 Pigging Project, Alameda County, California: Dr. Storesund served as field engineer, coordinating and conducting geotechnical exploratory test pits for a new PG&E maintenance access facility to service two 18-inch, high-pressure, gas mains. Site improvements included an enlarged access road and maintenance pad, rock cut slopes, and minor pipeline realignment.

Newby Island Gas Transmission Pipeline, Milpitas, California: Dr. Storesund served as a field engineer providing construction observations on trench backfill operations on a landfill methane gas recovery pipeline installed at the base of an existing Santa Clara County Flood Control Levee. Trench backfill consisted of lightweight concrete slurry, designed to isolate the installed pipeline and protect the structural integrity of the existing levee system.

South Transmission System Project Tanks, Sonoma County, California: Dr. Storesund served as a field engineer during the geotechnical exploration of this project. Seven water tank sites were evaluated during the field operations. Geotechnical explorations included seismic refraction studies, vertical soil borings, and geologic reconnaissance mapping.

Girard Vineyard, 50k Gallon Water Tank, Napa County, California: Dr. Storesund served as a field engineer during the geotechnical exploration of this project. Two tank sites were evaluated during the field operations by excavating test pits. Site-specific foundation design recommendations were generated.

Granada Sanitary District CIP, San Mateo County, California: Dr. Storesund organized and performed the field exploration for this project which consisted of “jack and bore” operations under Highway 1 in Granada. Engineering foundation design recommendations were generated for temporary shoring required during the construction process.

Earthquake Fault Explorations

North Livermore Properties, Livermore, California: Dr. Storesund served as a support field engineer for the project geologist on this fault rupture hazard study in Livermore. Tasks included geologic mapping, study of stereo-paired aerial photographs, and an extensive fault trenching investigation. Dr. Storesund was responsible for the setup of the fault trench shoring and dewatering pumping system design. Dr. Storesund also assisted the project geologist in field logging the excavated fault trench.

Centex Homes’ Farber Property, Livermore, California: Dr. Storesund served as a field engineer, assisting the project geologist, for a fault rupture hazard study for a proposed residential development located within the Alquist-Priolo Special Studies Zone for the Greenville Fault. The investigation included excavation and detailed logging of two trenches, totaling over 800 feet in length.

Alameda County Sherriff’s Facility Landslide Assessment, Hayward, California: Dr. Storesund served as a field engineer providing assistance during the fault trenching phase of the field investigation. The project involves demolishing the existing Animal Control Facility and constructing a new 160,000 square foot building that will include facilities for the Sheriff and Coroner and a parking garage for about 500 cars. The proposed building will be a multi-level structure, and the garage will extend one or two levels below grade. The structure will be a critical facility and must remain operational following an earthquake. Other improvements will include driveways, a visitor’s parking lot, underground utilities and landscaping. Preliminary schematics suggest that the facility will occupy the entire 4-acre site. The project included evaluating potential landslide and surface fault rupture hazards at the site.

Osgood Road Fault Trench, Fremont, California: Dr. Storesund served as the project manager responsible for the organization and implementation of backfill operations on a fault rupture hazard study for a proposed new PG&E gas main alignment in Fremont within a BART right-of-way zone. A total of three trenches (totaling approximately 350 linear feet and 12 feet deep) were excavated and backfilled according to BART specifications.

Dumbarton Quarry and Associates, Hayward, California: Dr. Storesund served as a support field engineer for the project geologist on this fault rupture hazard study project at the La Vista Quarry in Hayward. Tasks included geologic mapping, study of stereo-paired aerial photographs, and an extensive fault trenching investigation. Dr. Storesund was responsible for the setup of the fault trench shoring and dewatering pumping system design. Dr. Storesund also assisted the project geologist in field logging the excavated fault trench

LBL-50X AP Fault Study, Berkeley, California: Dr. Storesund acted as a field engineer for the fault location study for a proposed 6-story building to be constructed on a steep hillside within the State designated Fault Rupture Hazard Zone for the active Hayward Fault. The steep, vegetated slope made excavation of continuous trenches difficult and numerous trenches had to be excavated to provide appropriate coverage. No evidence of active or potentially active faulting was encountered in the trenches.

Transportation

Caltrans I-238 Widening Project, Alameda County, California: Dr. Storesund served as both a field engineer responsible for the coordination and implementation of the field investigation program and a staff engineer performing design calculations and analyses. The I-238 project includes the widening of the freeways and related replacement or improvement of existing connectors, overcrossings, and railroad underpasses. Existing embankments are to be widened which requires installation of concrete and MSE retaining wall. Field investigations performed for the project included an extensive subsurface exploration program utilizing continuous flight solid and hollow stem augers, rotary wash borings and Cone Penetration Test (CPTs) soundings. In addition, available subsurface data from previous investigations was reviewed as were published geologic and soil survey data. The field exploration program was complemented with geotechnical laboratory testing. Following completion of the field investigation and laboratory testing, analyses were performed to evaluate geotechnical engineering aspects of project, particularly settlement and liquefaction hazard studies.

Caltrans I-880/Mission Boulevard Widening Project, Alameda County, California: Dr. Storesund served as a support staff engineer for the I880/Mission Boulevard Widening Project. The project involved over 100 test borings, geotechnical laboratory analyses, engineering foundation design recommendations, flexible pavement design, and seismic design criteria for five roadway bridges and one railroad bridge. Other improvements included: a cut and cover tunnel box, box culverts, retaining walls, and ancillary structures.

Caltrans Guadalupe Highway 87 Renovation, San Jose, California: Dr. Storesund served as a field engineer providing AC pavement density testing Quality Control services during the construction phase of this project. The project included widening of the existing Highway 87, construction of a new overpass over Highway 101, and other retaining walls and street improvements.

Port of Oakland's Oakland Airport Expansion, Oakland, California: Dr. Storesund served as a field engineer for this roadway widening and expansion project, providing construction observations and testing services for, utility trench backfill compaction testing, roadway subgrade and base rock density testing, AC pavement testing, and concrete sampling. The project consisted of the construction of new roadway over and underpasses, roadway widening, and utility upgrades.

Petaluma Transit Mall, Petaluma, California: Dr. Storesund was the project engineer for this streetscape project in Petaluma who was responsible for the organization and execution of the field exploration program as well as generating design recommendations. The proposed streetscape improvements included sidewalks, PCC and AC pavements, information kiosks, and lighting standards.

Reid-Hillview Airport, San Jose, California: Dr. Storesund was the field engineer for this runway rehabilitation project. Dr. Storesund was responsible for quality control observations related to pavement section construction.

Nut Tree Airport, Fairfield, California: Dr. Storesund was a field engineer for this runway rehabilitation and expansion project in Fairfield. Dr. Storesund was responsible observations during new runway grading operations, pavement section construction, and provided support during asphalt content laboratory analyses.

First Street Bridge Replacement Project, Napa, California:
Dr. Storesund served as the project engineer for this project which involved the First Street Bridge Replacement Project located in Napa, California. Dr. Storesund coordinated and managed Fugro's field operation exploration program, performed the field exploration, analyzed the collected data, and provided a preliminary geotechnical design report.

Independent Technical Reviews (ITR)

Pier 36/Brannan Street Wharf Demolition, City and County of San Francisco, California: Dr. Storesund served as the project manager and project engineer for this technical review (on behalf of the San Francisco District US Army Corps of Engineers), which consisted of a geotechnical evaluation of submitted calculations and plans. The project entails the demolition of an existing wharf to make room for the construction of a new public open space wharf and associated boating facilities.

Hamilton Wetland Restoration Levee Raising Project, Novato, California: Dr. Storesund served as a project engineer for this technical review (on behalf of the San Francisco District US Army Corps of Engineers), which consisted of a geotechnical evaluation of submitted calculations, plans, and specifications. The project entails the raising of existing flood protection levees to account for settlements (experienced and anticipated) to the levees.

Marysville Unified School District Pipeline Review, Marysville, California: Dr. Storesund, as part of CCRM, performed a review of a natural gas pipeline risk assessment (per California Department of Education protocols) for the Marysville Unified School District.

Twin Rivers Unified School District Pipeline Review, Sacramento, California: Dr. Storesund, as part of CCRM, performed a review of a natural gas field risk assessment (per California Department of Education protocols) for the Twin Rivers Unified School District.

Milford Township School District Pipeline Review, Milford, Pennsylvania: Dr. Storesund, as part of CCRM, performed a review of a natural gas field risk assessment for the Milford Township School District on the citing of a new school.

Princeville, North Carolina Flood Risk Management Feasibility Study Integrated Feasibility Report and Environmental Assessment: Dr. Stroresund served as an expert reviewer for this USACE IEPR for the proposed Princeville flood protection improvement project. The tentatively selected plan (TSP) included measures to extend the existing levee and raise U.S. Highway 258 and Shiloh Farm Road north of the Town of Princeville to create a barrier to circumvention of the existing levee, as well as ramping residential, farm, and commercial driveways and subdivision streets to meet the new elevation. The TSP also includes non-structural measures consisting of an updated flood warning and evacuation plan, continued floodplain management and updating of local building and zoning codes, a flood risk management education and communication plan for both the community and local schools, and flood warning measures, all of which were ultimately deemed essential to an adequate flood risk management strategy for the Town of Princeville. The estimated cost of the TSP is $21,096.00 million.

Risk Assessments

Multiple Lines of Defense, Coastal Louisiana: Dr. Storesund worked in conjunction with the Lake Pontchartrain Basin Foundation to conduct an initial qualitative risk assessment of the hurricane flood protection system in the greater New Orleans area. The assessments follow the Quality Management Assessment System (QMAS) protocols. The assessment provides the basis for initial definition of the system, stakeholders, and identifies primary Factors of Concern. This assessment is the pre-cursor to detailed quantitative risk assessments.

Tsunami Risk-Based Design Committee, Northern California: Dr. Storesund is the Chair of this committee, sponsored by the ASCE San Francisco Section. The aim of the Working Group is to accomplish the following: (1) Formulate a group of appropriate stakeholders (local, county, state, federal levels); (2) Conduct a summary of ‘best practices’ and available resources (perhaps through a series of workshops) (a) Risk standards (b) Hazard studies (reports, maps, etc) (c) Design standards; (3) Develop Policy Statement (goals based on best practices and available info); and (4) Develop Guidelines for Risk-Based Tsunami Design Criteria in Coastal California.

PG&E Risk Management Framework Assessment: Dr. Storesund served as the project manager on an assessment committee to provide insights on their risk management framework. The insights included: (a) is the right RMF being used for the stated goals?; (b) are all significant RMR relationships being captured?; (c) strategies for visualizing and mapping risk; (d) identifying the 'right' risks and prioritizing; and (e) RMF resilience and maturity. Potential actionable outputs include: (1) reference practices (organizational examples); (2) listing of RMF activities to expand and advance; (3) listing RMF activities to modify/reconfigure; and (4) RMF performance metrics (i.e. targeted monitoring and review, leading/lagging indicators).

Forensic Evaluations

Bayer Communications Building, Berkeley, California: Dr. Storesund served as the field engineer to survey and evaluate settlements in the Bayer Communications Building, which was the 'nerve center' for all communication operations at the facility. Site surveys consisted of floor level surveys, review of historical soil exploration programs, and review of nearby construction activities. The study found that excavation operations associated with the upgrade of a sewer line immediately adjacent to the structure led to lateral stress relaxation and vertical displacement of the footings.

Bell Carter Foods Distressed Structure, Lafayette, California: Dr. Storesund organized and performed the foundation exploration which involved drilling soil test borings within the structure using portable hydraulic drilling equipment. The purpose of the project was to identify the foundation instability mechanism and provide mitigation strategies.

Mississippi River Gulf Outlet Wave-Induced Erosion, St. Bernard Parish, Louisiana: Dr. Storesund provided state of the art engineering analyses examining the contribution of damage to the Mississippi River Gulf Outlet levees as a result of wave action from Hurricane Katrina in 2005. The evaluations required the development of a validated method to assess the plausible range of erosion susceptibilities due to wave impact and run-up. These evaluations were published in the ASCE Journal of Waterway, Port, Coastal and Ocean Engineering.

Investigation of the Greater New Orleans Area Flood Defense System Failure, New Orleans, Louisiana: Dr. Storesund was a consultant for the National Science Foundation sponsored investigation of the failure of the New Orleans Flood Defense System. He aided in the initial field reconnaissance to survey system damage and contributed to the technical analyses evaluating system failure mechanisms. He aided in the use of state of the art methods for erosion sampling and testing as well as LiDAR remote sensing survey methods on the Mississippi River Gulf Outlet levees. Copies of the findings from the evaluation can be accessed at: www.ce.berkeley.edu/~new_orleans.

Upper Jones Tract Levee Failure, San Joaquin County, California: Dr. Storesund provided engineering evaluations associated with the June 2004 breach of the Upper Jones Tract Levee in conjunction with Dr. J. David Rogers. The evaluations included bathymetric surveys, RTK GPS surveys, development of digital terrain models using bathymetry and Aerial LiDAR data, hydraulic modeling, and levee failure analyses (seepage, slope stability). Dr. Storesund was responsible for: project management, planning, and tracking; geotechnical engineering evaluation and analyses; hydrodynamic evaluations; general engineering evaluations; standard of care evaluations; technical data evaluation; computer graphics/animations; digital cartography; scientific and technical writing. Dr. Storesund provided deposition and trial testimony.

East Bank Industrial Area (Lower 9th Ward), New Orleans, Louisiana: Dr. Storesund provided engineering support services to Dr. Robert Bea and Dr J. David Rogers for a field exploration program that included geoprobes, CPTs, and pump testing of the onsite "swamp/marsh" material in order to back calculate the permeability of this deposit. The work was performed in close coordination with all experts (plaintiffs and defense). Dr. Storesund served as the project manager for his $1.3 million project (completed in 3 months). Dr. Storesund was responsible for: project management, planning, and tracking; geotechnical engineering evaluation and analyses; hydrodynamic evaluations; general engineering evaluations; standard of care evaluations; technical data evaluation; computer graphics/animations; digital cartography; scientific and technical writing.

PNG Landslide, Papua New Guinea: Storesund Consulting worked in conjunction with Prof. J. David Rogers, Prof. Calvin Alexander, and Mr. Eldon Gath to assess the causal mechanism(s) of a landslide in Papua New Guinea. Available data was reviewed and a field reconnaissance trip to the failure site was performed in summer of 2012. Dr. Storesund provided geotechnical and liar data interpretation services.

LiDAR Surveys

Sunol Dam Removal, Alameda County, California: In 2006, the San Francisco Public Utilities Commission removed Sunil dam to improve fish passage, restore a self-sustaining population of steelhead to the Alameda Creek watershed, and reduce or eliminate an existing public safety hazard. The dam contained an estimated 37,000 yd^3 of impounded sediment. To create a baseline for future monitoring of impounded sediment transport, a combination of Aerial Liar, Terrestrial LiDAR, and conventional survey data was compiled and synthesized to generate a three dimensional model of the study area. High resolution characterization of the impounded sediments was accomplished using Terrestrial LiDAR, with an approximate point spacing of centimeters.

Pit Dam 3 Mapping, Burney, California: Storesund Consulting provided a Terrestrial LiDAR scan of select areas at the PGE Pit Dam 3 facility to aid in the evaluation of a fault system at the site. A high-accuracy point cloud was rendered of the fault are, allowing field geologists to geolocate fault features with high accuracy. Additionally, fault trenches were scanned and rectified orthoimages were rendered to aid in mapping fault trace features.

Quadrus Hill, Menlo Park, California: Storesund Consulting performed Terrestrial LiDAR scanning services for this office complex in a landscaped boulder area where high-precision mapping of boulder features was required to correctly situate a new deck.

Intarcia, Fremont, California: Dr. Storesund provided Terrestrial LiDAR scanning services for this project to map existing structural conditions as well as mechanical, electrical, and plumbing (MEP) facilities to facilitate BIM modeling and routing of new utilities (using 'clash detection').

1245 Market, San Francisco, California: Dr. Storesund provided Terrestrial LiDAR scanning services for this project to map existing structural conditions as well as mechanical, electrical, and plumbing (MEP) facilities to facilitate BIM modeling and routing of new utilities (using 'clash detection').

Veterans Administration Facility, Mather, California: Dr. Storesund provided Terrestrial LiDAR scanning services for this project to map existing structural conditions as well as mechanical, electrical, and plumbing (MEP) facilities to facilitate BIM modeling and routing of new utilities (using 'clash detection').

Yosemite Slough Wetland Erosion Study, San Francisco, California: Storesund Consulting performed annual erosion/deposition monitoring using Terrestrial LiDAR for the wetland restoration project. Hydrodynamic modeling was performed estimating erosion/deposition. This monitoring program provided a high resolution digital terrain model by which to measure erosion/deposition across the restoration area (3 acres).

Causby Mine Survey, Stanislaus County, California: Dr. Storesund served as the project manager and project engineer for this LiDAR mapping project of an abandoned mine tunnel for the U.S. Forest Service. Mapping consisted of the entrance and exit (for construction access) as well as the interior of the tunnel (for volume estimates and layout purposes). State of the Art LiDAR processing software was used to model the interior of the tunnel in 3D.

Tocaloma Backwater Project, Marin County, California: Dr. Storesund provided RTK GPS and Terrestrial LiDAR surveys for this backwater restoration project for the County of Marin. The work was provided for Balance Hydrologics (who performed the design). Aerial LiDAR was merged with the Terrestrial LiDAR to create a full 3D terrain model of the restoration area.

Arroyo de la Laguna, Alameda County, California: Arroyo de la Laguna is part of the stream system that includes the Dublin, Pleasanton, Livermore, as well as upland portions of northern Santa Clara County. Watershed hydrology and channel function have been historically impacted by urbanization (including drainage and flood control), roads, railroads, gravel mining, and the construction of Del Valle Reservoir, resulting in channel incision on the order of six meters. Severe stream bank erosion was identified on the outer bends of an “S” curve of the Arroyo de la Laguna Creek. Terrestrial LiDAR was used to generate cost-effective, high-accuracy mapping of as-built conditions of newly completed stream and river restoration projects, thereby establishing a baseline by which future monitor efforts can evaluate overall project performance through time.

Salt Pond A21, Alameda County, California: Dr. Storesund performed Terrestrial LiDAR survey for researchers at the University of California at Berkeley on this 160-acre wetland restoration project in Fremont, California. The surveys were used to monitor sediment accretion, scour, and erosion progression within this recently breached salt pond.

Tennessee Hollow, San Francisco, California: A storm drain creek daylighting project was completed at the San Francisco Presidio. LiDAR surveys were used to establish baseline topography following completion of construction in January of 2006. Subsequent surveys were performed to evaluate vegetation growth rates and growth zones. The baseline survey is anticipated to serve as an overall baseline by which future channel stability can be evaluated.

AMR, Roseville, California: Storesund Consulting provided high-resolution RTK GPS topographic survey and Terrestrial LiDAR surveys of vernal pools to provide a baseline micro-topographic terrain model which became the design ‘template’ for restoration of 150 acre vernal pool site.

Cache Creek, Woodland, California: Terrestrial LiDAR surveys were conducted at two specific locations where the creek channel shifted into the creek bank, causing the formation of a tall vertical bank. The terrestrial LiDAR surveys were conducted to map the conditions of the vertical bank. Additionally, aerial LiDAR surveys were also performed at this site and future studies will compare and contrast the resolution and accuracy between these two methods at this site.

Goodwin Creek, Oxford, Mississippi: The Goodwin Creek watershed is organized and instrumented for conducting extensive research on upstream erosion, stream erosion and sedimentation, and watershed hydrology. Land use and management practices that influence the rate and amount of sediment delivered to streams from the uplands range from timbered areas to row crops. About 13 percent of the watershed total area is under cultivation and the rest in idle pasture and forest land. Terrestrial LiDAR surveys were performed at one location in an attempt to evaluate the feasibility of utilizing LiDAR to measure and quantify sediment transport and vertical bank retreat rates.

Coldwater Creek, Mississippi: Coldwater Creek is part of a United States Department of Agriculture National Sedimentation Laboratory research watersheds. The quantity and quality of aquatic habitats along the lowland floodplain rivers in agricultural landscapes are in steep decline as a result of nonpoint source pollution. Terrestrial LiDAR surveys were performed at the site of an ephemeral gully in order to ascertain the feasibility of mapping these features with LiDAR to develop 3D surfaces by which more detailed analyses can be performed (including erosion rates) as opposed to the traditional cross-sectional survey method, which may not fully capture the behavior of the site.

Tolay Lake, Petaluma, California: This collaborative effort between the Sonoma County Parks and Recreation, Ducks Unlimited, and United States Geological Survey, will restore a seasonal lake on Tolay Creek in Sonoma County. Existing agricultural fields will be converted to a county park and will serve as a duck reserve in the fall and winter. Terrestrial LiDAR surveys were preformed to develop a detailed topographic map of the project site. Over 200 acres were surveyed in two days.

Ben Mar, Benicia, California: Dr. Storesund performed Terrestrial LiDAR survey for the United States Geological Survey on this 25-acre wetland restoration project in Benicia, California as part of a Caltrans mitigation project. The surveys were used to monitor sediment accretion within the completed restoration area.

Tilden Step Pool, Berkeley, California: Storesund Consulting worked in conjunction with Dr. Anne Chin (University of Colorado, Boulder) by mapping as-built conditions of a step pool sequence in Tilden Park. Change analyses will be performed over three storm events to ascertain step pool stability.

Colorado Wildfire Step Pool Evaluation, Colorado: Storesund Consulting worked in conjunction with Dr. Anne Chin (University of Colorado, Boulder) by analyzing terrestrial LiDAR scans of study areas before and after storm events to ascertain step pool stability.

Verona Bridge Creek Restoration, Pleasanton, California: Storesund Consulting performed a Terrestrial LiDAR survey of this in-stream habitat enhancement and slope stability restoration project in Pleasanton. The project was designed by the National Resource Conservation District.

Tubb, Vallejo, California: Dr. Storesund performed Terrestrial LiDAR survey for the United States Geological Survey on this 60-acre wetland restoration project in Sonoma County, California. The surveys were used to monitor sediment accretion within the completed restoration area.

Rodeo Creek, Hercules, California: LiDAR scanning services were performed on the newly acquired Rodeo Creek East Bay Regional Park property in Rodeo, California. Rodeo Creek was incised 20-30 feet below the floodplain and heavily vegetated, making it difficult to perform conventional topographic surveys. As a result of the LiDAR surveys, a 3D surface, topography, and cross-sections over a 1,000 foot stretch of creek was cost-effectively mapped.

Winfield Pin Oaks Levee Investigation, Winfield, Missouri: The Winfield Pin Oak levee is maintained by the Cap Au Gris Drainage and Levee District. The levee system (Figure 23) is estimated to prevent flooding of the protected area (493 hectares) up to a 14-year return period flood event on the Mississippi River. This site was overtopped for an extended period of time and breached as a result of overtopping-induced erosion. Terrestrial LiDAR surveys (georeferenced using RTK GPS) were performed in October 2008 for subsequent forensic analyses.

Norton Woods Levee Investigation, Elsberry, Missouri: The Elsberry levee at Norton Woods is maintained by the Elsberry Drainage District. This breach was the result of either a through-seepage induced or overtopping-induced (low crest elevation) failure. High water marks observed in the field indicate that the floodwaters did not exceed the general levee crest elevation. Terrestrial LiDAR surveys (georeferenced using RTK GPS) were performed in October 2008 for subsequent forensic analyses.

Kickapoo Levee Investigation, Elsberry, Missouri: The Elsberry levee at Kickapoo is maintained by the Elsberry Drainage District. This breach was reported by local residents to have been the result of through-seepage in the roadway base course that traversed the levee crest. The extents of levee erosion were generally limited to the pre-breach roadway alignment. Terrestrial LiDAR surveys (georeferenced using RTK GPS) were performed in October 2008 for subsequent forensic analyses.

San Francisco Pier 9, San Francisco, California: Storesund Consulting provided Terrestrial LiDAR scanning services for this renovation project to enable a 3D check against existing as-built documentation and facilitate BIM modeling. The new facility is a 3D printing center for Autodesk.

AT&T Facility MEP Scanning, California: Storesund Consulting provided Terrestrial LiDAR scanning services for this expansion project to map existing mechanical, electrical, and plumbing (MEP) facilities to facilitate BIM modeling as well as routing of a new fuel supply pipeline (using 'clash detection').

UCSF Helen Diller Center, San Francisco, California: Storesund Consulting provided Terrestrial LiDAR scanning services for this project to map existing structural conditions as well as mechanical, electrical, and plumbing (MEP) facilities to facilitate BIM modeling and routing of new utilities (using 'clash detection').

Novartis, Burlingame, California: Storesund Consulting provided Terrestrial LiDAR scanning services for this project to map existing structural conditions as well as mechanical, electrical, and plumbing (MEP) facilities to facilitate BIM modeling and routing of new utilities (using 'clash detection').

San Antonio Station, Mountain View, California: Storesund Consulting provided Terrestrial LiDAR scanning services for this project to map existing structural conditions as well as mechanical, electrical, and plumbing (MEP) facilities to facilitate BIM modeling and routing of new utilities (using 'clash detection').

Veterans War Memorial Building, San Francisco, California: Storesund Consulting provided Terrestrial LiDAR scanning services for this project to map existing structural conditions as well as mechanical, electrical, and plumbing (MEP) facilities to facilitate BIM modeling and routing of new utilities (using 'clash detection').

HWY 84 Interchange, Redwood City, California: Storesund Consulting performed a Terrestrial LiDAR scan of the HWY 84/HWY101 interchange in Redwood City to facilitate an improvement program.

Bryants Creek Levee Investigation, Elsberry, Missouri: The Elsberry levee at Kickapoo is maintained by the Elsberry Drainage District. This breach (Figure 52) occurred at the location of a duck pond that was reported to have been installed immediately adjacent to the levee system in order to attract ducks for the duck club located at the site. Terrestrial LiDAR surveys (georeferenced using RTK GPS) were performed in October 2008 for subsequent forensic analyses.

Indian Graves Levee Investigation, Quincy, Illinois: The Indian Graves Levee system is maintained by the Indian Graves Drainage District. The estimated protection level for the levee system is a 50-year return period flood and the protected area encompasses over 2,800 hectares. The sand with clay core levee system is situated immediately East of the Mississippi River. There were three breaches, two under seepage induced and one overtopping induced breach. Terrestrial LiDAR surveys (georeferenced using RTK GPS) were performed in October 2008 for subsequent forensic analyses.

Two Rivers Levee Investigation, Oakdale, Iowa: The Two Rivers Levee system is maintained by the Iowa Flint Creek Levee District No. 16. The estimated protection level for the levee system is a 100-year return period flood and the protected area encompasses approximately 7,100 hectares. The levee system is situated immediately South of the Iowa River, and west of the Mississippi River. Terrestrial LiDAR surveys (georeferenced using RTK GPS) were performed in October 2008 for subsequent forensic analyses.

Emeryville Shoreline Protection Project, Emeryville California: Terrestrial LiDAR was used to measure the volume of boulder rip-rap placed for this shoreline protection project. Due to the high void ratio and irregularity of the boulders, the very high point density of the Terrestrial LiDAR survey provided a more accurate modeling of rip-rap volume than traditional survey methods.

Dutra San Rafael Rock Quarry, San Rafael, California: The Dutra San Rafael quarry is one of the most active quarries in the Bay Area. LiDAR was used to image the physical configuration of the quarry, to create a 3D baseline survey. Subsequent LiDAR surveys will be compared against the initial baseline survey to determine material quantities as well as overall slope stability within the quarry.

Dutra Richmond Quarry, Richmond, California, California: LiDAR surveys were used to monitor a reclamation slope at the inactive Dutra Richmond Quarry. Due to the location of the slope and the geologic contacts, monitoring was required to demonstrate that no active movements are occurring and that the slope is stable. An initial baseline survey was performed in August, 2006 and subsequent surveys will be compared to the initial baseline to determine activity level.

Lower Santa Ynez, Santa Barbara County, California: The Lower Santa Ynez Bank Stabilization project was a collaborative effort with the California Conservation Corps and California Department of Fish and Game to utilize biotechnical methods to stabilize a 1,000-foot length of stream bank, adjacent to agricultural lands. Terrestrial LiDAR surveys were conducted to develop pre-project topography, as-built topography, erosion and scour quantities and estimated rates, and a coarse vegetation monitoring study.

Emery Point, Emeryville, California: Baseline Terrestrial LiDAR surveys were performed to monitor wave-induced erosion on Point Emery in Emeryville, California, which has experienced significant scour in the last 5 years. This man-made peninsula is a popular location with windsurfers and SF Bay Trail users. It is estimated that the location will be completely eroded in the next 25 years without mitigation.

Fremont Landing, Yolo County, California: The Fremont Landing project site is located along the south bank of the Sacramento River from RM 78.8 to 80.4 in one of the most hydraulically-complex portions of the river. At least five (5) major tributaries or distributaries are located within 2 miles of the site and all influence the hydrodynamics of the site. Terrestrial LiDAR surveys were performed to aid PWA develop a 2D hydrodynamic model of the project site and surrounding tributaries/distributaries. The model was used to allow examination of design issues related to fish stranding, rearing habitat, and flood conveyance.

Hamilton Wetland Restoration, Novato, California: This is a United States Army Corps of Engineers and California Coastal Commission joint project to convert over 500 acres of a decommissioned army airfield to a wetland restoration area using dredged spoil material. The area will consist of seasonal and tidal wetlands. Terrestrial LiDAR is being used to monitor fill placement and obtain volume quantities.

Mississippi River Gulf Outlet, New Orleans, Louisiana: LiDAR surveys were conducted of the southeastern completed levee segment. This survey was to serve as a baseline from which future LiDAR surveys can be conducted and analyses and evaluations of wind-induced wave impacts can be studies.

East Sand Slough Restoration, Red Bluff, California: Dr. Storesund provided terrestrial LiDAR mapping of this channel restoration project on the Sacramento River in Red Bluff, California. The LiDAR survey was integrated with existing bathymetry data. Habitat mapping using the collected LiDAR data was also conducted in general conformance with the California Rapid Assessment Method (CRAM) for Wetlands.

CZ-1 Site, Fresno County, California: Dr. Storesund provided terrestrial LiDAR mapping of this tree-root excavation and measurement study by Dr. Peter Hartsough (UC Davis) as part of his climate change research. The mapping of the tree roots provided Dr. Hartsough the ability to establish high-resolution digital root system baselines for future comparisons.

Research Projects

RESIN: Contemporary infrastructure, the systems necessary to provide sustainable services within the nation's power, transportation, waste management, water, and telecommunication sectors, has become very *complex*; that is adaptive, interdependent, unpredictable, nonlinear, and dynamic. This research seeks to discover new fundamental methods to assess and manage the resilience and sustainability of such complex systems (termed 3ICIS). These methods will facilitate the characterization of both resilience and sustainability by addressing multi-infrastructure, multi-physics, multi-scale (spatial, temporal), and multi-resource phenomena that impact the likelihood of these systems failing to achieve acceptable resilience and sustainability, as well as the associated consequences. The setting selected to develop these methods is the California Sacramento Delta focusing primarily on the following four critical infrastructure services, as well as interfaces with other critical infrastructure sectors as necessary:

- Water Supply – Includes water supply system for agriculture, commercial/industry, government, and the public. Issues of importance include supply, conveyance, and quality (*note: wastewater is part of this, but not addressed here*);
- Flood Protection – Includes the structural elements (levees, floodwalls, flood gates, dams, diversion channels, storm drain systems) as well as the natural rivers corridors, subsidence, settlement & consolidation, and hydrologic hazards (rain storms, snow melt) that inundate low lying areas and floodplains;
- Power Supply – Elements of the electrical power grid that supply electricity to agricultural, commercial/industrial, government and the public; and
- Ecosystem – Physical and biological components of the environment. Physical attributes include habitat areas, soil substrates, water supply and quality. Biological considerations include flora and fauna.

The California Sacramento Delta 3ICIS is a very complex highly interactive 'legacy' system embedded in similarly complex natural environmental and social - political systems. It is of critical importance directly for the population and environment of the State of California and indirectly for the rest of the United States.

The goals of this research project are to develop the following Quality Management Assessment System Process (QMAS):

1. System Definition and Conceptualization
2. Domain Expert / Key Informant Assessment Team Identification and Formation
3. Identification of the key vulnerabilities or chokepoints (aka Factors of Concern)
4. Failure Scenario Development
5. Detailed Qualitative and Quantitative Risk Assessment and Management that accounts for 3ICIS spatial variability, temporal variability (historical, current, future), and non-linearity (SYRAS++)

This research will answer the following fundamental questions:

1. What are the major drivers that threaten Resilience & Sustainability (current, future)?
2. What is the current Resilience & Sustainability state of the 3ICIS?
3. What future Resiliency & Sustainability states are expected given the status quo persists?
4. What are the potential consequences/impacts associated with future Resiliency & Sustainability states given the status quo persists?
5. What adaptation and mitigation strategies can be employed to create an "acceptable" Resilient & Sustainable 3ICIS?

2008 Midwest Levee Failure Investigation: Dr. Storesund was the lead researcher for this National Science Foundation sponsored collaborative research investigation between UC Berkeley, Texas A&M University, and the Missouri University of Science and Technology. The research was an immediate effort to collect sensitive and time-dependent perishable data will comprehensively characterize select levee failure locations to provide essential levee characterization and performance data for use in subsequent numerical analyses. The levee characterization consisted of:

1. An initial field reconnaissance to visit known breach sites along the Mississippi River between St. Louis, MO and Davenport, IA to document (via photographs) site conditions, collect eyewitness accounts, and develop a list for detailed site-specific analyses;

2. Conducting high-detail laser imaging survey (Terrestrial LiDAR) of breach and erosion/scour features in the levees. These surveys will be used to validate future numerical simulations that predict the final scour/erosion profile for specified overtopping conditions;

3. Characterization of the vegetative/grass cover on the earthen levee side slopes to determine erosion-resistance provided. This levee characteristic is frequently omitted from field characterization studies, yet is very important in the performance of the levee during overtopping conditions;

4. Characterization of the levee soil materials, including the United States Soil Classification (USCS) soil types, plasticity (Atterberg Limits), grain size distribution (sieve sizes), in-situ density, maximum dry density, Erosion Function Apparatus (EFA) erodibility characterization and jet erosion testing; and

5. Documentation of the river stage at the location of the levee failure based on eyewitness accounts as well as available USGS Stream Gage Data. This data is essential to correctly evaluate overtopping depths and durations and associated water velocities on the ‘protected side’ of the flood protection levee.

The sites investigated include: Brevator (Missouri); Winfield (MO); Cap au Gris (MO); Kings Lake (MO); Norton Woods (MO); Kickapoo (MO); Bryants Creek (MO); Indian Graves (IL); Two Rivers (IA).

National River Restoration Science Synthesis: The National River Restoration Science Synthesis (NRRSS) was a nation-wide effort to characterize the practice of river restoration. It consisted of three phases: synthesis of national and state restoration databases, phone surveys with select river restoration practitioners, and detailed river restoration post-project appraisals within California. Dr. Storesund was active, under the direction of Dr. G. M. Kondolf, and participated in the completion of 40 post project appraisals (PPA) of California river restoration projects. The PPA evaluations consisted of watershed delineations, hydraulic and hydrology characteristics determinations, review of planning and design approaches, review of permit applications, field surveys and performance assessments, and engineering documentation of post-construction performance.

Projects evaluated:

Ackerman Creek Restoration Project	Alameda Creek (Niles Dam Removal)
Alameda Creek (Sunol Dam Removal)	Alamo Creek (Main Branch)
Alamo Creek (East Branch) Project	Arroyo de la Laguna Bank Stabilization
Arroyo Mocho	Arroyo Viejo Creek Restoration
Baxter Creek (Booker T. Anderson)	Baxter Creek (Gateway)
Baxter Creek (Pointsett Park)	Bear Creek Restoration Project
Blackberry Creek (Thousand Oaks)	Brandy Creek (A-Frame Dam Removal)
Carmel River at deDampierre	Carmel River at Schulte Road
Castro Valley Creek Restoration	Cerrito Creek (El Cerrito Plaza)
Chorro Flats Enhancement Project	Clarks Creek
Clear Creek (McCormic Dam Removal)	Cold Creek
Crocker Creek Dam Removal	Cuneo Creek Restoration
Green Valley Creek	Lower Guadalupe River Reach B
Lower Ritchie Creek Dam Removal	Lower Silver Creek Reach I
Martin Canyon Creek	Miller Creek
Redwood Creek	Sausal Creek Restoration Project
Strawberry Creek	Tassajara Creek
Tennessee Hollow (Thompson Reach)	Uvas Creek Restoration
Village Creek (UC Berkeley)	Wildcat Creek at Alvarado Park
Wildcat Creek Flood Control Channel	Wilder Creek Restoration Project

More information on the NRRSS study and these specific PPA evaluations can

PROFESSIONAL AFFILIATIONS:

ASCE Leadership and Management Committee
Chair 2010 - 2012
Corresponding Member 2003 – 2009
ASCE San Francisco Section
Past President 2012-2013
President 2011-2012
President Elect 2010-2011
Vice President 2009 - 2010
American Society of Civil Engineers: San Francisco Section YMF President 2003-2004
ASCE San Francisco Section Water Resources Group
Director 2009 -2011
ASCE San Francisco Section Geotechnical Society Steering Committee
ASCE San Francisco Section Infrastructure Report Card Committee
ASCE GEO-Institute
National Academy of Forensic Engineers
National Society of Professional Engineers
California Society of Professional Engineers
UC Berkeley Geotechnical Engineering Society
UC Berkeley Engineering Alumni Society

AWARDS:

Eagle Scout, Troop 27, Eureka, California (1992)

Outstanding YMF Civil Engineer (2004) San Francisco Section ASCE

Outstanding YMF Civil Engineer in the Private Sector (2008) Western Regional Younger Member Council, ASCE

Outstanding ASCE Younger Member Forum Officer, ASCE Region 9 (2009)

President's Award, San Francisco Section ASCE (2012)

H.J. Brunnier Award, San Francisco Section ASCE (2013)

ASCE Edmund Friedman Young Engineer Award for Professional Achievement (2013)

Exhibit 2

Mario Ballard & Associates
Building and Fire Code Consultants

March 23, 2016

Subject: 3516-3526 Folsom Street
Fire Department Access

References:

-California Fire Code Section 503 "Fire Apparatus Access Roads"
-San Francisco Fire Department Informational Bulletin 5.01
-Department of Public Works 2015 Subdivision Regulation
-Table of contents Appendix-Technical Specifications Related to Engineering Document Section XII-B-3

The California Fire Code, San Francisco Fire Department Technical Bulletin 5.01 and the DPW 2015 Subdivision regulation include specific guidelines and requirements related to street widths, grade, angles of approach and departure and maximum grade related to Ariel truck operation.

Based on the information reviewed, the proposed development of Folsom Street North of Chapman will not meet the required specifications for Fire Department apparatus (See SFFD Bulletin 5.01) or Fire Department ambulance (EMR) access. All equipment, ladders, hoses as well as emergency medical equipment and supplies will need to be manually transported to the incident site which could impact firefighting operations and EMR response.

Mario Ballard

Mario Ballard

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Mario Ballard, Principal

CAREER SUMMARY

Principal, Mario Ballard and Associates	5/1/2007-Present
Principal, Zari Consulting Group	1/1/2013-Present
Captain, Bureau of Fire Prevention, Plan Review Division	2001- 4/21/2007
Lieutenant, Bureau of Fire Prevention, Plan Check Division	1994 - 2001
Inspector, San Francisco Fire Department	1991 - 1994
Firefighter, San Francisco Fire Department	1974 - 1991
Linebarger Plumbing and Construction, SF CA	1974 - 1980
Servadei Plumbing Company, SF CA	1974
United States Army, Army Security Agency	1972 - 1974

LICENSES

ICC, International Code Conference Certified Building Plans Examiner

CERTIFICATIONS

ICC Advanced Occupancy
ICC Advanced Schematic Design
ICC Building Areas and Fire Design
ICC Advanced Types of Construction
ICC Advanced Means of Egress
CFCA Certificate of Training of Locally Adopted Ordinances and Resolutions
IFC Institute Certificate Application of the UBC for Fire Code Enforcement
ICBO Certificate on Course Completion on Fundamentals of Exiting
ICBO Certificate on Course Completion Complex Exiting
ICBO Certificate on Course Completion Building Use and Construction Type
ICBO Certificate on Course Completion Fire Protection, Building Size and Location
ICBO Course Overview of the Uniform Building Code
California Fire Chief's Association Fire Prevention Officers' Section Fire Alarm Levels I & II
Fire Sprinkler Advisory Board of Northern California & Sprinkler Fitter Local 483 Fire Sprinkler Seminar
National Fire Sprinkler Association, Inc., Hydraulics for Sprinklers
EDI Code International, Innovative Code Enforcement Techniques
Certification State of California Title 19/Title 24

EDUCATION

Fire Strategy & Tactics 1981-1993
Fire Service Supervision
Fire Prevention 1A, 1B, 1C
Fire Prevention 2A, 2B
Fire Prevention Officer Level One
Firefighter Level One and Two
Arson 1A, 1B
Hazardous Materials 1A, 1B
Instructor 1A
Fire Management 1A

City College of San Francisco 1970-1972

COMMITTEE INVOLVEMENT

Building Code Advisory Committee
Hunters Point Development Team
Mission Bay Task Force
Treasure Island Development Team
Trans-Bay Transit Center
Muni Metro, Light Rail Third Street Corridor
Department of Building Inspection MIS Case Development
San Francisco Board of Examiners Fire Department Representative
Member California Fire Chief's Association Fire Prevention Officers
BOMA Code Advisory Committee
Mayor's Office of Economic Development Bio-Teck Task Force
Hunters Point Redevelopment Task Force
Building Code Standards Committee 1996-1999
Participant in the Eighth Annual California Fire Prevention-Institute Workshop, "*Providing the Optimum in Fire and Life Safety Training*"
Participant North/South California Fire Prevention Officers Workshops 1996 - 1998
Guest Speaker at SMACNA (Sheet Metal and Air Conditioning Contractors National Association)

PUBLIC SERVICE

Rooms That Rock For Chemo (RTR4C), Director Secretary 2011-Present
San Francisco Spina Bifida Association, (Past) Vice President

California Fire Code Section 503

“Fire Apparatus Access Roads”

FIRE COMMAND CENTER.

FIRE DEPARTMENT MASTER KEY.

FIRE LANE.

KEY BOX.

TRAFFIC CALMING DEVICES.

SECTION 503
FIRE APPARATUS ACCESS ROADS

503.1 Where required. Fire apparatus access roads shall be provided and maintained in accordance with Sections 503.1.1 through 503.1.3.

503.1.1 Buildings and facilities. Approved fire apparatus access roads shall be provided for every facility, building or portion of a building hereafter constructed or moved into or within the jurisdiction. The fire apparatus access road shall comply with the requirements of this section and shall extend to within 150 feet (45 720 mm) of all portions of the facility and all portions of the exterior walls of the first story of the building as measured by an approved route around the exterior of the building or facility.

Exception: The fire code official is authorized to increase the dimension of 150 feet (45 720 mm) where:

1. The building is equipped throughout with an approved automatic sprinkler system installed in accordance with Section 903.3.1.1, 903.3.1.2 or 903.3.1.3.
2. Fire apparatus access roads cannot be installed because of location on property, topography, waterways, nonnegotiable grades or other similar conditions, and an approved alternative means of fire protection is provided.
3. There are not more than two Group R-3 or Group U occupancies.

503.1.2 Additional access. The fire code official is authorized to require more than one fire apparatus access road based on the potential for impairment of a single road by vehicle congestion, condition of terrain, climatic conditions or other factors that could limit access.

503.1.3 High-piled storage. Fire department vehicle access to buildings used for high-piled combustible storage shall comply with the applicable provisions of Chapter 32.

503.2 Specifications. Fire apparatus access roads shall be installed and arranged in accordance with Sections 503.2.1 through 503.2.8.

[California Code of Regulations, Title 19, Division 1, §3.05(a)] Fire Department Access and Egress. (Roads)

(a) Roads. Required access roads from every building to a public street shall be all-weather hard-surfaced (suitable for use by fire apparatus) right-of-way not less than 20 feet in width. Such right-of-way shall be unobstructed and maintained only as access to the public street.

Exception: *The enforcing agency may waive or modify this requirement if in his opinion such all-weather hard-surfaced condition is not necessary in the interest of public safety and welfare.*

503.2.1 Dimensions. Fire apparatus access roads shall have an unobstructed width of not less than ~~20 feet (6096~~ mm), exclusive of shoulders, except for approved security gates in accordance with ~~Section 503.6, and~~ an unobstructed vertical clearance of not less than 13 feet 6 inches (4115 mm).

503.2.2 Authority. The fire code official shall have the authority to require an increase in the minimum access widths where they are inadequate for fire or rescue operations.

503.2.3 Surface. Fire apparatus access roads shall be designed and maintained to support the imposed loads of fire apparatus and shall be surfaced so as to provide all-weather driving capabilities.

503.2.4 Turning radius. The required turning radius of a fire apparatus access road shall be determined by the fire code official.

503.2.5 Dead ends. Dead-end fire apparatus access roads in excess of 150 feet (45 720 mm) in length shall be provided with an approved area for turning around fire apparatus.

503.2.6 Bridges and elevated surfaces. Where a bridge or an elevated surface is part of a fire apparatus access road, the bridge shall be constructed and maintained in accordance with AASHTO HB-17. Bridges and elevated surfaces shall be designed for a live load sufficient to carry the imposed loads of fire apparatus. Vehicle load limits shall be posted at both entrances to bridges when required by the fire code official. Where elevated surfaces designed for emergency vehicle use are adjacent to surfaces which are not designed for such use, approved barriers, approved signs or both shall be installed and maintained when required by the fire code official.

503.2.7 Grade. The grade of the fire apparatus access road shall be within the limits established by the fire code official based on the fire department's apparatus.

503.2.8 Angles of approach and departure. The angles of approach and departure for fire apparatus access roads shall be within the limits established by the fire code official based on the fire department's apparatus.

503.3 Marking. Where required by the fire code official, approved signs or other approved notices or markings that include the words NO PARKING—FIRE LANE shall be provided for fire apparatus access roads to identify such roads or prohibit the obstruction thereof. The means by which fire lanes are designated shall be maintained in a clean and legible condition at all times and be replaced or repaired when necessary to provide adequate visibility.

503.4 Obstruction of fire apparatus access roads. Fire apparatus access roads shall not be obstructed in any manner, including the parking of vehicles. The minimum widths and clearances established ~~in Section 503.2.1~~ shall be maintained at all times.

San Francisco Fire Department
Informational Bulletin 5.01

5.01 Street Widths for Emergency Access

Reference: 2010 S.F.F.C. Sections 503 and Appendix D, Section D105

The Division of Planning and Research of the San Francisco Fire Department has established requirements for minimum street widths to facilitate emergency equipment access. These requirements are specified as follows:

Minimum Street Widths and Access Roads

1. The San Francisco Fire Code (503.2.1) requires a minimum of 20 feet of unobstructed roadway and a vertical clearance of not less than 13' 6' for existing roadways. While a 20 foot wide roadway is permissible, past practice has shown that making ninety degree turns are not possible without the trucks moving into oncoming traffic. The vehicles can make the turn only on one way streets.
2. The San Francisco Fire Code (503.2.5) requires a turnaround for all dead-end fire access roads in excess of 150'. The San Francisco Fire Department has determined an 80 foot turnaround and a 40' radius to be sufficient.
3. The San Francisco Fire Code requires a minimum 26' wide street for new developments where the new buildings are greater than 30' in height from the lowest level of fire department vehicle access and are unsprinklered. These streets shall be located a minimum of 15' and a maximum of 30' from the buildings and shall be parallel to one entire side of the buildings.

SAN FRANCISCO FIRE DEPARTMENT VEHICLE SPECIFICATIONS

	ENGINES	TRUCKS
Outside tire extremity	8 ft. 2 in.	8 ft. 3 in.
Vehicle width (with mirrors)	10 ft. 4 in.	10 ft 1 in.
Truck width with one jack extended	n/a	12 ft. 9 in.
Truck width with two jacks extended	n/a	17 ft. 9 in.
Vehicle height	11 ft.	12 ft.
Length of vehicle	30 ft.	57 ft.
Gross vehicle weight	40,400 lbs.	70,000 lbs.
Street grades maximum	26% maximum	26% maximum
Approach and departure	15% maximum	15% maximum
Truck aerial operations	n/a	14% maximum

The Fire Department will determine, on a case-by-case review, where the truck aerial operations may not be required.

Department of Public Works 2015
Subdivision Regulation

C. STREET GUIDELINES

1. Alignment

All streets shall, as far as practicable, align with existing streets. The Subdivider shall justify any deviations based on written environmental and design objectives.

2. Intersecting Streets

Intersecting streets shall meet at right angles or as nearly so as practicable.

3. Naming

Streets of a proposed subdivision which are in alignment with existing streets shall bear the names of the existing streets. The Department of Public Works shall approve names for all new streets.

4. Street Grades

DPW shall not approve street grades in excess of 17% except as an exception and under unusual conditions.

Streets having grades in excess of 14% shall require separate consultation with the Fire Department prior to use for fire access purposes.

No gutter grade shall be less than 0.5%. The Subdivider shall provide concrete on any pavement grade less than 1.0%.

The Subdivider shall connect all changes in street grades, the algebraic sum of which exceeds 1.5%, with vertical curves of DPW-approved length sufficient to provide safe stopping sight distances and good riding quality. All changes in street grades shall have an absolute value of the algebraic difference in grades which does not exceed fifteen percent (15%), regardless of any vertical curves.

The Director with the consent of the SFFD may approve of any design modification to this standard on a case-by-case basis.

5. Surface Drainage

a. Subdivider shall grade streets to provide a continuous downhill path.

b. At low end cul-de-sacs and sumps, in addition to sewer drainage facilities, Subdivider shall provide surface drainage channels in dedicated easements as relief of overflow to prevent flooding of adjoining property.

c. Subdivider shall design street and drainage channel cross-sections to provide a transport channel for overland or surface flow in excess of the 5-years storm capacity of the sewer system. The channel capacity shall be the difference between the sewer capacity and the quantity of runoff generated by a 100-year storm as defined by the NOAA National Weather Service or by City-furnished data, applied over the tributary area involved.

d. Subdivider shall round street curb intersections by a curve generally having a radius equivalent to the width of the sidewalk and the design shall be in accordance with the Better Streets Plan. While allowing vehicle movements for emergency vehicles, the Subdivider shall use the smallest possible radius.

D. PRIVATE STREETS

Private streets shall have a minimum right-of-way width of 40 feet for through streets. Dead-end private streets shall have a minimum right-of-way width of 60 feet. The Subdivider shall consult with the Fire Department and Department of Building Inspection for all designs that might result in less than the minimum width.

E. BLOCKS

Technical Specifications Related to Engineering Document Section XII-B-3

DPW Disabilities Coordinator for specific provisions related to pavement materials, passenger loading zones, and path of travel for disabled persons.[27]

3. Fire Department Operations.

a. *All streets shall provide a minimum clear width of 20 feet of travel way between* obstructions. Obstructions may include parked vehicles, certain curbs greater than 6 inches in height[28] or any other fixed object that prevents emergency vehicular travel.

b. For purposes of calculating the clear width of the travel way, such width may include any combination of the following:

   i. That portion of any adjacent curbside parking space having a width greater than 7 feet,

   ii. a bike lane or any other adjacent pavement capable of supporting emergency vehicles where such lane or pavement is separated from the vehicular lanes by paint striping (Class II) or a mountable curb being no more than 2 inches in height (Class I), or other forms of pavement separation that may vary in material type, color, and texture.

c. Where adjacent buildings are greater than 40 feet in height and not of Type 1 (fire resistive) building construction, and the building entrance locations are not yet specified, the Director may require an operational width of at least 26 feet to accommodate Fire Department operational requirements along each street fronting such a building.

   i. "Operational width" shall be the combined total of the clear width of the travel way together with those unobstructed portions of adjacent pavement or sidewalks (if

[27] See also *Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way* as published by the United States Access Board.

[28] See San Francisco Fire Code Sec. 503.4, providing additional guidance on what may be considered an obstruction; see also Board of Supervisors Ordinance No. 116-13.

capable of supporting emergency vehicles).Reservation of portions of curbside parking for fire-only access or use of alternative mountable curb designs that allow for safe fire vehicle access to the sidewalk may accomplish this goal. The Fire Department, in consultation with other affected City agencies, may approve other proposals developed in the future.

ii. In such cases, the Subdivider shall provide sufficient right-of-way width on all abutting sides of a proposed development block to accommodate the foreseeable street design alternatives.

iii. Where DPW requires the portion of the block to have additional operational width (greater than 20 feet clear), the design engineer shall be locate this in segments along the building frontages with a maximum length of 200 feet for any one segment. Segments may have a minimum length of as little as 100 feet. The Subdivider shall ensure the existence of adequate space for emergency vehicles to pass each other and set up operations at the front entrance of the building. In addition, the design shall provide for meaningful traffic calming measures to ensure safe vehicle speeds along the street, including returning to the standard 20 foot travel way between widened segments. This provision shall not apply to blocks less than 200 feet in length.

iv. Subdividers are encouraged to consult with the Fire Department early in the subdivision process in advance of when the Subdivider anticipates the construction of such buildings. Information such as building access points, size of building and type of building construction are essential elements needed for constructive agency review.

v. Any decision to accommodate street widths having greater than 20 feet of travel way shall be approved by the Director only after consultation with and approval by an interagency working group composed of the Fire Department, the Municipal Transportation Agency, the Planning Department and any other affected city agency. When discussing the most appropriate widths of the travel way, the interagency working group shall consider such factors as the role and intended character of the street in the overall street network, the width of adjacent streets, the length of the street(s) in question, the anticipated traffic volume, and emergency and medical response.

4. Bicycle Lanes

All bicycle facilities shall meet or exceed the minimum lane widths provided in the *California Highway Design Manual*, the *California Manual on Uniform Traffic Control Devices.* Subdivider's shall design bicycle facilities in accordance with the *NACTO Urban Bikeway Design Guide.*

5. Parking Lane

The width of a curbside parallel parking lane shall be 8 feet. SFMTA may approve on a case by case basis angled curbside parking designs.

6. Curb Intersection Radii and Turning Movements

Subdividers shall design intersections for and accommodate turning vehicles in accordance with the Better Streets Plan. [29]

[29] http://www.sfbetterstreets.org/find-project-types/pedestrian-safety-and-traffic-calming/traffic-calming-overview/curb-radius-changes/

Exhibit 3

UNIVERSITY OF CALIFORNIA, BERKELEY

BERKELEY • DAVIS • IRVINE • LOS ANGELES • MERCED • RIVERSIDE • SAN DIEGO • SAN FRANCISCO   SANTA BARBARA • SANTA CRUZ

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CENTER FOR CATASTROPHIC RISK MANAGEMENT
DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING
BERKELEY, CALIFORNIA 94720-1710

June 29, 2016

Re: Inquiry about Gas Transmission Pipeline 109 from concerned SF residents
Proposed Project at 3516-3526 Folsom Street, San Francisco, CA

Dear Neighbors of Gas Transmission Pipeline 109:

Given the background information you have provided, yes, you should be concerned. There are several points in your summary that provide good basis for your concerns:

1) Old (1980's) PG&E gas transmission pipeline installed in area with highly variable topography,
2) Lack of records on the construction, operation, and maintenance of the pipeline,
3) No definitive guidelines to determine if the pipeline is 'safe' and reliable',
4) Apparent confusion about responsibilities (government, industrial-commercial) for the pipeline safety, reliability, and integrity.

This list is identical to the list of concerns that summarized causation of the San Bruno Line 132 gas pipeline disaster.

The fundamental 'challenge' associated with communicating your concern is tied to the word 'safe'. Unfortunately, it has been very rare that I have encountered organizations that have a good understanding of what that word means, and less of an understanding of how to demonstrate that a given system is 'safe enough.'

During my investigation of the San Bruno disaster, I did not find a single document (including trial deposition transcripts) that clearly indicated PG&E or the California PUC had a clear understanding of the word 'safe': "*freedom from undue exposure to injury and harm*." Further, it was clear they did not have a clear understanding of the First Minimal Principle of Civil Law: "*It is lawful to impose risks on people if and only if it is reasonable to assume that they have sufficient knowledge to understand the risks and have consented to accept those risks*."

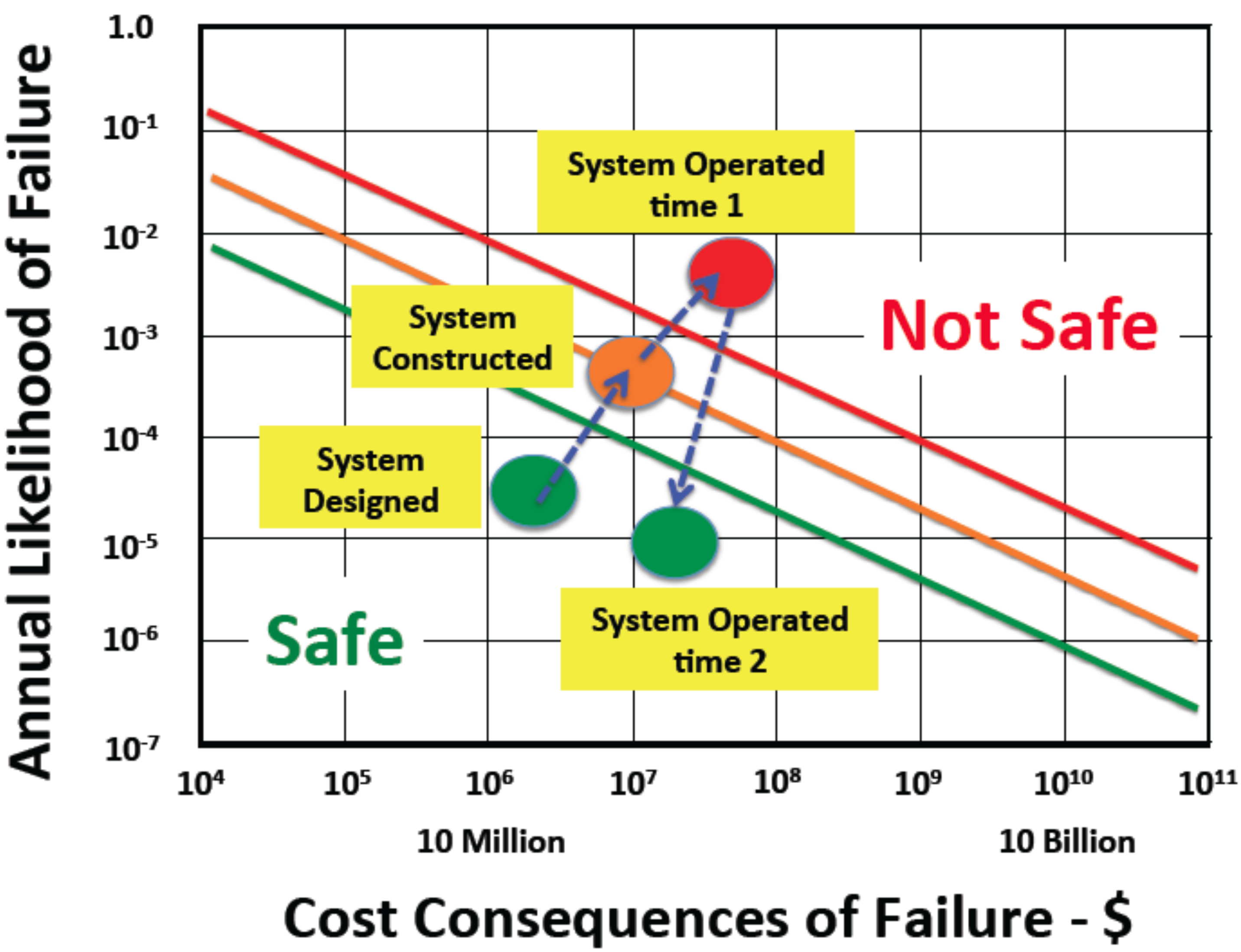
Much of this situation is founded in 'ignorance'. It is very rare for me to work with engineers or managers who have an accurate understanding of what the word 'safe' means - and no clue about how to determine if a system is either safe or unsafe. The vast majority of governmental regulatory agencies are even worse off.

I have attached a graph that helps me explain the important concepts associated with determining if a system is either safe or unsafe. The vertical scale is the annual likelihood of failure. The horizontal scale is the consequences associated with a failure. The diagonal lines separate the graph into two quadrants: Safe and Not Safe. If the potential consequences can be very high, then the probability of failure must be very low. Uncommon common sense.

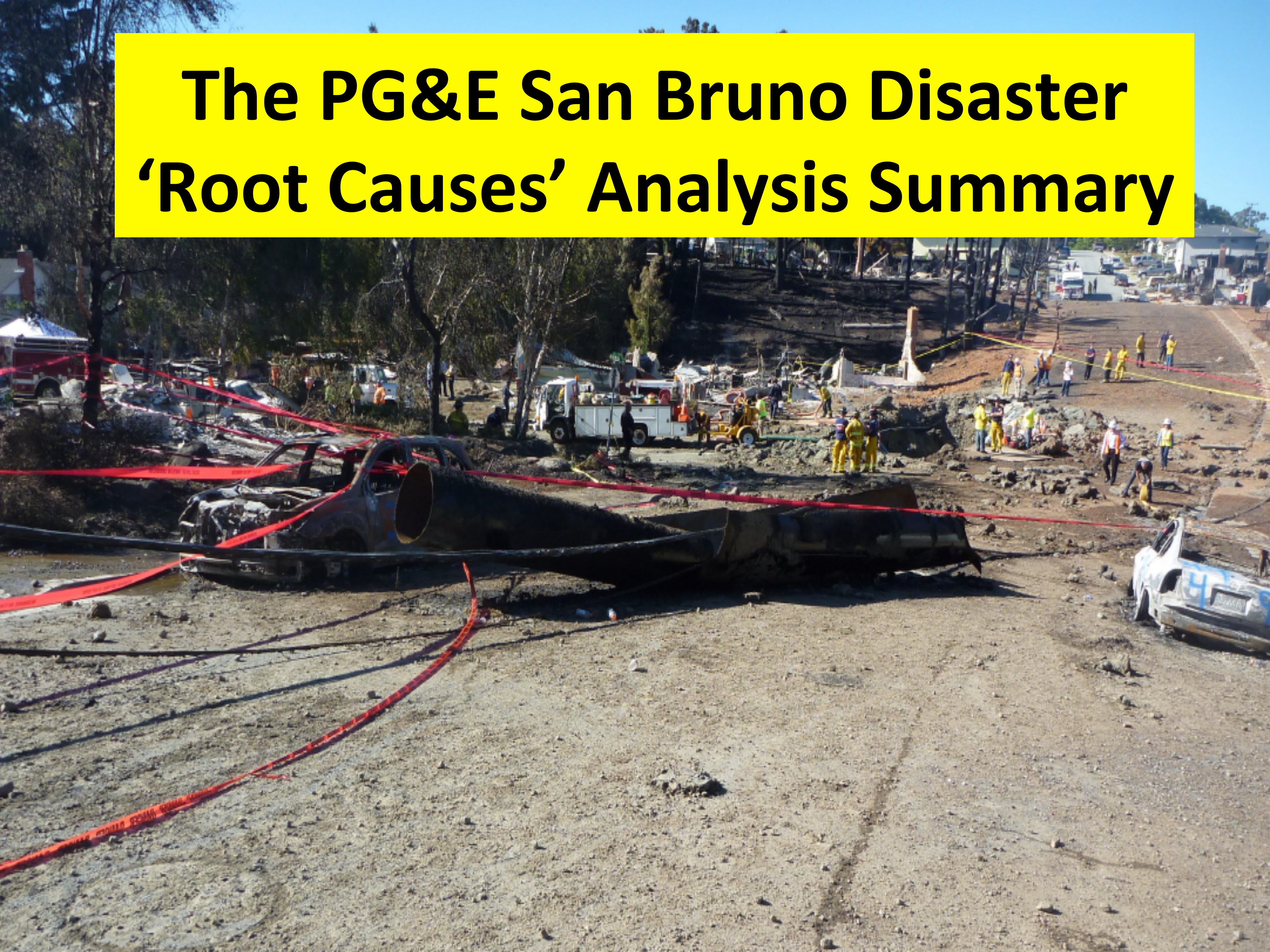
On the graph, I show a system that was designed for a particular 'risk' (combination of likelihood and consequences of failure). When it was constructed, the risk increased due to construction 'malfunctions' - like bad welding. When the system was put into service, the risk increased further - perhaps due to poor corrosion protection and due to the area around the pipeline being populated with homes, businesses, schools and other things that increase the potential consequences of a major failure. Once it is determined that the system that was originally designed to be safe is no longer safe, then it is necessary to do things that will allow the system to be safely operated—reduce the likelihood of failure (e.g. repair the corrosion) and reduce the consequences of failure (e.g. install pressure control shut off sensors and equipment that can detect a loss of gas and rapidly shut down the system)—or replace the segment of the pipeline that no longer meets safety-reliability requirements.

After I completed my investigation of the San Bruno disaster, I prepared a series of 'graphics' that summarized my findings. A copy of the file is attached. I hope it will help you understand how to better communicate your valid concerns regarding this development.

Robert Bea
Professor Emeritus
Center for Catastrophic Risk Management
University of California at Berkeley
email: bea@ce.berkeley.edu



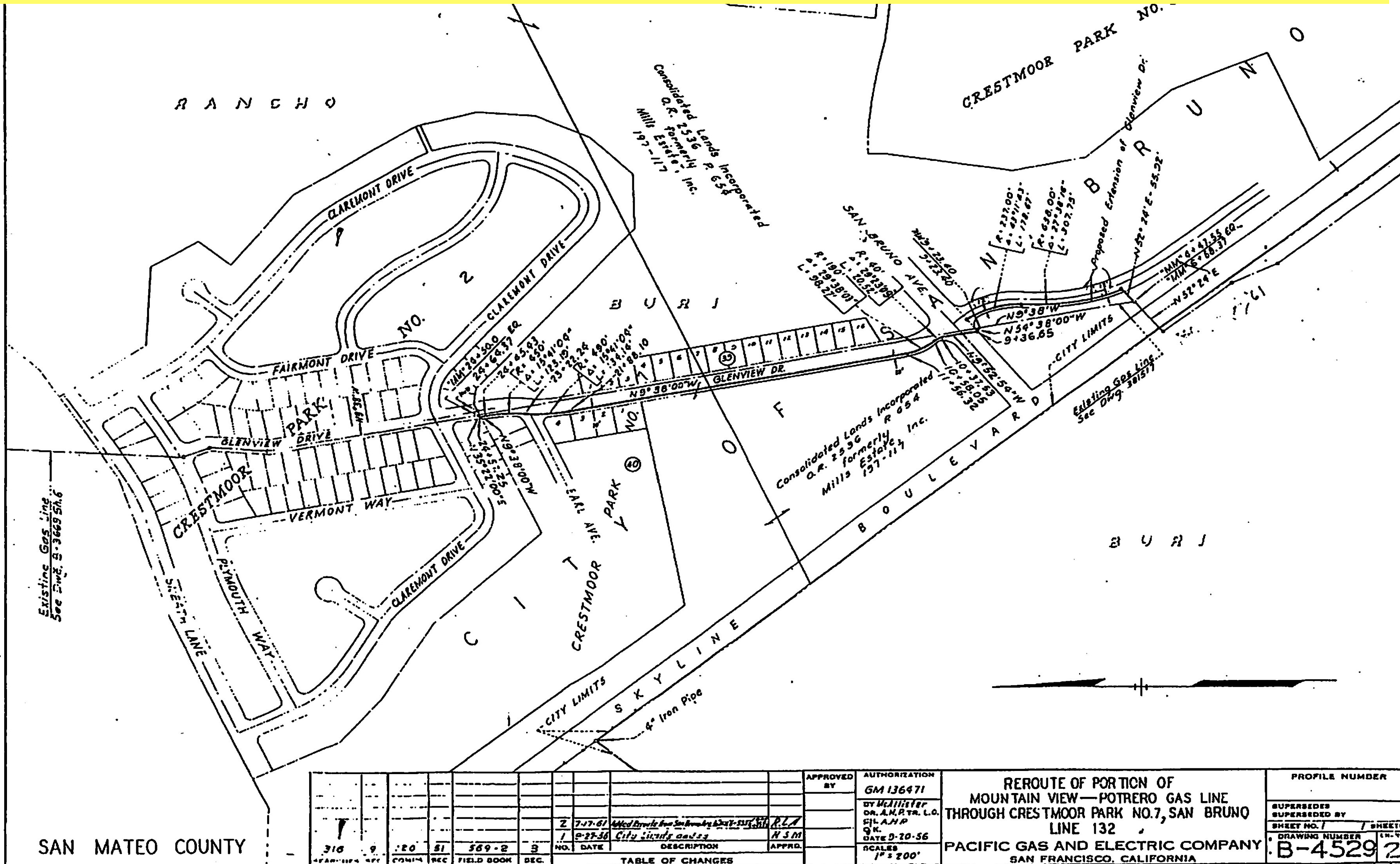
The PG&E San Bruno Disaster 'Root Causes' Analysis Summary



Crestmoor High Consequence Area

Installing Segment 180 in 1956

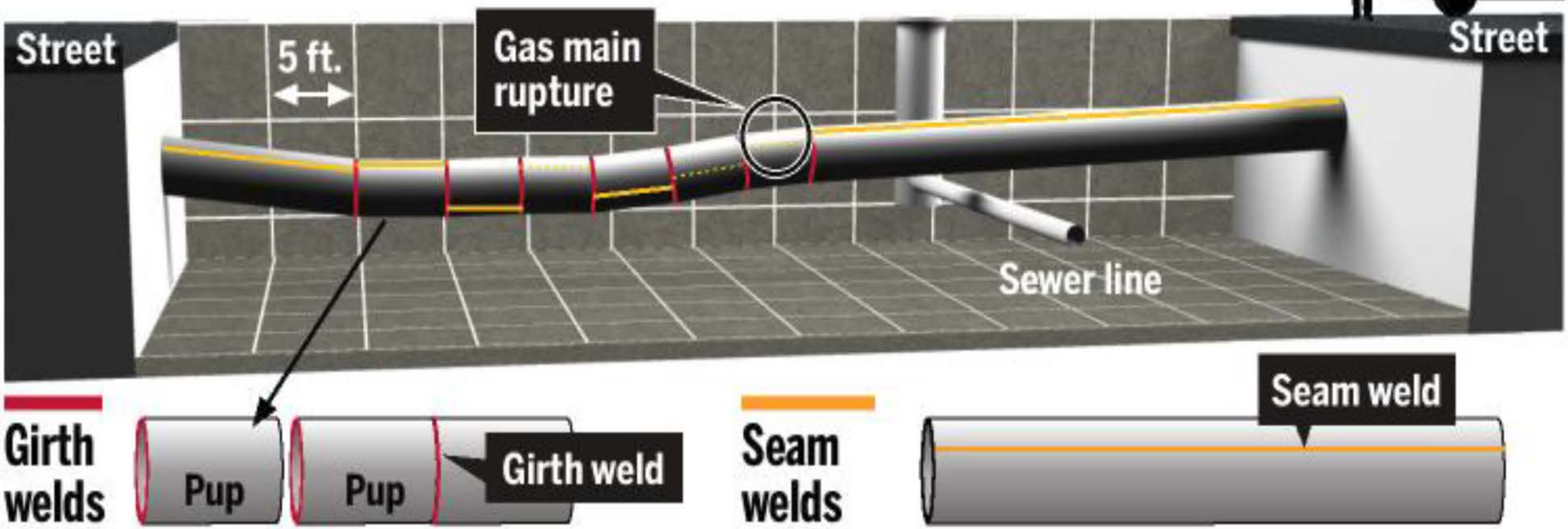
PG&E plans sent to field for 1956 relocation – details not provided for ravine profile



PG&E did not provide the construction 'details' to accommodate the change in vertical direction at the bottom of the 'ravine'

Gas pipeline construction

A report in January from the National Transportation Safety Board said that the natural gas pipeline that exploded in San Bruno in September 2010 had more than 100 spots with inadequate welds. These welds were either girth or seam welds, defined below.



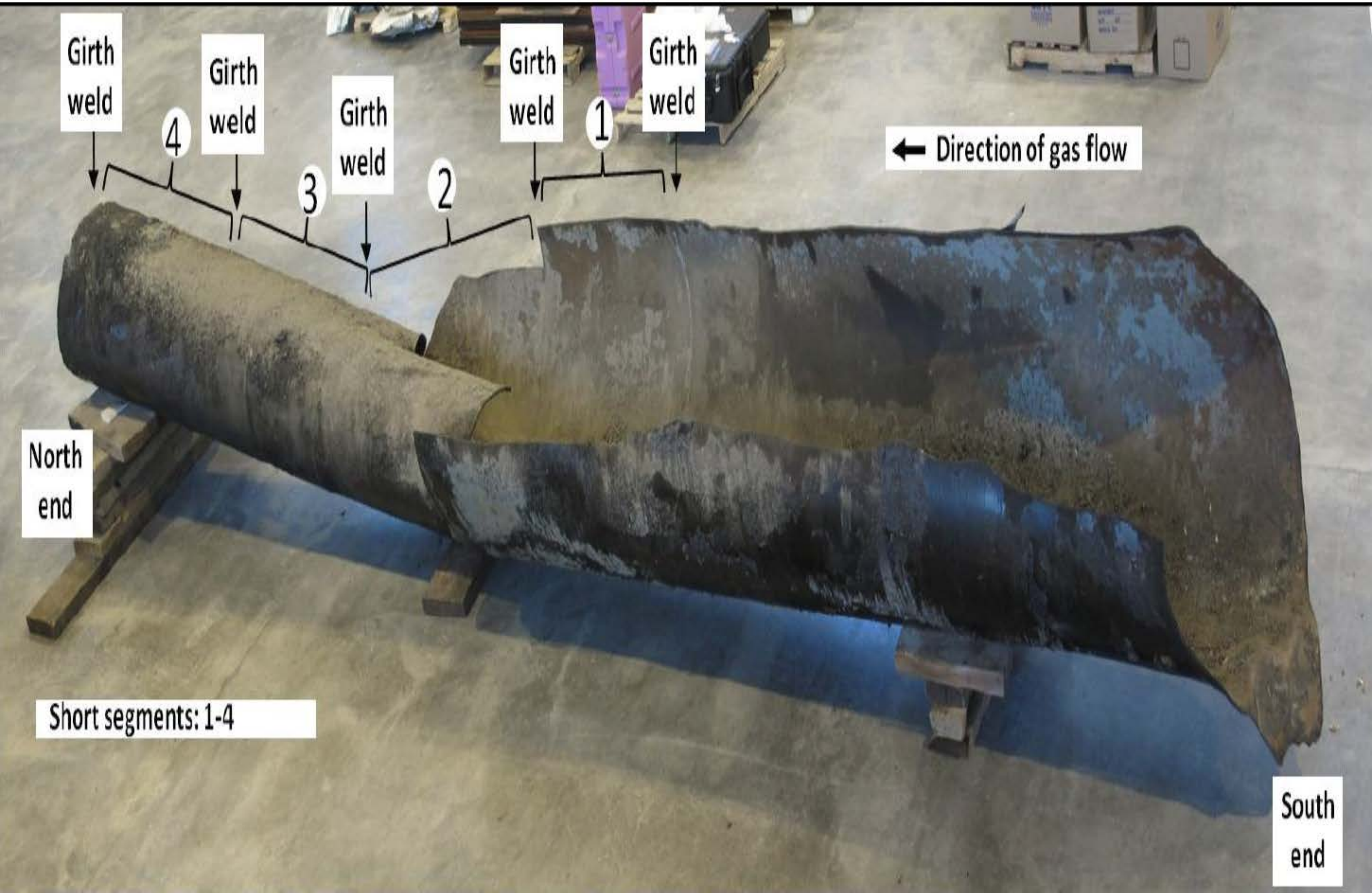
Within the 44-foot section of the damaged pipeline were six smaller pieces, known as "pups," all welded end-to-end at the girth on-site in 1956.

Done at a factory, pipes were made by rolling steel sheets and welding them at the seam. Investigators found numerous welds only penetrated halfway through the steel when they should have gone all the way.

Source: National Transportation Safety Board

PAI/MERCURY NEWS

PG&E installed a 'litter of pups' to accommodate the change in vertical direction at the bottom of the 'ravine'



Longitudinal welds inside pipe missing

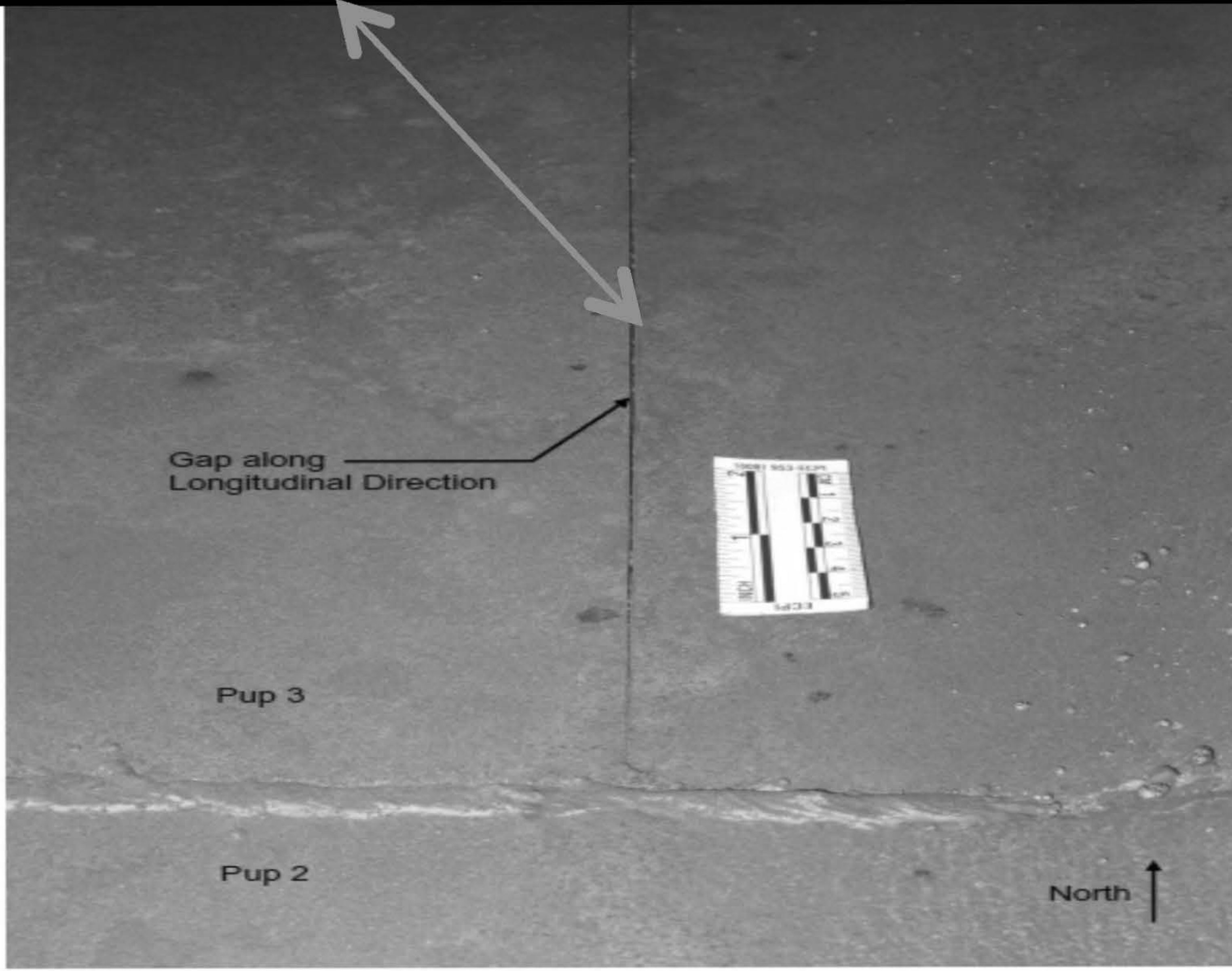


Figure 9: Inside wall of pup 3 showing a longitudinal gap that extended the length of the pup.

Welded from outside and ground flush

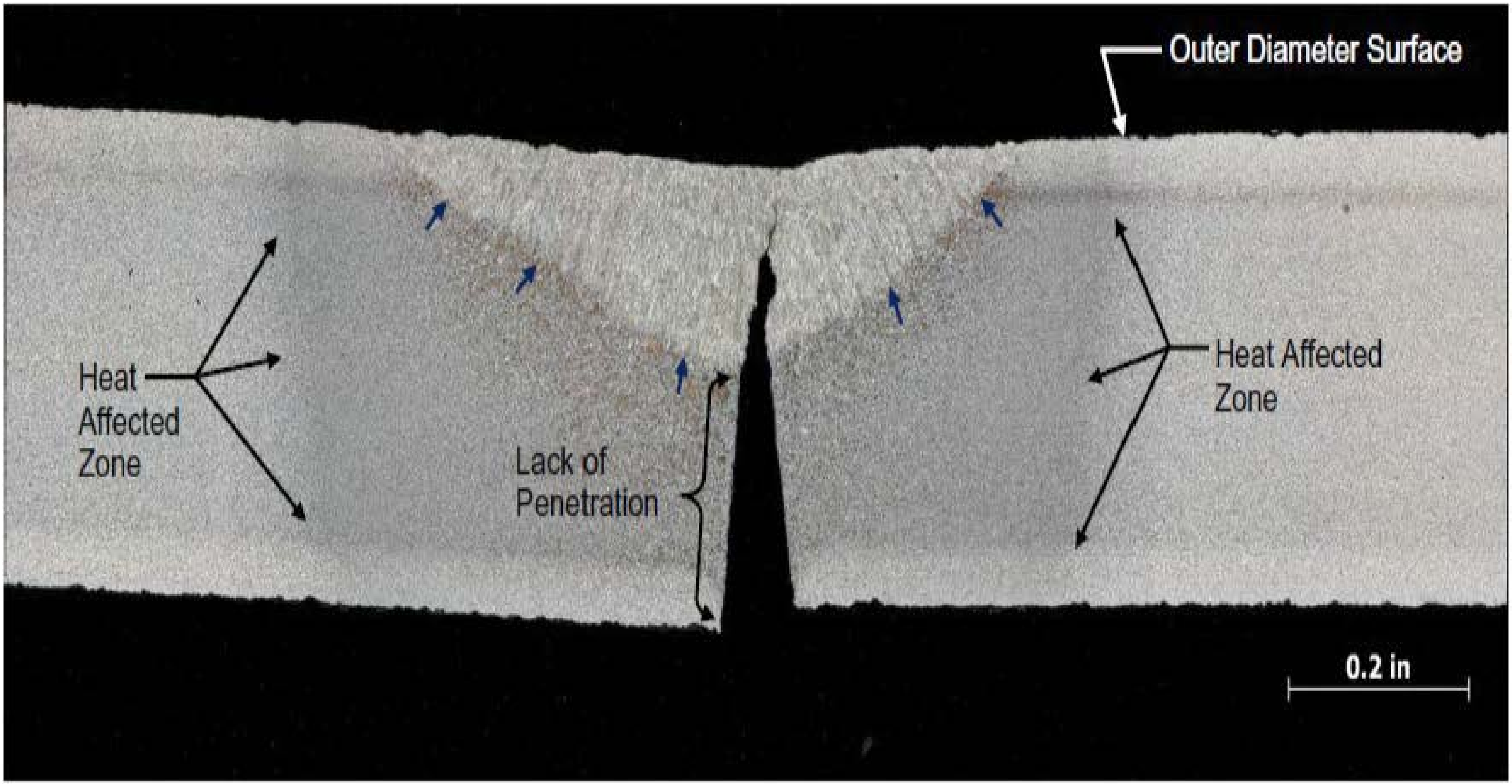


Figure 48: Etched metallographic cross section of the longitudinal seam in pup 3 taken 10 inch north of girth weld C3. The microstructure of the weld was consistent with a fusion welding process along the outer diameter surface of the seam. Blue arrows – weld pool boundary along outer diameter surface seam.

Weld flaws propagated by pressure fluctuations & 'spiking'

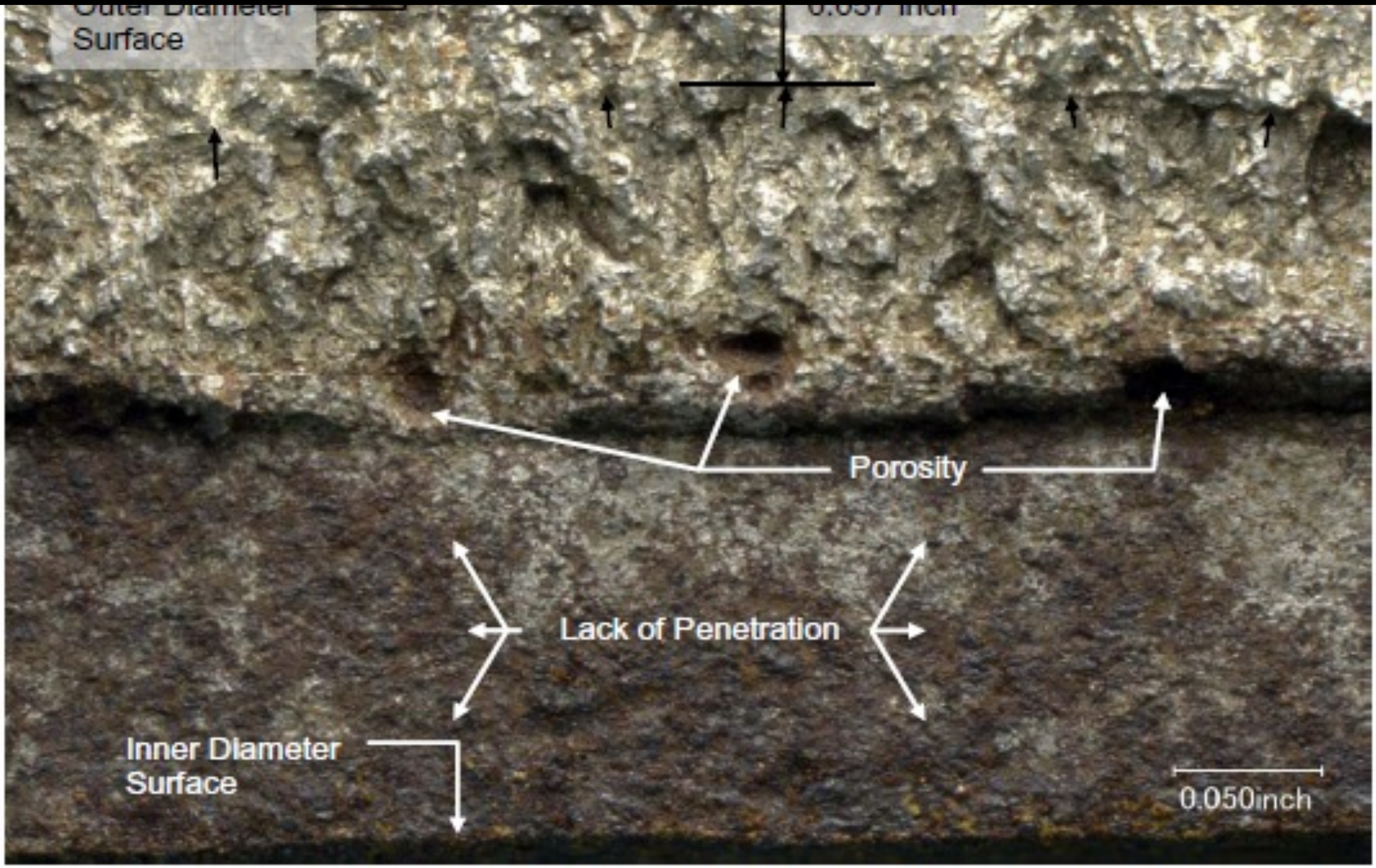


Figure 35: Micrograph of the initiation site in pup 1 at the 21.4 inch mark, the deepest point of the crack arrest mark. The profile of the arrest mark is indicated by the black arrows.


PG&E Milpitas control room operator:
“We’re Screwed!”
BRISTOL
VALVE 39
WARNING
V-37 SET POINT 385 PSIG
V-39 SET POINT 385 PSIG
REGULATOR OUTPUT
AUTOMATIC
MANUAL
INCREASE

The history of Line 132 Segment 180

'A Tyranny of Incremental Disastrous Decisions'

1956 construction 'work arounds' to relocate Line 132 and install Segment 180

1968 start intentional pressure 'Spiking' to maintain MAOP

1978 no action taken to hydrostatically test Line 132

1985 no action taken to replace Line 132 as part of the GPRP

1987 no action taken to uncover pipeline to determine what was 'in the ground'

The history of Line 132 Segment 180

'A Tyranny of Incremental Disastrous Decisions'

1988 no action taken to determine cause of leak in Line 132

1996 no actions taken to install RCVs or ASVs to reduce effects of rupture

1998 no actions taken to validate information contained in pipeline GIS

2000 replaced GPRP with Risk Management Program to reduce costs

2003 repeat intentional pressure 'Spiking' to maintain MAOP

2004 integrity survey discloses 13 leaks with 'unknown' causes

Line 132 Bunker Hill longitudinal weld leak

The history of Line 132 Segment 180
'A Tyranny of Incremental Disastrous Decisions'

2008 no actions taken to determine 'unknown' causes of 26 leaks in Line 132

2008 repeat intentional pressure 'Spiking' to maintain MAOP

2008 no inspection of Segment 180 uncovered for sewer replacement

2009 Enterprise Risk Management report recognizes pipeline explosion risks

2010 audit of PG&E's Integrity Management Program discloses dilution through exception process and insufficient allocation of resources

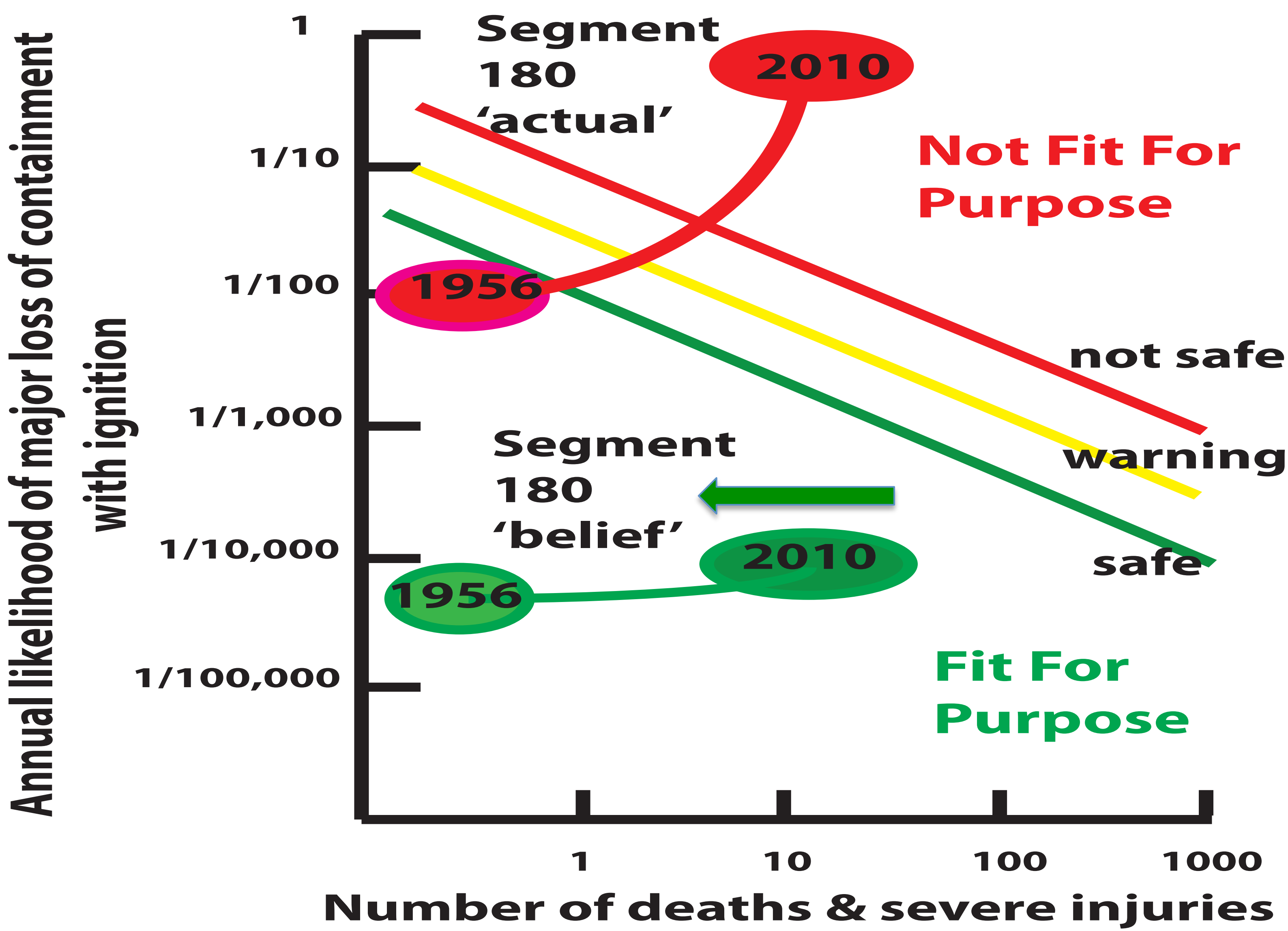
The history of Line 132 Segment 180

'A Tyranny of Incremental Disastrous Decisions'

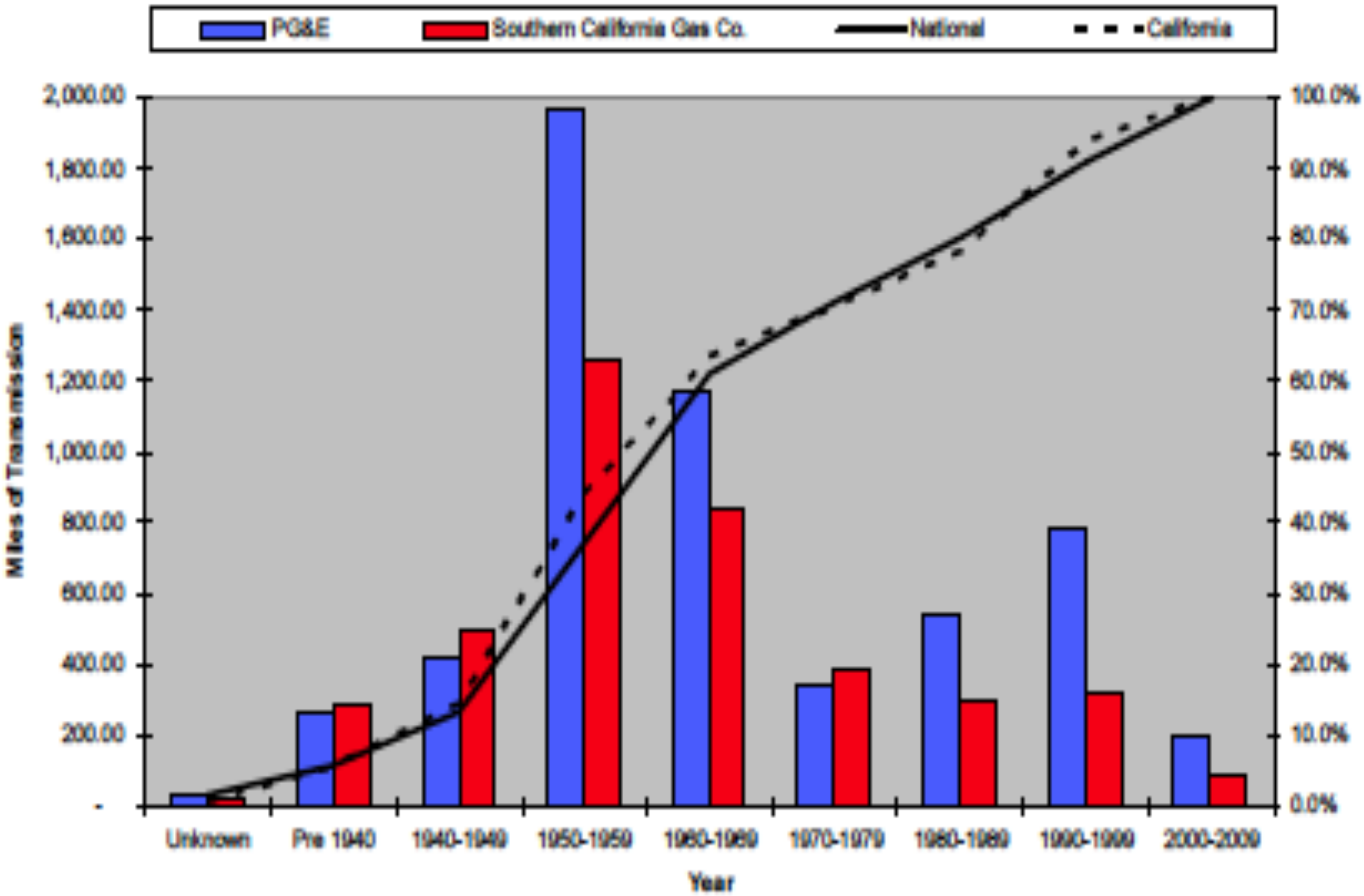
2010 additional manufacturing defect discovered in Line 132 girth weld

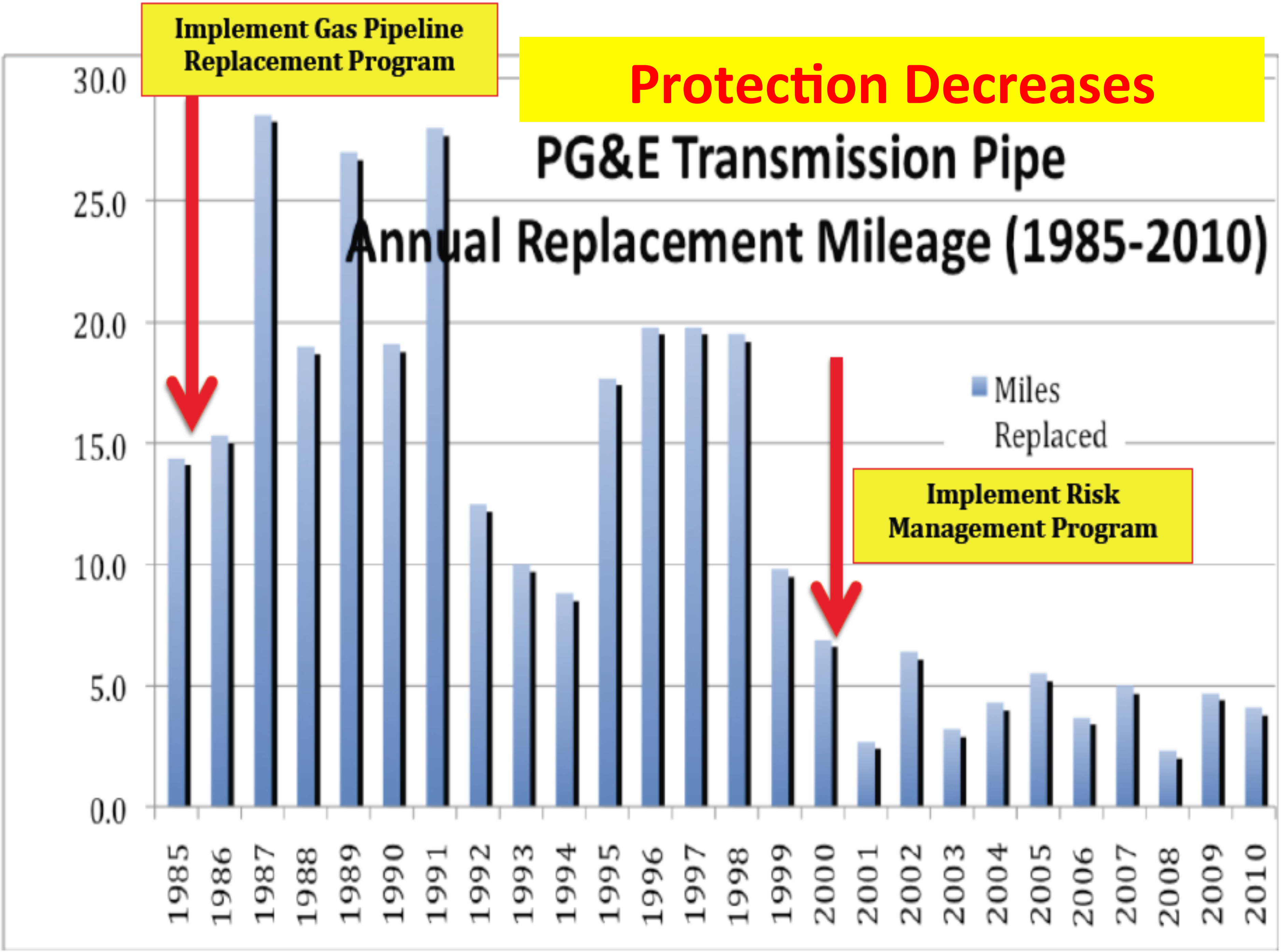
2010 September 9 at 6:11 PM Line 132 Segment 180 ruptures with catastrophic effects

PG&E Segment 180 Integrity Mis-management



Production Increases





Implement Gas Pipeline Replacement Program
Protection Decreases
PG&E Transmission Pipe
Annual Replacement Mileage (1985-2010)
Miles Replaced
Implement Risk Management Program
30.0
25.0
20.0
15.0
10.0
5.0
0.0
1985
1986
1987
1988
1989
1990
1991
1992
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2002
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2005
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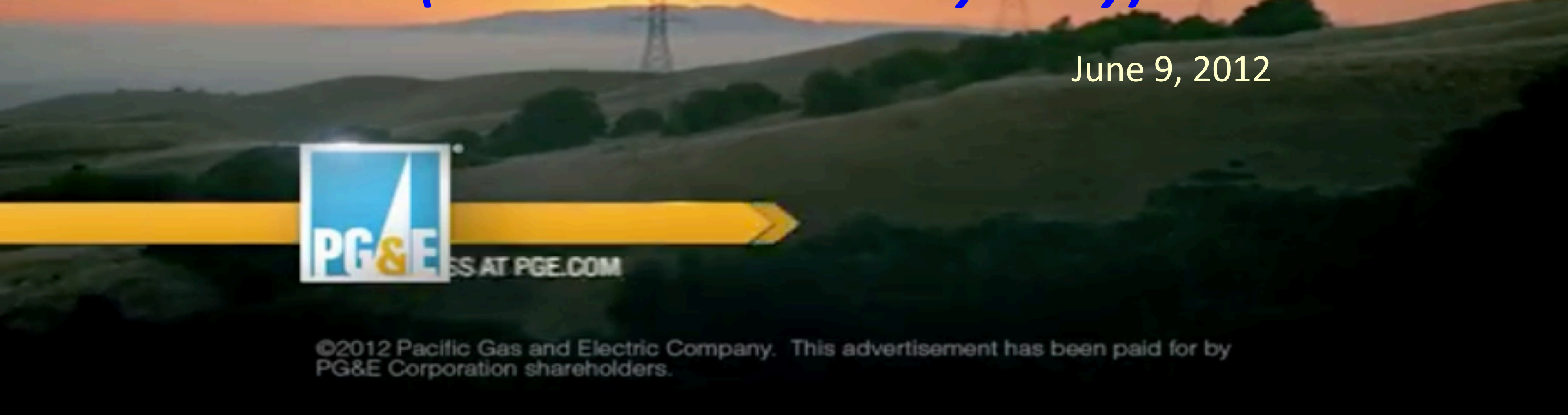
2007
2008
2009
2010

"I saw a company that lost its way"

(New PG&E CEO Tony Early)

June 9, 2012



Line 132 Segment 180
was
MANAGED TO FAILURE
by
PG&E

Exhibit 4

SIERRA CLUB
SAN FRANCISCO BAY

Serving Alameda, Contra Costa, Marin and San Francisco counties

SAN FRANCISCO GROUP

Please reply to 1474 Sacramento St., #305, San Francisco, CA 94109-4002

November 30, 2016

To Whom it May Concern:

SUPPORTING UPPER FOLSOM STREET CEQA APPEAL

The Sierra Club San Francisco Group supports the withdrawal or appeal of the categorical exemption for the Bernal Heights Upper Folsom Street Right-of-Way Housing Development (Planning Dept. Case No. 2013.1383ENV, hereinafter the "Project") and supports the preparation of an Environmental Impact Report for the Project.

The San Francisco Group speaks for the Sierra Club on city issues, on behalf of its 6,000 members and are one of the four chapters in the 4-county Bay Chapter's 30,000 members including Marin, Alameda, Contra Costa and San Francisco Counties. Our members, as well as the general public, will be directly affected by the Project's adverse environmental impacts on parkland, open space, and the Bernal Heights neighborhood.

The Upper Folsom Street Project received a Class 3 categorical exemption under CEQA Guidelines Section 15303(a). Pursuant to CEQA Guidelines Section 15300.2(c), however, a "categorical exemption shall not be used for an activity where there is a reasonable possibility that the activity will have a significant effect on the environment due to unusual circumstances." This proposed Project involves a number of unusual circumstances that will result in significant adverse environmental impacts.

*The exemption was granted to this proposed Project based on the fact that "the project site is not located in a particularly sensitive or hazardous area." Yet the proposed access to the Project will be built over a 26-inch 30-year-old gas transmission pipeline on a City right-of-way with an approximately 35 percent grade slope – including significant excavation. The Project site is adjacent to Bernal Heights Park and Bernal Heights Community Garden, in a densely populated area. City departments have stated they do not take responsibility for the safety of the pipeline, which is one of only three major gas lines in San Francisco. Despite federal recommendations, no informed assessment has taken place to assure local residents of the safety of this Project. This circumstance poses a risk of catastrophic environmental impacts, yet no environmental review has been completed.

SIERRA CLUB
SAN FRANCISCO BAY

Serving Alameda, Contra Costa, Marin and San Francisco counties

The U.S. Department of Transportation's Office of Pipeline Safety states that most gas transmission pipeline accidents occur on rights-of-way by private contractors – exactly the situation being proposed. A new, privately built access road over a major transmission pipeline –with the potential for multiple future adjacent private excavations on a steep slope – is unusual in San Francisco, if not unique. The proposed Project exposes a dense urban population to an unacceptable risk of environmental catastrophe, with no environmental review.

*CEQA Guidelines Section 15303 (2) can exempt construction of up to three single-family Residents. Guidelines Section 15300.2(b), however, prohibits the use of a categorical exemption where "the cumulative impact of successive projects of the same type in the same place, over time is significant." In this case, there are six undeveloped lots in the proposed Project area; the current Project proposes two 2,500 – 3,000 square foot homes including multi-car garages. If this Project is approved, it will set a precedent for the other four lots for further development in the near future.

*The proposed Project will have a number of additional impacts, including massing, loss of sunlight, and destruction of open space. It sets a precedent for large-scale houses in a neighborhood with traditionally smaller-scale housing and single car garages. The Project site is located within 300 feet of a possible urban bird refuge, within a steep slope district, and requires unusually extensive excavation. Moreover, as the categorical exemption determination notes, the Project site "is in an area that would be exposed to strong earthquake shaking." It notes that the Project's geotechnical reports recommend "seismic design parameters" to be used "during the Department of Building Inspection (DBI) building permit plan check process." It is inappropriate to suggest the use of mitigation measures in a categorical exemption, especially where those mitigation measures constitute undefined subsequent changes to the Project – precluding an "accurate, stable and finite project description." County of Inyo v. City of Los Angeles (1977) 71 Cal.App.3d 185, 193.

For these reasons, we request that the City withdraw the categorical exemption for Case No. 2013.1383ENV and complete an EIR for the proposed Project. Should the City fail to complete an EIR, the Sierra Club San Francisco Bay Chapter supports the appeal of the Project's flawed environmental determinations and opposes the issuance of Project permits, including BPA Nos. 2013.12.16.4322 and 2013.12.16.4318.

Becky Evans

Becky Evans

Vice Chair, San Francisco Group

Exhibit 5

BERNAL HEIGHTS DEMOCRATIC CLUB

Chartered since 1988 to give the residents of Bernal Heights an effective voice in government

---

April 20, 2016

To: SF PLANNING COMMISSION

RODNEY FONG, COMMISSION PRESIDENT
planning@rodneyfong.com

DENNIS RICHARDS, COMMISSION VICE-PRESIDENT
dennis.richards@sfgov.org

MICHAEL ANTONINI, COMMISSIONER
wordweaver21@aol.com

RICH HILLIS, COMMISSIONER
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JOHN RAHAIM, DIRECTOR OF PLANNING
John.Rahaim@sfgov.org

JONAS P. IONIN, COMMISSION SECRETARY
Commissions.Secretary@sfgov.org

DAVID CAMPOS, DISTRICT 9 SUPERVISOR
David.Campos@sfgov.org

FROM: Bernal Heights Democratic Club
bernalheightsdemclub@gmail.com

The Bernal Heights Democratic Club supports the opposition to the Upper Folsom Street Development in Bernal Heights, based on significant public safety concerns. There is clear danger from the major aging PG&E gas transmission pipeline; extreme steepness and narrow width of the proposed street; and unresolvable limited access to emergency vehicles.

It is our understanding that the two proposed lots now seeking permits will be followed by four more immediately adjacent. These types of construction will do nothing to address San Francisco's housing crisis, and are unsafe and inappropriate developments on these lots.

We appreciate your consideration of our input in this matter.

Exhibit 6

Bernal Heights Neighborhood Center

July 18, 2016

San Francisco Board of Supervisors
City Hall
San Francisco, CA 94102
Dear Honorable Members of the Board

Re: Appeal of CEQA Categorical Exemption ("CatEx") Determination for Planning Case No. 2013.1383E

We request a **complete, open, coordinated and transparent environmental impact review (EIR)** for the proposed project at **3516 and 3526 Folsom Street**.

We are concerned that the Bernal Heights neighborhood will be negatively impacted by this project based on our understanding that:

- It would threaten public safety as it is located adjacent to an aging 26-inch major gas transmission line 109. Heavy equipment would be traveling over this line in this very steep area during construction.
- It would negatively impact traffic safety as well as parking availability.
- It would be a "gateway" for four other adjacent sites, creating in essence a six unit "mini-sub-division". Such piecemeal planning is not in the best interests of San Francisco's neighborhoods.

It appears that the project developer has not heeded the concerns expressed by the East Slope Design Review Board, which was established by the San Francisco Planning Commission in 1986.

Thank you for your consideration. We ask that you **oppose this Categorical Exemption**.

Sincerely,

BHNC Board of Directors

Exhibit 7

From:	Samir Halteh <shalteh@gmail.com>
Sent:	Tuesday, March 29, 2016 7:05 PM
To:	Ryan Patterson
Cc:	Lupe Hernandez
Subject:	Folsom Street Extension
Follow Up Flag:	Flag for follow up
Flag Status:	Completed

Hi Ryan - please find my statement below. Hope this helps! -S

To Whom It May Concern:

My name is Samir Halteh and I have been a resident of the 300 block of Bradford Street, currently the steepest street in San Francisco) since September 2011.

In my relatively short period of time living on the block I've been witness to two separate car accidents as a result of the steep grade of the street. That does not even include others that other residents of the street have witnessed (including a few over-turned vehicles).

The first accident happened when a gentleman employed to repair a garage door on the block got stuck on the steeper portion of the street. He was unable to turn around because the street was too narrow and because of the high center of gravity of his vehicle. When he tried to get down in reverse, he ended up losing control of the vehicle and it crashed into two separate parked cars which then ricocheted it into two separate homes.

The second accident occurred when a taxi mistakenly navigated up the street. While attempting a three-point-turn, he drove up a curb which caused the vehicle to be lifted off the ground, suspended between the steepest part of the street with the part above it. He was unable to move since the car appeared to be in a position where it would flip over. We ended up having to call SFPD which later brought in SFFD as well as a tow truck to help get the car to safety.

On top of these incidents, there are countless people who navigate up the street looking for parking and end up getting stuck. I have watched countless times as they destroy our landscaping and privacy walls trying to get down.

Every call to a repairman or a delivery comes with a sense of dread (and good amount of forewarning) due to the grade of the street.

Replicating a street that is too narrow, steep, and without access from both sides is irresponsible, in my opinion. It strikes me as remarkably shortsighted to build homes with garage parking and street access in a location that so obviously cannot facilitate it safely. If the homes are to be built, I believe that the only solution is to give them access via staircase like those on Joy street.

Best,
Samir Halteh

354 Bradford Street
San Francisco, CA 94110

Ryan Patterson

From: Aaron W. <adwplanner@gmail.com>
Sent: Monday, April 25, 2016 5:49 PM
To: Ryan Patterson
Subject: Fwd: Upper Folsom Street Proposal - Folsom at Powhattan street

Here you go Ryan.

Sent from my portable telephone

Begin forwarded message:

From: "A-RON D.W." <adwplanner@gmail.com>
Date: March 30, 2016 at 4:48:36 PM PDT
To: richard.sucre@sfgov.org
Subject: Upper Folsom Street Proposal - Folsom at Powhattan street

Dear Mr. Sucre:

I am writing to express my concerns as a Bernal resident over the proposed street addition at upper Folsom street near Powhattan.

I reside on Bradford Street, the steepest hill in San Francisco. I believe the Folsom street addition will be of a similar slope. We have had issues with emergency vehicles not being able to navigate the hill. We have had cars where the emergency brake has snapped resulting in damage. We regularly have vehicles blocking passage in one direction or another. My father recently lost control of his balance and fell, breaking his leg. We have had people with belongings in shopping carts that have lost control of the carts, causing damage to vehicles.

I urge your committee to consider the potential hazards of inserting such a narrow and steep hill into the existing fabric of this location of Bernal.

Thank you.

Exhibit 8

WWW.BUSCOVICH.COM

Buscovich & Buscovich

235 MONTGOMERY STREET, SUITE 1140, SAN FRANCISCO, CALIFORNIA 94104 • TEL: (415) 760-0636 • E-Mail: patrick@buscovich.com

Board of Supervisor
City Hall
San Francisco, CA
Job Number: 14.145

July 8,2016

3516 Folsom
Rahul ShaI

Patrick Buscovich Civil Engineer

The following is a Civil Engineering Study and analysis of the proposed "Street": It is Current and unimproved dirt hill

- The Bureau of Street Use and Mapping (BSUM) have standards for street design and construction for the city to maintain a street after it is built. The current design is so out of conformance with city standards, the city will never accept this street for maintence. The street has varying slope from the intersection up the hill and the sidewalks are not level with each other. Warping of a street like this is not allowed. The fronting property owner will then have to maintain this street in perpetuity. In Addition, drainage down the street may flood the downhill homes
- This proposed street will be one of the steepest streets in San Francisco at +/- 36% slope. It will be 16 feet wide with no vehicle turn around at the top. It is a dead end street. Streets this steep are almost always thru streets or at a minimum have a turnaround. Without a turn around at the top, cars will back down the street in reverse. California vehicle code (CVC) discourages this manuever due to loosing control of a vehicle.
- Most vehicles, other than a specialized car, will not be able to drive onto this dead end street and into the houses. Most passenger cars will stop at the corner of Folsom & Chapman and park.
- It will be a challenge to turn around and change direction on this street in a vehicle, based upon the narrow width of 16 feet and extreme slope. Average cars length range from 15 feet to 18 feet long. It will be difficult to have an average car turn from uphill, to 90° to curb, to down hill. At 16 foot wide, an 18 foot car does not fit in the 90° position. Further, at 36° slope, vehicles with a medium to high center of mass will experience "tipping over" when turning around in the 90°

position. Thus any vehicle that are tall (i.e. mail truck, pick up, delivery van, garbage truck, etc) or have a long wheel base (sedan) will not be able to drive onto this dead end street. The only passenger car that could use this dead end street is low height, short wheel base, compact cars. Backing down the hill is not going to be a viable or safe solution. Ironically, the only vehicle that can turn around on this street (i.e. compact car) will not be able to transverse the base of the dead end street. The base is a flat intersection, a transition section and a steep hill (36°). Most cars will bottom out the tail pipe going uphill or the front fender going down. Even with a transition section of the street going from flat 0°, a short transition of 18° and then street 36° is not enough. No extension of car beyond the rear wheel or front wheel will work. To cross the intersection and go up/down this street will require a car with no front or rear end. This vehicle will also need to cross a very steep sidewalk and down a warp driveway; this will require a high undercarriage. A compact car with a high undercarriage and no front or rear end. The only vehicle that meets this description is a off road Jeep. It is short, has a low center of mass, high undercarriage clearance and no front or rear end. It is not a passenger vehicle. It is for off road driving which is what will be required to drive this hill. This vehicle is not meant for speed in excess of 50 MPH.

It is also important to note that garbage truck will not go up this street and Recology will not walk up the street to pick up recycling. Recycling bins will have to be left at the corner of Folsom and Chapman. With two homes now and two proposed with 4 more sites ready, the size of this garbage zone will be large. There is no sidewalk envision at the corner so no garbage zone is available. This is problem that needs to be addressed now in the street design for these homes to be livable.

Additionally, the mail truck will not go up this street. The mailman will have to hike up this street leaving his truck at the corner. This will potentially create a traffic issue at the intersection of Folsom and Chapman. I also hope that the project sponsor has talked to the US Postal Service to confirm they will hike the street to deliver the mail. Otherwise, a mail boxed will be required by the USPS at the intersection of Chapman and Folsom. There is no location I see that works for a mail box, let alone the recycle garbage bin zone.

The proposed two homes will need off street vehicle parking. Plausibly one vehicle could be a true off road Jeep, which could drive this street. The jeep will also be able to traverse the sidewalk cross slope. Most passenger vehicles can not traverse the extremely warped driveway. Exiting the garage and backing up the driveway will create a blind spot for the driver. At a minimum, a second car will be used at this house. Due to the steepness of Bernal I question the viability of a bike to replace a car but at a minimum, one addition car will be used for a house of this size. This second car is not going to be a jeep but a passenger car. This car will not be able to use the garage parking in the house but will use Street Parking. On this 16 foot section of Folsom St. there is no street parking. For planning purpose, six home time 1 car per home need to be accounted for neighborhood parking. For guest visits, more parking will be require. A simple study shows the need for 10 additional street parking spot in a neighbor with an acute shortage of on street parking. These "10 cars" not go up and down the street or across the sidewalk down the warped driveway. There is no street parking in front of these homes. These 10 cars are going to park in a 200 foot walking radius on the adjoining block of Folsom street, below the intersection or the adjoining block of Chapman. In this walking radius there are roughly 50 to 60 street parking spots that are almost always full. Adding 20% more parking is impossible. The garage in these homes will not work and a 16 foot wide Street with no street parking in front of homes will congest parking in this neighbor and will cause issues with Proposition Statement 2 "neighborhood character is conserved and protected". I am also concerned that this parking congestion issue will impede emergency vehicles (Police, Fire, and EMS).

Summary

In summary, the vehicle issue and parking demand will create a traffic mess for this neighborhood. This problem has simply not even been addressed by the project sponsor. It will be borne by the neighborhood. This problem is exacerbating by the size of the homes and number of bedroom proposed by the project by the project sponsor. This will be the steepest street with driveways in San Francisco, if not the State. In addition, the lack of thru or turn around will, in my professional opinion, create a significant traffic and parking problem.

Sincerely

Patrick Buscovich

REGISTERED PROFESSIONAL ENGINEER
PATRICK BUSCOVICH
S 2708
EXP. 6-30-
STRUCTURAL
STATE OF CALIFORNIA

WWW.BUSCOVICH.COM

Patrick Buscovich & Associates Structural Engineers, Inc.

235 MONTGOMERY STREET, SUITE 823, SAN FRANCISCO, CALIFORNIA 94104-3105 • TEL. (415) 788-2708 . FAX: (415) 788-0653

Patrick Buscovich S.E.

Education:

University of California, Berkeley – Bachelors of Science, Civil Engineering 1978
– Masters of Science, Structural Engineering 1979

Organizational:

State of California, Building Standards Commission
Commissioner 2000 – 2002
City & County of San Francisco, Department of Building Inspection (DBI)
Commissioner\Vice President 1995 – 1996
Chair, SF Housing Code Update 1995
UMB Appeals Board 2005 – 2006
Code Advisory Committee 1990 – 1992
Chair of Section 104 Sub-Committee.
Structural Engineers Association of Northern California (SEAONC)
President 1997 1998
Vice President 1996 – 1997
Board of Directors 1994 – 1999
College of Fellows Elected 2002
Edwin Zacher Award 1999
Structural Engineers Association of California (SEAOC)
Board of Directors 1996 – 2000
Applied Technology Council (ATC)
President 2007 – 2008
Board of Directors 2000 – 2009

Licenses:

California, Civil Engineer C32863, 1981
Structural Engineer S2708, 1985

Experience:

Patrick Buscovich and Associates, Structural Engineer – Senior Principal (1990 to Present)
Specializing in Existing Buildings, Seismic Strengthening/Structural Rehabilitation, Building Code/Permit Consultation, Peer Review, Expert Witness/Forensic Engineering

- Code Consulting and Peer Review for projects in San Francisco (Planning Department, Fire Preventing, Street Use & Mapping, Building Department, Board of Appeals).
- Permit Consultant in San Francisco (DBI, DCP, SFFD, BSUM & BOA).
- Expert Witness/Forensic Engineering/Collapse & Failure Analysis
- Seismic Retrofit Consultation.
- Member of the following SEAONC/DBI Committees:
  - Committee to revise San Francisco Building Code Section 104F/3304.6.
  - 1988-1990 Committee to draft San Francisco UMB ordinance.
  - 1993 Committee to revise the San Francisco UMB ordinance.
  - SEONC Blue-Ribbon panel to revise earthquake damage trigger, 1998
  - Secretary, Blue Ribbon Panel on seismic amendments to the 1998 SFBC.
  - Secretary, Blue Ribbon Panel Advising The San Francisco Building Department on CAPSS.
- Co-Authored of the following SF Building Code Sections.
  - EQ damage trigger SFBC 3404.7.2, Repair 3405.1.3, Change of Occupancy 3408.4.1., Lateral Forces Existing Building 1604 11 1
- Author SFBC Administrative Bulletin: AB102 (Seismic alteration) & AB103 (CFC)
- Coordinator/Speaker for SEAONC San Francisco UMB Seminars 1992, 1993 & 1994.
- Speaker at 2009 SEAONC Seminar on San Francisco UMB Code, 1850 to Present.
- Member of 1993 San Francisco UMB Bond Advisory Board.
- Speaker at numerous San Francisco Department of Building Inspection Seminars on UMB.
- Speaker at numerous code workshops for the San Francisco Department Building Inspection.
- Co-author of 1990 San Francisco UMB Appeals Board Legislation.
- Co-author of San Francisco Building Code Earthquake Damage Trigger for Seismic Upgrade, Committee Rewrite 2008.
- As a San Francisco Building Commissioner:
  - Directed formulation of Building Occupancy Resumption Plan (BORP)
  - Chaired the 1995 update on the San Francisco Housing Code.
  - Directed formulation of UMB tenant protection program
- Consultant to the City of San Francisco for evaluation of buildings damaged in the Loma Prieta Earthquake (October 17, 1989) to assist the Bureau of Building Inspection regarding shoring or demolition of "Red-Tagged" structures (SOHA).
- Consultant to San Francisco Department of Building Inspection on the Edgehill Land Slide 1997.
- Consultant to 100's of private clients for evaluating of damage to their buildings from the October 17, 1989 Loma Prieta Earthquake.
- Project Administrator for multi-team seismic investigation of San Francisco City-owned Buildings per Proposition A, 1989 ($350 million bond). (SOHA).
- Project Manager for seismic strengthening of the Marin Civic Center (SOHA).
- Structural Engineer for the Orpheum Theater, Curran Theater and Golden Gate Theater.
- Consultant on numerous downtown SF High Rise Buildings.
- Rehabilitation & Seismic Strengthening design for 1000's of commercial and residential buildings in San Francisco.
- Commercial Tenant Improvement
- Structure Rehabilitation of Historic Building.
- Structural consultant for 1000's of single family homes and apartment buildings alteration in San Francisco

Previous Employment

- SOHA 1980-1990, Associate
- PMB 1979-1980, Senior Designer

Public Service:

Association of Bay Area Government – Advisory Panels
Holy Family Day Home – Board of Director
Community Action Plan for Seismic Safety (CAPPS), Advisory Panel.

Awards:

Congressional Award, 2003.
SFDBI Certificate of Recognition, 1996.

Exhibit 9

Search

Example of incompatible vegetation planted within the ROW.

Tree Roots

Example of the impact tree roots can have on a pipeline.

se

Trees, large bushes and structures are not permitted within the pipeline right-of-way.

Keeping the right-of-way clear maintains the integrity of the pipeline and increases public safety.

TO REPORT A GAS LEAK OR OTHER EMERGENCY CALL:

QUESTAR GAS	1-800-767-1689
QUESTAR PIPELINE or QUESTAR OVERTHRUST	1-800-300-2025
QUESTAR SOUTHERN TRAILS PIPELINE	1-800-261-0668

QUESTAR®

59090 01/09

Trees and their potential to damage pipelines

Questar is an integrated natural gas company headquartered in Salt Lake City, Utah. Through subsidiaries Questar Gas, Questar Pipeline, Questar Southern Trails Pipeline, and Questar Overthrust Pipeline, the corporation owns and operates 29,000 miles of transmission and distribution pipelines in the western United States.

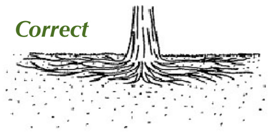
This brochure explains why planting deep-rooted vegetation, specifically trees, in Questar's pipeline rights-of-way is not permitted.

TREES & THEIR POTENTIAL TO DAMAGE PIPELINES

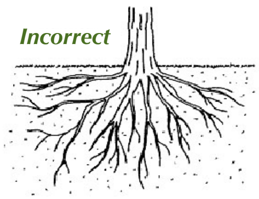
THE TRUTH ABOUT ROOTS

Tree roots are commonly depicted as a mirror image of the branches above. This is not an accurate picture of how tree roots actually grow. In fact, a tree's roots typically spread much further than its branches. Tree roots are also generally shallower than expected, with 90 percent of the roots contained in the first three feet of soil depth. However, if tree roots can get oxygen, they will reach deeper in search of water and nutrients. These deeper roots pose potential risks for pipeline safety.

Correct



Incorrect



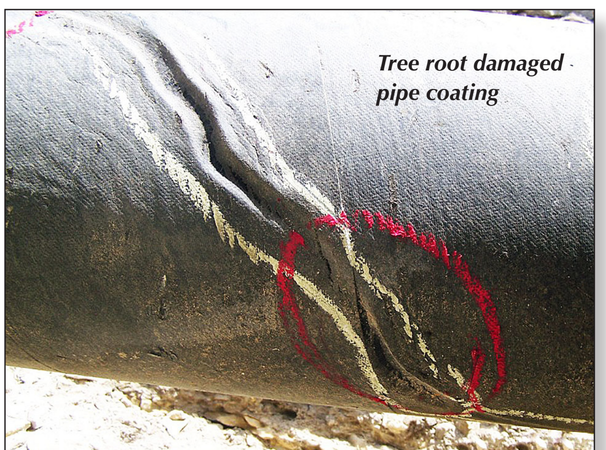
PIPELINE SAFETY VS. ROOTS

Trees planted too close to a pipeline can cause several potential safety-related problems. Roots follow the path of least resistance and grow easily in the less compact soils that typically surround a buried pipeline.

As roots continue to grow around the pipeline, they can damage the protective coating on the pipe. The protective coating helps to minimize corrosion on the pipeline.

As the trees and roots grow larger the risk to the pipeline increases.

If the tree is uprooted in a storm, it could rupture or severely damage the pipeline.



Tree root damaged pipe coating

Tree roots wrapped around the pipeline

These are the most common examples of how trees planted too near a pipeline can cause damage and leaks that put the community at risk and may possibly disrupt service to our customers.

MAINTENANCE AND EMERGENCY ACCESS

In order for maintenance or emergency response equipment to investigate or remedy a problem, trees, large bushes and shrubs, and structures including landscaping (e.g. rock walls) and fences that limit access to the pipeline or our rights-of-way must be removed. Obstacles like these can increase the time it takes to access the pipeline if there's a problem and may make the situation more dangerous.

LANDSCAPING

Deep-rooted plants and trees, and retaining walls, are not permitted within the right-of-way. Grasses, low-growing plants and shrubs, and gardens may be planted within the right-of-way. If landscaping is disturbed during Questar's maintenance activities, the property owner is responsible for restoration.

CALL BEFORE YOU DIG

Before doing any digging or excavating, always dial 811 at least 48 hours ahead of time. Someone will come and mark the location of buried pipelines and other utilities for no charge.

811 Know what's **below.** **Call** before you dig.

ADDITIONAL INFORMATION

For additional information about Questar's operations or facility locations, visit www.questar.com or contact:

Questar Gas Call Center 801-324-5111

Questar Pipeline Co. Operations Center 307-382-8882

Questar Southern Trails Pipeline 307-382-8882

180 East 100 South
P.O. Box 45360
Salt Lake City, UT 84145-0360

For information about Questar's Public Awareness Programs contact: Questar Corporate Communication Department at 801-324-5548

QUESTAR®

Exhibit 10

SPECIAL PUBLICATION 117

GUIDELINES FOR EVALUATING AND MITIGATING SEISMIC HAZARDS IN CALIFORNIA

Adopted March 13, 1997 by the State Mining and Geology Board in Accordance with the Seismic Hazards Mapping Act of 1990

Copies of these Guidelines, California's Seismic Hazards Mapping Act, and other related information are available on the World Wide Web at
Copies also are available for purchase from the Public Information Offices of the California Geological Survey.

CALIFORNIA GEOLOGICAL SURVEY'S PUBLIC INFORMATION OFFICES:

Southern California Regional Office
655 South Hope Street, Suite 700
Los Angeles, CA 90017-3231
(213) 239-0878

Publications and Information Office
801 K Street, MS 14-33
Sacramento, CA 95814-3532
(916) 445-5716

Bay Area Regional Office
185 Berry Street, Suite 210
San Francisco, CA 94107-1728
(415) 904-7707

Contents

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CHAPTER 1

INTRODUCTION

Prompted by damaging earthquakes in northern and southern California, in 1990 the State Legislature passed the Seismic Hazards Mapping Act. The Governor signed the Act, codified in the Public Resources Code as Division 2, Chapter 7.8 (see Appendix A), which became operative on April 1, 1991.

The purpose of the Act is to protect public safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure, and other hazards caused by earthquakes. The program and actions mandated by the Seismic Hazards Mapping Act closely resemble those of the Alquist-Priolo Earthquake Fault Zoning Act (which addresses only surface fault-rupture hazards) and are outlined below:

1. **The State Geologist** is required to delineate the various "seismic hazard zones."

2. **Cities and Counties**, or other local permitting authority, must regulate certain development "projects" within the zones. They must withhold the development permits for a site within a zone until the geologic and soil conditions of the project site are investigated and appropriate mitigation measures, if any, are incorporated into development plans.

3. **The State Mining and Geology Board** provides additional regulations, policies, and criteria, to guide cities and counties in their implementation of the law (see Appendix B). The Board also provides guidelines for preparation of the Seismic Hazard Zone Maps (available at http://www.consrv.ca.gov/dmg/shezp/zoneguid.html) and for evaluating and mitigating seismic hazards (this document).

4. **Sellers (and their agents)** of real property within a mapped hazard zone must disclose that the property lies within such a zone at the time of sale.

This document constitutes the guidelines for evaluating seismic hazards other than surface fault-rupture, and for recommending mitigation measures as required by Public Resources Code Section 2695(a). Nothing in these Guidelines is intended to conflict with or supersede any requirement, definition, or other provision of Chapter 7.8 of the Public Resources Code; California Code of Regulations, Title 14, Division 2, Chapter 8, Article 10; the Business and Professions Code; or any other state law or regulation.

Objectives

The objectives of these Guidelines are twofold:

1. To assist in the evaluation and mitigation of earthquake-related hazards for projects within designated zones of required investigations; and

2. To promote uniform and effective statewide implementation of the evaluation and mitigation elements of the Seismic Hazards Mapping Act.

The Guidelines will be helpful to the owner/developer seeking approval of specific development projects within zones of required investigation and to the engineering geologist and/or civil engineer who must investigate the site and recommend mitigation of identified hazards. They will also be helpful to the lead agency engineering geologist and/or civil engineer who must complete the technical review, and other lead agency officials involved in the planning and development approval process. Effective evaluation and mitigation ultimately depends on the combined professional judgment and expertise of the evaluating and reviewing professionals.

The methods, procedures, and references contained herein are those that the State Mining and Geology Board, the Seismic Hazards Mapping Act Advisory Committee, and its Working Groups believe are currently representative of quality practice. Seismic hazard assessment and mitigation is a rapidly evolving field and it is recognized that additional approaches and methods will be developed. If other methods are used, they should be justified with appropriate data and documentation.

For a general description of the Department's Seismic Hazards Mapping Program, its products and their uses, refer to the User's Guide (available in draft form on the World-Wide Web at http://www.consrv.ca.gov/dmg/shezp/userguid.html). A hard-copy edition of the User's Guide will be available later in 1997.

CHAPTER 2

Definitions, Caveats, and General Considerations

Definitions

Key terms that will be used throughout the Guidelines are defined in the Act and related regulations. These are:

- **"Seismic Hazards Mapping Act"**— California Public Resources Code Sections 2690 and following, included as Appendix A.

- **"Seismic Hazards Mapping Regulations"**— California Code of Regulations (CCR), Title 14, Division 2, Chapter 8, Article 10, included as Appendix B.

- **"Owner/Developer"** is defined as the party seeking permits to undertake a "project", as defined below.

- **"Mitigation"** means those measures that are consistent with established practice and reduce seismic risk to "acceptable levels" [Public Resources Code (PRC) Section 2693(c)].

- **"Acceptable level"** of risk means that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project [CCR Title 14, Section 3721(a)].

- **"Lead agency"** means the state agency, city, or county with the authority to approve projects [CCR Title 14, Section 3721(b)].

- **"Certified Engineering Geologist"** means an engineering geologist who is certified in the State of California [CCR Title 14, Section 3721(c); Business and Professions Code (BPC) Sections 7804 and 7822] and practicing in his or her area of expertise. These professionals will be referred to throughout these Guidelines as "engineering geologists." See page 8 (*Engineers or Geologists— Who Does What?*) for a discussion of scope of involvement in site-investigation reports and related reviews.

- **"Registered Civil Engineer"** means a civil engineer who is registered in the State of California [CCR Title 14, Section 3721(c); BPC Sections 6701-6704] and practicing in his or her area of expertise. These professionals will be referred to throughout these Guidelines as "civil engineers." See page 8 (*Engineers or Geologists—Who Does What?*) for a discussion of scope of involvement in site-investigation reports and related reviews.

- **"Site-Investigation Report"** means a report prepared by a certified engineering geologist and/or a civil engineer practicing within the area of his or her competence, which documents the results of an investigation of the site for seismic hazards and recommends mitigation measures to reduce the risk of identified seismic hazards to acceptable levels. In PRC Section 2693(b) and elsewhere, this report is referred to as a "geotechnical report."

- The term **"Project"** is defined by the Seismic Hazards Mapping Act as any structures for human occupancy, or any subdivision of land that contemplates the eventual construction of structures for human occupancy. Unless lead agencies impose more stringent requirements, single-family frame dwellings are exempt unless part of a development of four or more dwellings. (The definition is complex; see Table 1 for specific language.)

- **"Seismic Hazard Zone Maps"** are maps issued by the State Geologist under PRC Section 2696 that show zones of required investigation.

- **"Seismic Hazard Evaluation Reports"** document the data and methods used by the State Geologist to develop the "**Seismic Hazard Zone Maps**."

- **"Zones of Required Investigation,"** referred to as "**Seismic Hazard Zones**" in CCR Section 3722, are areas shown on Seismic Hazard Zone Maps where site investigations are required to determine the need for mitigation of potential liquefaction and/or earthquake-induced landslide ground displacements.

Definitions of technical terms appear in Appendix C.

Caveats

Minimum Statewide Safety Standard

Based on the above definitions of "mitigation" and "acceptable risk," the Seismic Hazards Mapping Act and related regulations establish a statewide minimum public safety standard for mitigation of earthquake hazards. This means that the minimum level of mitigation for a project should reduce the risk of ground failure during an earthquake to a level that does not cause the collapse of buildings for human occupancy, but in most cases, **not** to a level of no ground failure at all. However, nothing in the Act, the regulations, or these Guidelines precludes lead agencies from enacting more stringent requirements, requiring a higher level of performance, or applying these requirements to developments other than those that meet the Act's definition of "project."

Areal Extent of Hazard

The Seismic Hazard Zone Maps are developed using a combination of historic records, field observations, and computer-mapping technology. The maps may not identify all areas that have potential for liquefaction, earthquake-induced landsliding, strong ground shaking, and other earthquake and geologic hazards. Although past earthquakes have caused ground failures in only a small percentage of the total area zoned, a worst-case scenario of a major earthquake during or shortly after a period of heavy rainfall is something that has not occurred in northern California

TABLE 1. Definition of "Project"

Public Resources Code Section 2693.

As used in [Chapter 7.8, the Seismic Hazards Mapping Act]:

(d) "Project" has the same meaning as in Chapter 7.5 (commencing with Section 2621), except as follows:

(1) A single-family dwelling otherwise qualifying as a project may be exempted by the city or county having jurisdiction of the project.

(2) "Project" does not include alterations or additions to any structure within a seismic hazard zone which do not exceed either 50 percent of the value of the structure or 50 percent of the existing floor area of the structure.

Public Resources Code Section 2621.6.

(a) As used in (Chapter 7.5, the Alquist-Priolo Earthquake Fault Zoning Hazard Act), "project" means either of the following:

(1) Any subdivision of land which is subject to the Subdivision Map Act (Division 2 (commencing with Section 66410) of Title 7 of the Government Code), and which contemplates the eventual construction of structures for human occupancy.

(2) Structures for human occupancy, with the exception of either of the following:

(A) Single-family wood-frame or steel-frame dwellings to be built on parcels of land for which geologic reports have been approved pursuant to paragraph (1).

(B) A single-family wood-frame or steel-frame dwelling not exceeding two stories when that dwelling is not part of a development of four or more dwellings.

(b) For the purposes of this chapter, a mobile home whose body width exceeds eight feet shall be considered to be a single-family wood-frame dwelling not exceeding two stories.

California Code of Regulations Section 3601 (Policies and Criteria of the State Mining and Geology Board, With Reference to the Alquist-Priolo Earthquake Fault Zoning Act).

The following definitions as used within the Act and herein shall apply:

(e) A "structure for human occupancy" is any structure used or intended for supporting or sheltering any use of occupancy, which is expected to have a human occupancy rate of more than 2,000 person-hours per year.

(f) Story" is that portion of a building included between the upper surface of any floor and the upper surface of the floor next above, except that the topmost story shall be that portion of the building included between the upper surface of the topmost floor and the ceiling or roof above. For the purpose of the Act and this subchapter, the number of stories in a building is equal to the number of distinct floor levels, provided that any levels that differ from each other by less than two feet shall be considered as one distinct level."

since 1906, and has not been witnessed in historic times in southern California. The damage from such an event in a heavily populated area is likely to be more widespread than that experienced in the 1971 San Fernando earthquake, the 1989 Loma Prieta earthquake, or the 1994 Northridge earthquake.

Off-Site Origin of Hazard

The fact that a site lies outside a zone of required investigation does not necessarily mean that the site is free from seismic or other geologic hazards, regardless of the information shown on the Seismic Hazard Zone Maps. The zones do not always include landslide or lateral spread run-out areas. Project sites that are outside of any zone may be affected by ground failure runout from adjacent or nearby sites.

Finally, neither the information on the Seismic Hazard Zone Maps, nor in any technical reports that describe how the maps were prepared nor what data were used, is sufficient to serve as a substitute for the required site-investigation reports called for in the Act.

Relationship of these Guidelines to Local General Plans and Permitting Ordinances

Public Resources Code Section 2699 directs cities and counties to "take into account the information provided in available seismic hazard maps" when it adopts or revises the safety element of the general plan and any land-use planning or permitting ordinances. Cities and counties should consider the information presented in these guidelines when adopting or revising these plans and ordinances.

Relationship of these Guidelines to the CEQA Process and Other Site Investigation Requirements

Nothing in these guidelines is intended to negate, supersede, or duplicate any requirements of the California Environmental Quality Act (CEQA) or other state laws and regulations. At the discretion of the lead agency, some or all of the investigations required by the Seismic Hazards Mapping Act may occur either before, concurrent with, or after the CEQA process or other processes that require site investigations.

Some of the potential mitigation measures described herein (e.g., strengthening of foundations) will have little or no adverse impact on the environment. However, other mitigation measures (e.g., draining of subsurface water, driving of piles, densification, extensive grading, or removal of liquefiable material) may have significant impacts. If the CEQA process is completed prior to the site-specific investigation, it may be desirable to discuss a broad range of potential mitigation measures (any that might be proposed as part of the project) and related impacts. If, however, part or all of the site-specific investigation is conducted prior to completion of the CEQA process, it may be possible to narrow the discussion of mitigation alternatives to only those that would provide reasonable protection of the public safety given site-specific conditions.

For hospitals, public schools, and essential service buildings, more stringent requirements are prescribed by the California Building Code (CCR Title 24). For such structures, the requirements of the Seismic Hazards Mapping Act are intended to complement the CCR Title 24 requirements.

Criteria for Project Approval

The State's minimum criteria required for project approval within zones of required investigation are defined in CCR Title 14, Section 3724, from which the following has been excerpted:

"The following specific criteria for project approval shall apply within seismic hazard zones and shall be used by affected lead agencies in complying with the provisions of the Act:

(a) A project shall be approved only when the nature and severity of the seismic hazards at the site have been evaluated in a geotechnical report and appropriate mitigation measures have been proposed.

(b) The geotechnical report shall be prepared by a registered civil engineer or certified engineering geologist, having competence in the field of seismic hazard evaluation and mitigation. The geotechnical report shall contain site-specific evaluations of the seismic hazard affecting the project, and shall identify portions of the project site containing seismic hazards. The report shall also identify any known off-site seismic hazards that could adversely affect the site in the event of an earthquake. The contents of the geotechnical report shall include, but shall not be limited to, the following:

(1) Project description.

(2) A description of the geologic and geotechnical conditions at the site, including an appropriate site location map.

(3) Evaluation of site-specific seismic hazards based on geological and geotechnical conditions, in accordance with current standards of practice.

(4) Recommendations for appropriate mitigation measures as required in Section 3724(a), above.

(5) Name of report preparer(s), and signature(s) of a certified engineering geologist and/or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation.

(c) Prior to approving the project, the lead agency shall independently review the geotechnical report to determine the adequacy of the hazard evaluation and proposed mitigation measures and to determine the requirements of Section 3724(a), above, are satisfied. Such reviews shall be conducted by a certified engineering geologist or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation."

Lead agencies can have other, more stringent criteria for project approval. The State Mining and Geology Board recommends that the official professional Registration or Certification Number and license expiration date of each report preparer be included in the signature block of the report. In addition, Chapter 3 provides a list of topics that should be addressed in site-investigation reports prepared for liquefaction and/or earthquake-induced landslides.

Engineers or Geologists - Who Does What?

The Act and Regulations state that the site-investigation reports must be prepared by a certified engineering geologist or registered civil engineer, who must have competence in the field of seismic hazard evaluation and mitigation, and be *reviewed* by a certified engineering geologist or registered civil engineer, also competent in the field of seismic hazard evaluation and mitigation. *Although the Seismic Hazards Mapping Act does not distinguish between the types of licensed professionals who may prepare and review the report, the current Business and Professions Code (Geologist and Geophysicist Act, Section 7832; and Professional Engineers Act, Section 6704) restricts the practice of these two professions. Because of the differing expertise and abilities of engineering geologists and civil engineers, the scope of the site-investigation report for the project may require that* ***both*** *types of professionals prepare and review the report, each practicing in the area of his or her expertise*. Involvement of both engineering geologists and civil engineers will generally provide greater assurance that the hazards are properly identified, assessed, and mitigated.

The State Mining and Geology Board recommends that engineering geologists and civil engineers conduct the assessment of the surface and subsurface geological/geotechnical conditions at the site, including off-site conditions, to identify potential hazards to the project. It is appropriate for the civil engineer to design and recommend mitigation measures. It also is appropriate for both engineering geologists and civil engineers to be involved in the implementation of the mitigation measures– engineering geologists to confirm the geological conditions and civil engineers to oversee the implementation of the approved mitigation measures.

CHAPTER 3

OVERVIEW OF INVESTIGATIONS FOR ASSESSING SEISMIC HAZARDS

Introduction

Investigation of potential seismic hazards at a site can be performed in two steps or stages: (1) a preliminary **screening investigation**, and (2) a **quantitative evaluation** of the seismic hazard potential and its consequences. As noted below, it is possible to successfully complete the investigation by skipping one or the other stage. For example, a consultant's screening investigation may find that a previous site-specific investigation, on or adjacent to the project site, has shown that no seismic hazards exist, and that a quantitative evaluation is not necessary. Conversely, a consultant may know from experience that a project site is susceptible to a given hazard, and may opt to forego the screening investigation and start with a quantitative evaluation of the hazard.

Some lead agency reviewers recommend that for large projects the developer's consultant(s) meet with the lead agency technical reviewer prior to the start of the site investigation. This allows the consultant and technical reviewer to discuss the scope of the investigation. Topics of this discussion may include the area to be investigated for various hazards, the acceptability of investigative techniques to be used, on-site inspection requirements, or other local requirements.

Items to Consider in the Site Investigation Study

The following concepts are provided to help focus the site-investigation report:

1. Consultants are encouraged to utilize, if possible, the latest California Department of Conservation, Division of Mines and Geology (DMG) seismic ground-motion parameter data. This information is available in DMG's Seismic Hazard Evaluation Reports. The hazard zone mapping procedure for liquefaction and earthquake-induced landsliding utilizes state-of-the-art probabilistic ground-motion parameters developed jointly by the DMG and the U.S. Geological Survey, and published by the DMG (Petersen and others, 1996).

2. The fact that a site lies within a mapped zone of required investigation does not necessarily indicate that a hazard requiring mitigation is present. Instead, it indicates that regional (that is, not site-specific) information suggests that the probability of a hazard requiring mitigation is great enough to warrant a site-specific investigation. However, the working premise for the planning and execution of a site investigation within Seismic Hazard Zones is that ***the suitability of the site should be demonstrated***. This premise will persist until either: (a) the site investigation satisfactorily demonstrates the absence of liquefaction or landslide hazard, or (b) the site investigation satisfactorily defines the liquefaction or landslide hazard and provides a suitable recommendation for its mitigation.

3. The fact that a site lies outside a mapped zone of required investigation does not necessarily mean that the site is free from seismic or other geologic hazards, nor does it preclude lead agencies from adopting regulations or procedures that require site-specific soil and/or geologic investigations and mitigation of seismic or other geologic hazards. It is possible that development proposals may involve alterations (for example, cuts, fills, and/or modifications that would significantly raise the water table) that could cause a site outside the zone to become susceptible to earthquake-induced ground failure.

4. Lead agencies have the right to approve (and the obligation to reject) a proposed project based on the findings contained in the site-investigation report and the lead agency's technical review. The task of the developer's consulting engineering geologist and/or civil engineer is to demonstrate, to the satisfaction of the lead agency's technical reviewer, that:

- The site-specific investigation is sufficiently thorough;
- The findings regarding identified hazards are valid; and
- The proposed mitigation measures achieve an acceptable level of risk, as defined by the lead agency and CCR Title 14, Section 3721(a).

Screening Investigation

The purpose of screening investigations for sites within zones of required investigation is to evaluate the severity of potential seismic hazards, or to screen out sites included in these zones that have a low potential for seismic hazards. If a screening investigation can **clearly** demonstrate the absence of seismic hazards at a project site, and if the lead agency technical reviewer concurs with this finding, the screening investigation will satisfy the site-investigation report requirement and no further investigation will be required. If the findings of the screening investigation cannot demonstrate the absence of seismic hazards, then the more-comprehensive quantitative evaluation needs to be conducted.

The documents reviewed should be both regional and, if information is available, site-specific in scope. The types of information reviewed during a screening investigation often includes topographic maps, geologic and soil engineering maps and reports, aerial photographs, water well logs, agricultural soil survey reports, and other published and unpublished references. The references used should focus on current journals, maps, reports, and methods. Seismic Hazard Evaluation Reports, which summarize the findings and data on which DMG's Seismic Hazard Zone Maps are based, can provide much of the regional geologic and seismic information needed for a screening investigation. Aerial photographs can be useful to identify existing and potential landslide and/or liquefaction features (headwall scarps, debris chutes, fissures, grabens, sand boils, etc.) that suggest or preclude the existence of ground failure hazards that might affect the site. Several sets of stereoscopic aerial photographs that pre-date project site area development, and taken during different seasons of the year are particularly useful for identifying subtle geomorphic features. A field reconnaissance of the area is highly recommended to verify the information developed in the earlier steps to fill in information in questionable areas, and to observe the surface features and details that could not be determined from other data sources.

Quantitative Evaluation of Hazard Potential

Detailed Field Investigations – General Information Needs

Within the zone of required investigations, the objective of the detailed field investigation is to obtain sufficient information on which the engineering geologist and/or civil engineer can evaluate the nature and severity of the risk and develop a set of recommendations for mitigation. In the case of projects where the property is to be subdivided and sold to others undeveloped, the aim of the investigation is to determine which parcels contain buildable sites that meet the previously defined acceptable level of risk. The work should be based upon a detailed, accurate topographic base map prepared by a registered civil engineer or land surveyor. The map should be of suitable scale, and should cover the area to be developed as part of the project, as well as adjacent areas: which affect or may be affected by the project.

The detailed field investigation commonly involves the collection of subsurface information from trenches or borings, on or adjacent to the site. The subsurface exploration should extend to depths sufficient to expose geologic and subsurface water conditions that could affect slope stability or liquefaction potential. A sufficient quantity of subsurface information is needed to permit the engineering geologist and/or civil engineer to extrapolate with confidence the subsurface conditions that might affect the project, so that the seismic hazard can be properly evaluated, and an appropriate mitigation measure can be designed by the civil.

The preparation of engineering geologic maps and geologic cross sections is often an important step to developing an understanding of the significance and extent of potential seismic hazards. These maps and/or cross sections should extend far enough beyond the site to identify off-site hazards and features that might affect the site.

Content of Reports

The site investigation report should contain sufficient information to allow the lead agency's technical reviewer to satisfactorily evaluate the potential for seismic hazards and the proposed mitigation. No attempt is made here to define the limits of what constitutes a complete screening investigation or quantitative evaluation report. Site-specific conditions and circumstances, as well as lead agency requirements, will dictate which issues and what level of detail are required to adequately define and mitigate the hazard(s). The following list (Table 2) is provided to assist investigators and reviewers in identifying seismic hazard-related factors significant to the project. Not all of the information in the list will be relevant or required, and some investigations may require additional types of data or analyses.

Table 2. Recommended content for site-investigation reports within zones of required investigations.
Reports that address liquefaction and/or earthquake-induced landslides should include, but not necessarily be limited to, the following data:
1. Description of the proposed project's location, topographic relief, drainage, geologic and soil materials, and any proposed grading.
2. Site plan map of project site showing the locations of all explorations, including test pits, borings, penetration test locations, and soil or rock samples.
3. Description of seismic setting, historic seismicity, nearest pertinent strong-motion records, and methods used to estimate (or source of) earthquake ground-motion parameters used in liquefaction and landslide analyses.
4. 1:24,000 or larger-scale geologic map showing bedrock, alluvium, colluvium, soil material, faults, shears, joint systems, lithologic contacts, seeps or springs, soil or bedrock slumps, and other pertinent geologic and soil features existing on and adjacent to the project site.
5. Logs of borings, test pits, or other subsurface data obtained.
6. Geologic cross sections depicting the most critical (least stable) slopes, geologic structure, stratigraphy, and subsurface water conditions, supported by boring and/or trench logs at appropriate locations.
7. Laboratory test results; soil classification, shear strength, and other pertinent geotechnical data.
8. Specific recommendations for mitigation alternatives necessary to reduce known and/or anticipated geologic/seismic hazards to an acceptable level of risk.
Reports that address earthquake-induced landslides may also need to include:
1. Description of shear test procedures (ASTM or other) and test specimens.
2. Shear strength plots, including identification of samples tested, whether data points reflect peak or residual values, and moisture conditions at time of testing.
3. Summary table or text describing methods of analysis, shear strength values, assumed groundwater conditions, and other pertinent assumptions used in the stability calculations.
4. Explanation of choice of seismic coefficient and/or design strong-motion record used in slope stability analysis, including site and/or topographic amplification estimates.
5. Slope stability analyses of critical (least-stable) cross sections, which substantiate conclusions and recommendations concerning stability of natural and as-graded slopes.
6. Factors of safety against slope failure and/or calculated displacements for the various anticipated slope configurations (cut, fill, and/or natural slopes).
7. Conclusions regarding the stability of slopes with respect to earthquake-induced landslides and their likely impact on the proposed project.
8. Discussion of proposed mitigation measures, if any, necessary to reduce damage from potential earthquake-initiated landsliding to an acceptable level of risk.
9. Acceptance testing criteria (e.g., pseudo-static factor of safety), if any, that will be used to demonstrate satisfactory remediation.
Reports that address liquefaction hazards may also need to include the following:
1. If methods other than Standard Penetration Test (SPT; ASTM D1586-92) and Cone Penetration Test (CPT; ASTM 3441-94) are used, description of pertinent equipment and procedural details of field measurements of penetration resistance (borehole type, hammer type and drop mechanism, sampler type and dimensions, etc.).
2. Boring logs showing raw (unmodified) N-values if SPT's are performed; CPT probe logs showing raw qc-values and plots of raw sleeve friction if CPT's are performed.
3. Explanation of the basis and methods used to convert raw SPT, CPT, and/or other non-standard data to "corrected" and "standardized" values.
4. Tabulation and/or plots of corrected values used for analyses.
5. Explanation of methods used to develop estimates of field loading equivalent uniform cyclic stress ratios (CSReq) used to represent the anticipated field earthquake excitation (cyclic loading).

Table 2. Recommended content for site-investigation reports within zones of required investigations.
6. Explanation of the basis for evaluation of the equivalent uniform cyclic stress ratio necessary to cause liquefaction (CSRliq) within the number of equivalent uniform loading cycles considered representative of the design earthquake
7. Factors of safety against liquefaction at various depths and/or within various potentially liquefiable soil units.
8. Conclusions regarding the potential for liquefaction and its likely impact on the proposed project.
9. Discussion of proposed mitigation measures, if any, necessary to reduce potential damage caused by liquefaction to an acceptable level of risk.
10. Criteria for SPT-based, CPT-based, or other types of acceptance testing, if any, that will be used to demonstrate satisfactory remediation.

CHAPTER 4

ESTIMATION OF EARTHQUAKE GROUND-MOTION PARAMETERS

Introduction

Quantitative analyses of in-situ liquefaction resistance and earthquake-induced landslide potential requires site-specific assessment of ground shaking levels suitable for those purposes. A simplified Seed-Idriss (1982) liquefaction analysis requires an estimation of peak ground acceleration (PGA) and earthquake magnitude. A pseudo-static slope stability analysis may require estimates of PGA and magnitude for the selection of an appropriate seismic coefficient. If a seismic site response analysis is needed, or if a finite element analysis, a Newmark analysis or a dynamic analysis is to be performed, a representative strong-motion record will need to be selected on the basis of site-specific ground-motion parameter estimates. The following sections of this Chapter provide guidance on the selection of site-specific ground-motion parameters and representative strong-motion records.

Simple Prescribed Parameter Values (SPPV)

Probabilistic ground-motion parameter values on firm rock for PGA, predominant magnitude, and distance in the form of statewide maps have been jointly prepared by DMG and the U.S. Geological Survey for a 10 percent probability of exceedance over a 50-year period (Petersen and others, 1996). Versions of these maps covering a 7.5 minute quadrangle area at a scale of 1:100,000 are included in the Seismic Hazard Evaluation Reports that accompany Seismic Hazard Zone Maps. The predominant magnitude and distance maps are not dependent on site conditions, and can be used for site-specific purposes. PGA can be dependent on site conditions and several maps have been prepared to accommodate these differences, each based on site-dependent attenuation relations consistent with the soil profile types identified in the Uniform Building Code (ICBO, 1997). These maps are included in the Seismic Hazard Evaluation Reports issued by DMG, and can be used to obtain PGA as follows:

1. Classify the site according to the procedures and soil profile types defined in Chapter 16 of the Uniform Building Code (ICBO, 1997), and interpolate PGA from the corresponding PGA map; or

2. Interpolate PGA from the representative bedrock PGA map, and apply an appropriate scaling factor based on the soil profile type; or Perform a site response analysis (e.g., using a finite-element or

3. Perform a site response analysis (e.g., using a finite-element or SHAKE program to simulate the effects of ground-motion propagating through a soil column). Bedrock PGA and predominant magnitude and distance obtained from the above maps can be used to select an appropriate strong-motion record for input into the response analysis.

PGA estimated by the above procedures may still require additional adjustment to account for topographic and basin effects. Use of the SPPV method is not recommended for sites located very near to seismic sources, where reliable ground-motion estimates may require consideration of near-field source effects.

Probabilistic Seismic Hazard Analysis (PSHA)

Site-specific probabilistic seismic hazard analyses can be performed, and can supersede the SPPV-values of PGA for seismic hazard studies, even if PSHA studies result in adoption of a lower level of seismic ground motion. PSHA studies typically include the following:

1. A database consisting of potentially damaging earthquake sources, including known active faults and historic seismic source zones, their activity rates, and distances from the project site. This should include a comparison with DMG-developed slip rates for faults considered in the DMG statewide probabilistic seismic hazard map. Differences in slip rates should be documented and the reasons for them explained (for example, revised slip rates or new paleoseismic information from recent studies). DMG recommends that their earthquake source database be used directly, because it is updated regularly and is readily available (Petersen and others, 1996; see the World Wide Web at http://www.consrv.ca.gov/dmg/shezp/);
2. Use of published maximum moment magnitudes for earthquake sources, or estimates that are justified, well-documented, and based on published procedures;
3. Use of published curves (or those used by DMG) for attenuation of PGA with distance from earthquake source, as a function of earthquake magnitude and travel path (e.g., see special issue of Seismological Research Letters, v. 68, n. 1, 1997);
4. An evaluation of the likely effects of site-specific response characteristics (e.g., amplification due to soft soils, deep sedimentary basins, topography, near-source effects, etc.);
5. Characterization of the ground motion at the site in terms of PGA with a 10 percent probability of exceedance in 50 years, taking into account historical seismicity, available paleoseismic data, the geological slip rate of regional active faults, and site-specific response characteristics.

Useful references include Reiter, 1990; National Research Council, 1988; Hayes, 1985; Algermissen and others, 1982; Cornell, 1968; Youngs and Coppersmith, 1985; Working Group on California Earthquake Probabilities, 1990 and 1995; Okumura and Shinozuka, 1990; and Kramer, 1996.

Deterministic Seismic Hazard Analysis (DSHA)

Deterministic evaluation of seismic hazard can also be performed, and the results of correctly performed and suitably comprehensive DSHA studies can also supersede SPPV values of PGA. DSHA studies typically include the following:

1. Evaluation of potentially damaging earthquake sources, and deterministic selection of one or more suitable "controlling" sources and seismic events. The deterministic earthquake event magnitude for any fault should be a ***maximum*** value that is specific to that seismic source. Maximum earthquakes may be assessed by estimating rupture dimensions of the fault (e.g., Wells and Coppersmith, 1994; dePolo and Slemmons, 1990). The DMG database of earthquake sources is readily available (see section on PSHA).

2. Use of published curves for the effects of seismic travel path using the shortest distance from the source(s) to the site (e.g., see special issue of Seismological Research Letters, v. 68, n.1, 1997);

3. Evaluation of the effects of site-specific response characteristics on either (a) site accelerations, or (b) cyclic shear stresses within the site soils of interest.

Selection of a Site-Specific Design Strong-Motion Record

In the course of performing a seismic slope stability or liquefaction analysis, it is often necessary to select a design strong-motion record that represents the anticipated earthquake shaking at a project site. For a seismic slope-stability analysis the design strong-motion record will be used to evaluate the site seismic response (site amplification) and/or for the calculation of Newmark displacements. In some cases, the strong-motion record will be the input ground motion for a detailed dynamic analysis. For liquefaction evaluations the design strong-motion record will be used for the site seismic response to determine the appropriate peak ground acceleration to use in a simplified Seed-Idriss liquefaction analysis. It could also be used for a detailed finite-element analysis where the magnitude of potential lateral spread displacements are critical to the proposed project.

The selection process typically involves two steps: (1) estimating magnitude, epicentral distance and peak ground acceleration parameters for the project site, and (2) searching for existing strong-motion records that have parameters that closely match the estimated values. The methods described in the preceding sections of this chapter describe the recommended approaches to the parameter estimates. The selection of a representative strong-motion record should consider the following:

1. The selection should be based primarily on matching magnitude, epicentral distance, site conditions and PGA between the site and the record, generally in that order;

2. It may not always be possible to find a good match between the site parameters and the existing strong-motion records, and it may be necessary to use a record that does not match the site parameter criteria and scale it to fit those parameters, making sure that the duration of the scaled record is appropriate for the anticipated magnitude;
3. If the site to be analyzed is underlain by soils or weakly cemented rock, and a strong-motion recording site with similar characteristics cannot be found, a seismic site response analysis should be performed (e.g., SHAKE91, Idriss and Sun, 1992; SHAKE, Schnabel and others, 1972);

4. For project sites that could experience earthquakes from both high-frequency, near-source events and low-frequency, long-duration events, multiple records representative of these events should be included in the analysis.

A database of strong-motion records is available at the DMG World Wide Web site { http://www.consrv.ca.gov/dmg/ }. This and other sources for acquiring strong-motion records are provided in Appendix D.

CHAPTER 5

ANALYSIS AND MITIGATION OF EARTHQUAKE-INDUCED LANDSLIDE HAZARDS

Screening Investigations for Earthquake-Induced Landslide Potential

The purpose of screening investigations for sites within zones of required investigation for earthquake-induced landslides is to evaluate the severity of the hazard, or to screen out sites included in these zones that have a low potential for landslide hazards. If a screening investigation can ***clearly*** demonstrate the absence of earthquake-induced landslide hazard at a project site, and if the lead agency technical reviewer concurs with this finding, the screening investigation will satisfy the site-investigation report requirement and no further investigation will be required. If the findings of the screening investigation cannot demonstrate the absence of the hazard, then the more-comprehensive quantitative evaluation needs to be conducted.

An important aspect of evaluating the potential for earthquake-induced landslides is the recognition of the types of slope failures commonly caused by earthquakes. Keefer (1984) studied 40 historical earthquakes and found that different types of landslides occur with different frequencies. Table 3 summarizes Keefer's findings. In addition, Keefer (1984) summarized the geologic environments that are likely to produce earthquake-induced landslides. A table of these environments is provided in Appendix E to assist in the evaluation of project sites for the screening investigations.

The screening investigation should evaluate, and the report should address, the following basic questions:

- **Are existing landslides, active or inactive, present on, or adjacent (either uphill or downhill) to the project site?**

  An assessment of the presence of existing landslides on the project site for a screening investigation will typically include a review of published and unpublished geologic and landslide inventory maps of the area and an interpretation of aerial photographs. The distinctive landforms associated with landslides (scarps, troughs, disrupted drainages, etc.) should be noted, if present, and the possibility that they are related to landslides should be assessed.

Table 3. Relative abundance of earthquake-induced landslides from 40 historical earthquakes (Keefer, 1984; Table 4, p. 409).

Relative Abundance of Earthquake-Induced Landslides	Description

Very Abundant (more than 100,000 in the 40 earthquakes)	Rock falls, disrupted soil slides, rock slides
Abundant (10,000 to 100,000 in the 40 earthquakes)	Soil lateral spreads, soil slumps, soil block slides, soil avalanches
Moderately common (1000 to 10,000 in the 40 earthquakes)	Soil falls, rapid soil flows, rock slumps
Uncommon (100 to 1000 in the 40 earthquakes)	Subaqueous landslides, slow earth flows, rock block slides, rock avalanches

- **Are there geologic formations or other earth materials located on or adjacent to the site that are known to be susceptible to landslides?**

  Many geologic formations in California, notably late Tertiary and Quaternary siltstones and shales (for example, the Orinda and Modelo formations), are highly susceptible to landsliding. These rock units are generally well known among local engineering geologists. For some areas, susceptible formations have also been noted on the Landslide Hazard Identification Maps published by DMG.

- **Do slope areas show surface manifestations of the presence of subsurface water (springs and seeps), or can potential pathways or sources of concentrated water infiltration be identified on or upslope of the site?**

  Subsurface water in slopes can be an important indicator of landslide potential. Water may be forced to the surface along impermeable layers such as landslide rupture surfaces. Springs, seeps, or vegetation (phreatophytes) may result from impermeable layers and near-surface water. Topographic depressions, heavy irrigation, or disrupted surface water channels can cause ponding and increased infiltration of surface water. These features may be visible on pre- and/or post-development aerial photographs taken during certain seasons, or during a field reconnaissance. Presence of shallow subsurface water is significant because pore-water pressure reduces the forces resisting landslide movement.

- **Are susceptible landforms and vulnerable locations present? These include steep slopes, colluvium-filled swales, cliffs or banks being undercut by stream or wave action, areas that have recently slid.**

  In addition to existing landslide deposits, certain other slopes are especially susceptible to landsliding. These include very steep slopes, and ones where the support at the base of the slope has been removed or reduced. Removal of support at the base of a slope occurs naturally by stream or wave erosion and the same effect can be produced by grading of cut slopes. Colluvium-filled swales usually develop naturally over thousands of years, and the resulting thick, deeply weathered soil may be especially susceptible to debris flows. Hazardous slope features can generally be noted on aerial photographs, sufficiently detailed topographic maps, or from a geologic field reconnaissance.

- **Given the proposed development, could anticipated changes in the surface and subsurface hydrology (due to watering of lawns, on-site sewage disposal, concentrated runoff from impervious surfaces, etc.) increase the potential for future landsliding in some areas?**

  Misdirected runoff from streets during rainstorms can cause saturation of surficial materials and, in turn, development of catastrophic debris flows. Improperly designed highway culverts and watering of lawns on marine terraces can create unstable gullies, undermined coastal bluffs, or both. It is likely that the proposed development will alter the local groundwater regime in some way. The investigation should describe the likely effects that altered runoff patterns, lawn watering or septic systems will have on slope stability; identify sensitive areas; and, when warranted, recommend mitigation.

Additional Considerations

The Earthquake-Induced Landslides Working Group recommends that the screening investigation should include a site reconnaissance by the project's engineering geologist and/or civil engineer. This will allow for the recognition of potential earthquake hazards that cannot normally be recognized in a purely office-based screening investigation.

Guidance on the preparation of a report for the screening investigation is provided in Chapter 3 of these Guidelines. If the results of the screening investigation show that the potential for earthquake-induced landsliding is low, the report should state the reasons why a quantitative evaluation is not needed for the project site.

Quantitative Evaluation of Earthquake-Induced Landslide Potential

If the screening investigation indicates the presence of potentially unstable slopes affecting the proposed project site, a quantitative evaluation of earthquake-induced landslide potential should be conducted. The major phases of such a study typically includes a detailed field investigation, drilling and sampling, geotechnical laboratory testing, and slope stability analyses. Reference should be made to Chapter 3 for guidance on what types of information from the following sections should be included in the site-investigation report.

Detailed Field Investigation

Engineering Geologic Investigations

The engineering geologic investigation phase of the project site investigation consists of surface observations and geologic mapping. The overall scope of the engineering geologic investigation for earthquake-induced landslide hazards is fundamentally the same as the work that would be conducted for any project that has potential landslide hazards, regardless of the triggering mechanism. However, the investigator should keep in mind the environments and the relative abundance of landslide types triggered by earthquakes as described by Keefer (1984) and shown in Appendix E and Table 3, respectively. The engineering geologic investigation is significant because it provides the basis for the subsurface investigations, field instrumentation, and geotechnical analyses that follow.

Prior to the site reconnaissance, the area of the project should be identified, and available geologic and geotechnical information, stereoscopic aerial photographs, and topographic maps should be collected and reviewed (Keaton and DeGraff, 1996). If a screening investigation has been conducted for the site, much of this information may already have been reviewed. Once the results of the office-based investigation have been completed and understood, on-site engineering geologic mapping can be conducted.

The purpose of the on-site engineering geologic mapping is to document surface conditions which, in turn, provides a basis for projecting subsurface conditions that may be relevant to the stability of the site. The on-site engineering geologic mapping should identify, classify, and locate on a map the features and characteristics of existing landslides, and surficial and bedrock geologic materials. Other important aspects of the site to document include: landslide features and estimates of depth to the rupture surface; distribution and thickness of colluvium; rock discontinuities such as bedding, jointing, fracturing and faulting; depth of bedrock weathering; surface water features such as streams, lakes, springs, seeps, marshes, and closed or nearly closed topographic depressions.

Engineering geologic cross sections should be located so as to provide information that will be needed for planning subsurface investigations and stability analyses. The most useful orientation is typically perpendicular to topographic contours and longitudinally down existing landslide deposits. The projected shape of the rupture surface, geologic contacts and orientations, and groundwater surfaces should be shown along with the topographic profile. Estimates of the depth to the landslide rupture surface is an important parameter for planning a subsurface investigation and longitudinal cross sections can be helpful in making these estimates (McGuffey and others, 1996).

The results of the engineering geologic mapping can be presented in many forms, but generally should include a map, cross sections, and proposed subsurface investigation locations and/or field instrumentation sites. Whatever method of presentation is chosen, it should be remembered that the presentation of the surface mapping information needs to be characterized in terms that are meaningful for, and usable by the design engineer. Doing so will help ensure that key factors that must be accommodated in the construction are understood (Keaton and DeGraff, 1996).

Subsurface Investigation

Planning

Exploratory work by the engineering geologist and civil engineer should be conducted at locations considered most likely to reveal any subsurface conditions which may indicate the potential for earthquake-induced landslide failures. In particular, an investigation should locate and define the geometry of bedding and fracture surfaces, contacts, faults, and other discontinuities as well as actual landslide rupture surfaces.
Subsurface exploration methods can be classed as direct methods and indirect methods (Hunt, 1984a). Direct methods, such as test borings and the excavation of test pits or trenches, allow the examination of the earth materials, usually with the removal of samples. Indirect methods, such as

geophysical surveys and the use of the cone penetrometer, provide a measure of material properties that allows the estimation of the material type (McGuffey and others, 1996).

Subsurface investigations should be supervised by an experienced engineering geologist and/or civil engineer to ensure that the field activities are properly executed and the desired results are achieved. According to McGuffey and others (1996), the subsurface investigation field supervision should:

1. Ensure that technical and legal contract specifications are followed,

2. Maintain liaison with the designer of the exploration program,

3. Select and approve modifications to the program as new or unanticipated conditions are revealed,

4. Ensure that complete and reliable field reports are developed; and

5. Identify geologic conditions accurately.

The depth to which explorations should extend can be difficult to define in advance of the subsurface investigation. Cross sections from a surface engineering geological investigation can be helpful in planning the depths of excavations required in a subsurface investigation. In general, borings or other direct investigative techniques should extend deep enough (a) to identify materials that have not been subjected to movements in the past but might be involved in future movements, and (b) to clearly identify underlying stable materials. The exploration program plan should be flexible enough to allow for expanding the depth of investigation when the data obtained suggest deeper movements are possible (McGuffey and others, 1996).

Samples and Sampling

Soil and rock samples that may be obtained from subsurface borings and excavations belong to one of two basic categories: disturbed and undisturbed samples. Disturbed samples are collected primarily for soil classification tests where the preservation of the soil structure is not essential, or for remolding in the laboratory and subsequent strength and compressibility tests. Undisturbed samples do not entirely represent truly undisturbed soil or rock conditions because the process of sampling and transporting inevitably introduces some disturbance into the soil or rock structure.

These samples are taken primarily for laboratory strength and compressibility tests and for the measurement of in-situ material properties.

Samples of the soil, the existing landslide rupture materials, and the weakest components of rock units should be taken for laboratory measurement of engineering properties. Special care should be taken to obtain oriented samples of existing zones of weakness or rupture surfaces. For shallow landslides it may be possible to expose and sample critical zones of weakness in the walls of trenches or test pits. For deep-seated landslides it often is extremely difficult to sample the zones of weakness with typical geotechnical drilling equipment, and it may be appropriate to consider using bucket auger drilling and down-hole geologic logging and sampling techniques (Scullin, 1994).

It is the responsibility of the field supervising geologist or engineer to accurately label and locate the collected samples. He or she is also responsible for the proper transportation of collected samples, particularly undisturbed samples, to prevent sample disturbance by excessive shocks, allowing samples to dry or slake, or by exposing samples to heat or freezing conditions. A large variety of soil boring techniques and sampler types is available. A detailed explanation of the many types is beyond the scope of these Guidelines, but is readily available in the literature (Hvorslev, 1948; ASTM, 1971 and 1997; U.S. Bureau of Reclamation, 1974 and 1989; U.S. Navy, 1986; Hunt, 1984a; Krynine and Judd, 1957; Acker, 1974; Scullin, 1994; Johnson and DeGraff, 1988; McGuffey and others, 1996).

Subsurface Water

The presence of subsurface water can be a major contributing factor to the dynamic instability of slopes and existing landslides. Therefore, the identification and measurement of subsurface water in areas of suspected or known slope instability should be an integral part of the subsurface investigation. The location and extent of groundwater, perched groundwater and potential water barriers should be defined. Subsurface water conditions within many landslides are best considered as complex, multiple, partially connected flow systems. McGuffey and others (1996) have listed the following recommendations:

1. Surface observations are essential in determining the effect of subsurface water on landslide instability.

2. Periodic or seasonal influx of surface water to subsurface water will not be detected unless subsurface water observations are conducted over extended time periods.

3. Landslide movements may open cracks and develop depressions at the head of a landslide that increase the rate of infiltration of surface water into the slide mass.

4. Ponding of surface water anywhere on the landslide may cause increased infiltration of water into the landslide and should be investigated.

5. Disruption of surface water channels and culverts may also result in increased infiltration of surface water into the landslide.

6. Landslide movements may result in blockage of permeable zones that were previously freely draining. Such blockage may cause a local rise in the groundwater table and increased saturation and instability of the landslide materials. Subsurface observations should therefore be directed to establishing subsurface water conditions in the undisturbed areas surrounding the landslide.

7. Low permeability soils, which are commonly involved in landslides, have slow response times to changes in subsurface water conditions and pressures. Long-term subsurface water monitoring is required in these soils.

8. Accurate detection of subsurface water in rock formations is often difficult because shale or claystone layers, intermittent fractures, and fracture infilling may occlude subsurface water detection by boring or excavation.

9. Borings should never be the only method of subsurface water investigation; nevertheless they are a critical component of the overall investigation.

Geotechnical Laboratory Testing

The geotechnical testing of soil and rock materials typically follows accepted published standards (ASTM, 1997; Head, 1989). Good professional judgment is expected in the selection of appropriate samples, shear tests, and interpretation of the results in arriving at strength characteristics appropriate for the present and anticipated future slope conditions. The following guidelines are provided for evaluating soil properties.

1. Soil properties, including unit weight and shear strength parameters (cohesion and friction angle), may be based on appropriate conventional laboratory and field tests.

2. Testing of earth materials should be in accordance with the appropriate ASTM Standards that are updated annually (ASTM, 1997).

3. Prior to shear tests, samples should be soaked a sufficient length of time to approximate a saturated moisture condition.

4. Stability analyses generally should use the lowest values derived from the suite of samples tested.

5. Residual test values should be used for static analysis of existing landslides, along shale bedding planes, highly distorted bedrock, over-consolidated fissured clays, and for paleosols and topsoil zones under fill. Peak values may be used for pseudo-static or dynamic calculations if the buildup of pore pressures is not anticipated and if permitted by the lead agency. Consideration of reducing the strength values for dynamic analyses should be made in light of the measured material properties and anticipated subsurface water conditions (see section on Effective-Stress vs. Total-Stress Conditions below).

6. Appropriate analyses of existing failures (back-calculated strengths) in slopes similar to that under consideration in terms of height, geology, and soil or rock materials may be helpful in determining or verifying proposed shear strength parameters.

7. Laboratory shear strength values used for design of fill slopes steeper than two horizontal to one vertical (2:1) and for buttress fills should be verified by testing during slope grading. In the event that the shear strength values from field samples are less than those used in design, the slope should be reanalyzed and modified as necessary to provide the required factor of safety for stability.

Slope Stability Analysis

General Considerations

Slope stability analysis will generally be required by the lead agency for cut, fill, and natural slopes whose slope gradient is steeper than two horizontal to one vertical (2:1), and on other slopes that possess unusual geologic conditions such as unsupported discontinuities or evidence of prior landslide activity. Analysis generally includes deep-seated and surficial stability evaluation under both static and dynamic (earthquake) loading conditions.

Evaluation of deep-seated slope stability should be guided by the following:

1. The potential failure surface used in the analysis may be composed of circles, planes, wedges or other shapes considered to yield the minimum factor of safety against sliding for the appropriate soil or rock conditions. The potential failure surface having the lowest factor of safety should be sought.

2. Forces to be considered include the gravity loads of the potential failure mass, structural surcharge loads and supported slopes, and loads due to anticipated earthquake forces. The potential for hydraulic head (or significant pore-water pressure) should be evaluated and its effects included when appropriate. Total unit weights for the appropriate soil moisture conditions are to be used.

Evaluation of surficial slope stability should be guided by the following:

1. Calculations may be based either on analysis procedures for stability of an infinite slope with seepage parallel to the slope surface or on another method acceptable to the lead agency. For the infinite slope analysis, the minimum assumed depth of soil saturation is the smaller of either a depth of one (1) meter or depth to firm bedrock. Soil strength characteristics used in analysis should be obtained from representative samples of surficial soils that are tested under conditions approximating saturation and at normal loads approximating conditions at very shallow depth.
2. Appropriate mitigation procedures and surface stabilization should be recommended, in order to provide the required level of surficial slope stability.
3. Recommendations for mitigation of damage to the proposed development caused by failure of off-site slopes should be made unless slope-specific investigations and analyses demonstrate that the slopes are stable. Ravines, swales, and hollows on natural slopes warrant special attention as potential sources of fast-moving debris flows and other types of landslides. If possible, structures should be located away from the base or axis of these types of features.

Analysis Methods Available

There are four generally accepted methods of slope stability analysis for seismic loading conditions. Two of these methods, the pseudo-static analysis and the Newmark analysis, have practical applications for most residential and commercial development projects affected by Seismic Hazard Zone Maps, and will be discussed in some detail in the following sections. The other two methods, the Makdisi-Seed (1978) analysis and the dynamic analysis, are not generally applicable to these types of developments. These latter two methods will only be briefly summarized in this section.

The simplest approach to a dynamic slope stability calculation is the **pseudo-static analysis,** in which the earthquake load is simulated by an "equivalent" static horizontal acceleration acting on the mass of the landslide, in a limit-equilibrium analysis (Nash, 1987; Janbu, 1973; Bromhead, 1986; Chowdhury, 1978; Morgenstern and Sangrey, 1978; Hunt, 1984b; Duncan, 1996). The pseudo-static approach has certain limitations (Cotecchia, 1987; Kramer, 1996), but this methodology is considered to be generally conservative, and is the one most often used in current practice.

The second procedure is known as the **Newmark or cumulative displacement analysis** (Newmark, 1965; Makdisi and Seed, 1978; Hynes and Franklin, 1984; Houston and others, 1987; Wilson and Keefer, 1983; Jibson, 1993). The procedure involves the calculation of the yield acceleration, defined as the inertial force required to cause the static factor of safety to reach 1.0, from the traditional limit-equilibrium slope stability analysis. The procedure then uses a design earthquake strong-motion record which is numerically integrated twice for the amplitude of the acceleration above the yield acceleration to calculate the cumulative displacements. These analytical displacements are then evaluated in light of the slope material properties and the requirements of the proposed development. The pseudo-static and Newmark analyses will be described in more detail in the following sections.

The third method is referred to as a **Makdisi-Seed analysis** (Makdisi and Seed, 1978; Kramer, 1996). Makdisi and Seed's work (1978) sought to define seismic embankment stability in terms of acceptable deformations in lieu of conventional factors of safety, using a modified Newmark analysis. Their method presents a rational means by which to determine yield acceleration, or the average acceleration required to produce a factor of safety of unity. This value, in turn is affected by the cyclic-yield strengths of embankment materials, which turned out to be about 80 percent of static strength. Design curves were developed to estimate the permanent earthquake-induced deformations of embankments 100 to 200 feet high using finite element analyses. These same methods have since been applied to sanitary landfill and highway embankments. Very little application of this method has been made to pre-existing landslides, and the method will not be reviewed in detail in these guidelines.

The most sophisticated method for seismic slope stability calculations is known as a **dynamic analysis** (Cotecchia, 1987) or a **stress-deformation analysis** (Kramer, 1996) and it typically incorporates a finite-element or finite-difference mathematical model. In this type of analysis ground motion is incorporated in the form of an acceleration time history. Seismically induced permanent strains in each element of the finite element mesh are integrated to obtain the permanent deformation of the slope. The results of the analysis include a time history of the compressive and tensile stresses, natural frequencies, effects of damping, and slope displacements. Because this type of analysis will only rarely be used for the types of projects affected by the Seismic Hazard Zone Maps, it will not be discussed further in these Guidelines.

Pseudo-Static Analysis

The ground-motion parameter used in a pseudo-static analysis is referred to as the seismic coefficient "k". The selection of a seismic coefficient has relied heavily on engineering judgment and local code requirements because there is no simple method for determining an appropriate value. In California, many state and local agencies, on the basis of local experience, require the use of a seismic coefficient of 0.15, and a minimum computed pseudo-static factor of safety of 1.0 to 1.2 for analyses of natural, cut, and fill slopes. The evaluation should follow the lead agency practice guidelines for seismic coefficient and factor of safety values. If no local guidelines exist, the following discussion should assist in the estimation an appropriate seismic coefficient.

Cautionary Note: *The seismic coefficient "k" is* ***not*** *equivalent to the peak horizontal ground acceleration value, either probabilistic or deterministic; therefore PGA should not be used as a seismic coefficient in pseudo-static analyses. The use of PGA will usually result in overly conservative factors of safety (Seed, 1979; Chowdhury, 1978). Furthermore, the practice of reducing the PGA by a "repeatable acceleration" factor to obtain a pseudo-static coefficient has no basis in the scientific or engineering literature.*

<u>Selection of a Seismic Coefficient</u>

There have been a number of published articles that provide guidance in the selection of an appropriate seismic coefficient for pseudo-static analyses. Most can be regarded as being within a range of values enveloped by the recommendations of two publications, Seed (1979), and Hynes and Franklin (1984).

Seed’s 1979 article (the 19th Rankine Lecture) summarizes the factors to be considered in evaluating dynamic stability of earth-and rock-fill embankments. After evaluating all of the available data on earthquake-induced deformations of embankment dams, Seed recommended some basic guidelines for making preliminary evaluations of embankments to ensure acceptable performance (i.e., permanent deformations which would not imperil the overall structural integrity of an embankment dam). These recommendations were: using a pseudo-static coefficient of 0.10 for magnitude 6½ earthquakes and 0.15 for magnitude 8¼ earthquakes, with an acceptable factor of safety of the order of 1.15. Seed believed that his guidelines would ensure that permanent ground deformations would be acceptably small. Seed also made extensive commentary on the choice of appropriate material strengths, and limited his recommendations to those embankments composed of materials that do not undergo severe strength loss due to seismic shaking with an expected crest acceleration of less than 0.75g.

Hynes and Franklin (1984) provided amplification factors to be used when considering the crest of an embankment in comparison to the input accelerations at the base, with the intention of identifying those embankments which could be expected to experience unacceptable deformations.

They suggested using one-half the bedrock acceleration applied to the embankment crest with an acceptable factor of safety greater than 1.0, with a 20 percent reduction on material strengths. Hynes and Franklin limited the assessment to earthquakes of less than magnitude 8 with non-liquefiable materials comprising the embankment.

Although the two references discussed above were written specifically for application to earth embankments, they represent the best understanding of the range of appropriate seismic coefficients to use in slopes composed of other materials. Figure 1 presents a summary of the recommended values of "k" for the ranges of factor of safety and earthquake parameters presented in these two articles. Other suggested ranges have been added for comparison. Figure 1 is presented as a guide for selecting a seismic coefficient for a pseudo-static analysis in jurisdictions where pseudo-static coefficients have not been adopted by the lead agency.

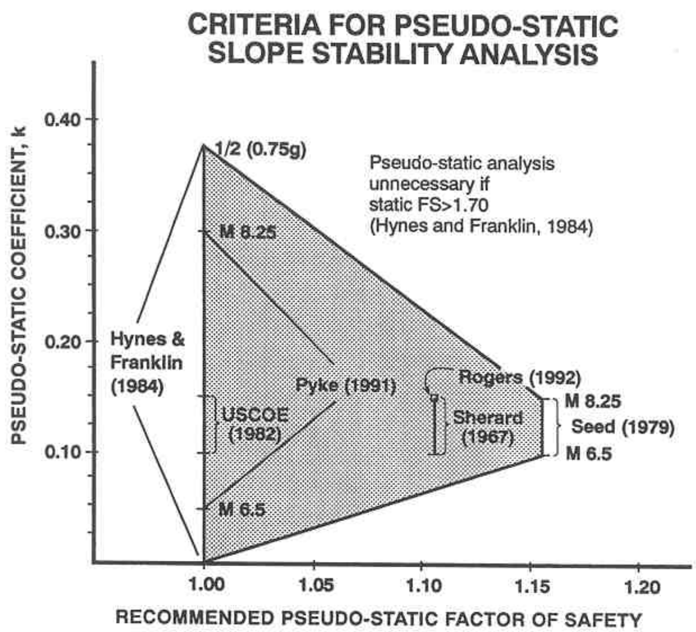

Figure 1. Approximate range of pseudo-static seismic coefficient "k" for anticipated factor of safety as proposed in the literature (references on the diagram)

Topographic Effects

Ashford and Sitar (1994) presented a method to analyze topographic amplification of site response on slopes. They specifically addressed the expected response of very steep slopes in weakly cemented rock. Amplification was found to increase with inclined seismic waves traveling into the slope crest. They found that the fundamental site period dominates the seismic response of any given slope. The relationship between wave-length and slope height controls the degree of amplification. However, as the slopes decrease in steepness (i.e., less than 30 degrees), the slope-induced amplification becomes less and less important, and geologic contacts between dissimilar strata appear to exert more influence on observed failures.

Material Strengths

The pseudo-static analysis does not take into account any loss of material strength due to pore-pressure buildup along the anticipated slide surface due to earthquake loading (effective-stress conditions). For most investigations where the slopes are unsaturated or partially saturated, this assumption will be valid and the results of the analysis will tend to be conservative. If, however, the slopes being evaluated are saturated or are anticipated to be saturated, the loss of material strength during long-duration earthquake shaking may be expected and the analysis using total strength parameters may be more appropriate (see section on Effective-Stress vs. Total-Stress Conditions below).

Newmark Analysis

A Newmark analysis consists of three basic steps, as outlined below:

1. The first step is to perform a limit-equilibrium stability analysis to determine the location and shape of the critical slip surface (the slip surface with the lowest factor of safety), and the yield acceleration (K_y), defined as the acceleration required to bring the factor of safety to 1.0. Most computer-based slope stability programs include iterative routines for finding both of these parameters. If a computer program with these options is not available, the critical slip surface can be obtained through iterative trial-and-error, and the yield acceleration can be calculated from Newmark's relation

$$K_y = (FS - 1)g \sin a$$

   where FS is the static factor of safety, g is the acceleration due to gravity, and a is the angle from the horizontal that the center of mass of the landslide first moves.

2. The second step is to select an acceleration time history that represents the expected ground motions at the project site. The selection process typically involves estimating magnitude, source-to-site distance, and peak ground acceleration seismic parameters for the project site, and searching for existing strong-motion records that have parameters that closely match the estimated values. Methods for determining these site parameters and selecting a representative strong-motion record are outlined in Chapter 4. For Newmark analyses, Jibson (1993) recommended using: (1) Arias Intensity (Wilson, 1993; Wilson and Keefer, 1985), (2) magnitude and source distance, and (3) PGA and duration as criteria for selecting a suite of strong-motion records having characteristics of interest at a project site. Smith (1994a; 1994b) compiled a database of these characteristics for a large number of strong-motion records. Analysis of multiple records spanning a range of estimated shaking characteristics produces a range of calculated displacements, which provides a better basis for judgment of slope performance than one displacement calculated from a single record that may have unique idiosyncrasies. If the slopes to be analyzed are composed of soils or weakly cemented rock, and a strong-motion recording site with similar characteristics cannot be found, a seismic site response analysis should be performed. Houston and others (1987) described a method of using a one-dimensional wave propagation program (e.g., SHAKE91, Idriss and Sun, 1992; SHAKE,

Schnabel and others, 1972) to find the average response at the slip surface prior to calculating displacements. As described in Chapter 4, sources for acquiring strong-motion records are provided in Appendix D.

3. The final step in a Newmark analysis is to calculate the cumulative displacements anticipated for the landslide under investigation. To do this, the design strong-motion record is integrated twice for those accelerations that exceed the yield acceleration, and the displacements are added to determine cumulative displacement. Computer software capable of calculating displacements from strong-motion records is available (Jibson, 1993; Houston and others, 1987) and can greatly simplify the analysis.

Jibson (1993) pointed out that, because Newmark's model assumes that landslides behave as rigid-plastic materials, the method might underestimate displacements for materials that lose shear strength as a function of strain, and overestimate displacements for soils that behave as viscoplastic materials. Due to the many assumptions that need to be made in the analysis, it is probably appropriate to consider calculations indicative only to within an order-of-magnitude of the actual displacements (e.g., centimeters, tens of centimeters, or meters). Considerable engineering judgment is required to establish 'stability.'

Effective-Stress vs. Total-Stress Conditions

In principle, a pseudo-static or Newmark analysis can be performed on either a total-stress or effective-stress basis. The geotechnical industry practice for 'typical' developments has been to determine shear strength parameters from direct shear tests (effective-stress conditions) and assume that static and dynamic shear strengths are the same. For most investigations where the slopes are unsaturated or partially saturated, this assumption will be valid and the results of the analysis will tend to be conservative. However, for saturated slopes this assumption ignores the build-up of pore pressures due to dynamic loading, which can lower the shear resistance to failure and, in some cases, result in unconservative stability evaluations.

Seed (1966) presented an approach to a total-stress analysis for earth embankments that uses dynamic shear tests to derive a factor of safety that accounts for (a) initial conditions; (b) changes in stress and reorientation of principal stress; (c) decrease in strength due to cyclic loading conditions; and (d) decrease in strength due to undrained conditions during earthquake loading. This method is rigorous, and provides good estimates of the dynamic behavior of saturated materials but may be too costly for most projects.

A simpler approach to a total stress analysis would be to determine total-stress strength parameters from undrained triaxial shear tests and use those values in the stability analysis. Jibson and Keefer (1993) showed how to conduct such an analysis, and their results indicated that factors of safety and critical slip surfaces differed significantly from those generated from an effective stress analysis. The U.S. Army Corps of Engineers practice is to use a composite shear strength envelope (based on a consolidated-drained test at low confining pressures and a consolidated-undrained test at high confining pressures) for permeable soils, and a consolidated-undrained strength envelope for soils with low permeability (Hynes and Franklin, 1984).

Makdisi and Seed (1978) have shown that substantial permanent displacements may be produced by cyclic loading of soils to stresses near the yield stress, while essentially elastic behavior is observed under many cycles of loading at 80 percent of the undrained strength. They recommend the use of 80 percent of the undrained strength for soils that exhibit small increases in pore pressure during cyclic loading, such as clayey soils, dry or partially saturated cohesionless soils, or very dense saturated cohesionless materials. This practice has been adopted by the U.S. Army Corps of Engineers with an allowable pseudo-static factor of safety of 1.0 (Hynes and Franklin, 1984) and may be appropriate for many stability analyses in the absence of a more rigorous total stress analysis.

Evaluation of Potential Earthquake-Induced Landslide Hazards

The determination of dynamic slope stability (i.e., pseudo-static factors of safety or analytical displacements), and the acceptable parameters used in the analysis, should follow the standards defined by the lead agency. If no standards exist, the following general values may be used for defining the stability of slopes for static and dynamic loads.

Pseudo-Static Analysis

Slopes that have a pseudo-static factor of safety greater than 1.1 using an appropriate seismic coefficient can be considered stable. If the pseudo-static analysis results in a factor of safety lower than 1.1, the project engineer can either employ a Newmark analysis (or other displacement-type analysis method if acceptable to the lead agency) to determine the magnitude of slope displacements, or design appropriate mitigation measures.

Newmark Analysis

The Newmark analysis models a highly idealized and simplistic failure mechanism; thus, as discussed previously, the calculated displacements should be considered order-of-magnitude estimates of actual field behavior. Rather than being an accurate guide of observable landslide displacement in the field, Newmark displacements provide an index of probable seismic slope performance, and considerable judgment is required in evaluating seismic stability in terms of Newmark displacements. In some jurisdictions, less than 10 cm is considered stable, whereas, more than 30 cm is considered unstable. As a general guideline,

1. Newmark displacements of 0 to 10 cm are unlikely to correspond to serious landslide movement and damage.

2. In the 10 to 100 cm range, slope deformation may be sufficient to cause serious ground cracking or enough strength loss to result in continuing (post-seismic) failure. Determining whether displacements in this range can be accommodated safely requires good professional judgment that takes into account issues such as landslide geometry and material properties.

3. Calculated displacements greater than 100 cm are very likely to correspond to damaging landslide movement, and such slopes should be considered unstable.

Mitigation of Earthquake-Induced Landslide Hazards

Basic Considerations

For any existing or proposed slopes that are determined to be unstable, appropriate mitigation measures should be provided before the project is approved. The hazards these slopes present can be mitigated in one of three ways:

1. **Avoid the Failure Hazard**: Where the potential for failure is beyond the acceptable level and not preventable by practical means, as in mountainous terrain subject to massive planar slides or rock and debris avalanches, the hazard should be avoided. Developments should be built sufficiently far away from the threat that they will not be affected even if the slope does fail. Planned development areas on the slope or near its base should be avoided and relocated to areas where stabilization is feasible.

2. **Protect the Site from the Failure**: While it is not always possible to prevent slope failures occurring above a project site, it is sometimes possible to protect the site from the runout of failed slope materials. This is particularly true for sites located at or near the base of steep slopes, which can receive large amounts of material from shallow disaggregated landslides or debris flows. Methods include catchment and/or protective structures such as basins, embankments, diversion or barrier walls, and fences. Diversion methods should only be employed where the diverted landslide materials will not affect other sites.

3. **Reduce the Hazard to an Acceptable Level**: Unstable slopes affecting a project can be rendered stable (that is, by increasing the factor of safety to > 1.5 for static and > 1.1 for dynamic loads) by eliminating the slope, removing the unstable soil and rock materials, or applying one or more appropriate slope stabilization methods (such as buttress fills, subdrains, soil nailing, crib walls, etc.). For deep-seated slope instability, strengthening the design of the structure (e.g., reinforced foundations) is generally not by itself an adequate mitigation measure.

The zones of required investigation for earthquake-induced landslides do not always include landslide or lateral spread run-out areas. Project sites that are outside of a zone of required investigation may be affected by ground-failure runout from adjacent or nearby slopes. Any proposed mitigation should address all recognized significant off-site hazards. If stabilization of source areas of potential off-site failures that could impact the project is not practical, it may be possible to achieve an acceptable level of risk by using one or more protective structures, as suggested below.

Stabilization Options

The stabilization method chosen depends largely on the type of instability, which is anticipated at the project site. The two general techniques used to stabilize slopes are: (1) to reduce the driving force for failure, or (2) to increase the resisting force. These consist of different mechanisms, depending on the type of failures in question. The following list is presented to provide a range of stabilization options, but other options may be recommended provided analyses are presented to prove their validity.

Rock and Soil Falls

Principal failure mechanism is loss of cohesion or tensile strength of the near-surface material on a very steep slope.

<u>Mitigation Strategies</u>

1. **Reduce driving force** by reducing the steepness of the slope through grading, or by scaling off overhanging rock, diverting water from the slope face, etc.;

2. **Increase resisting force** by pinning individual blocks, covering the slope with mesh or net, or installing rock anchors or rock bolts on dense spacing; and/or,

3. **Protect the site from the failure** by constructing catchment structures such as basins, or protective structures such as walls and embankments.

Slides, Slumps, Block Glides

Principal failure mechanism is loss of shear strength, resulting in sliding of a soil or rock mass along a rupture surface within the slope.

<u>Mitigation Strategies</u>

1. **Reduce driving force**, by reducing the weight of the potential slide mass (cutting off the head of the slide, or totally removing the landslide), flattening the surface slope angle ('laying back' the slope face) through grading, preventing water infiltration by controlling surface drainage, or reducing the accumulation of subsurface water by installing subdrains; and/or,

2. **Increase resisting force**, by replacing slide debris and especially the rupture surface with compacted fill, installing shear keys or buttresses, dewatering the slide mass, pinning shallow slide masses with soil or rock anchors, reinforced caissons, or bolts, or constructing retaining structures at the edge of the slide.

Flows of Debris or Soil

Principal failure mechanism is fluidization of the soil mass, commonly by addition of water and possibly by earthquake shaking.

Mitigation Strategies

1. **Reduce driving force** by removing potential debris from site using grading or excavating procedures, or diverting water from debris so that it cannot mobilize, by means of surface drains and/or subsurface galleries or subdrains;

2. **Increase resisting force** by providing shear keys or buttresses, together with subsurface drainage; and/or,

Protect the site from the failure by diverting the flow away from project using diversion barriers or channels, or providing catchment structures to contain the landslide material.

CHAPTER 6

Analysis and Mitigation of Liquefaction Hazards

Screening Investigations for Liquefaction Potential

The purpose of screening investigations for sites within zones of required investigation for liquefaction is to determine whether a given site has obvious indicators of a low potential for liquefaction failure (e.g., bedrock near the surface or deep ground water without perched water zones), or whether a more comprehensive field investigation is necessary to determine the potential for damaging ground displacements during earthquakes.

If a screening investigation can ***clearly*** demonstrate the absence of liquefaction hazards at a project site, and if the lead agency technical reviewer concurs with this finding, the screening investigation will satisfy the site-investigation report requirement. If there is a reasonable expectation that liquefiable soils exist on the site and the engineering geologist and/or civil engineer can demonstrate that large lateral spread displacements (of more than 0.5 meter) are unlikely (e.g., Bartlett and Youd, 1995), the local agency may give them the option to forego the quantitative evaluation of liquefaction hazards and provide a structural mitigation for certain classes of structures. These mitigation methods are outlined in the mitigation section of this chapter. If the findings of the investigation fall outside these two options, then the more-comprehensive quantitative evaluation described below needs to be conducted.

Screening investigations for liquefaction hazards should address the following basic questions:

- **Are potentially liquefiable soil types present?**

  Given the highly variable nature of Holocene deposits that are likely to contain liquefiable materials, most sites will require borings to determine whether liquefiable materials underlie the project site. Borings used to define subsurface soil properties for other purposes (e.g., foundation investigations, environmental or groundwater studies) may provide valuable subsurface geologic and/or geotechnical information.

  The vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Cohesive soils are generally not considered susceptible to soil liquefaction. However, cohesive soils with: (a) a clay content (percent finer than 0.005 mm) less than 15 percent, (b) a liquid limit less than 35 percent, and (c) a moisture content of the in-place soil that is greater than 0.9 times the liquid limit (i.e., sensitive clays), are vulnerable to significant strength loss under relatively minor strains (Seed and others, 1983). Although not classically defined as "liquefaction" and so not addressed by these Guidelines, these soils represent an additional seismic hazard that, if present, should be addressed.
  In addition to sandy and silty soils, some gravelly soils are potentially vulnerable to liquefaction. Most gravelly soils drain relatively well, but when: (a) their voids are filled with finer particles, or (b) they are surrounded by less pervious soils, drainage can be impeded and they may be vulnerable to cyclic pore pressure generation and liquefaction. Gravelly geologic

units tend to be deposited in a more-turbulent depositional environment than sands or silts, tend to be fairly dense, and so generally resist liquefaction. Accordingly, conservative "preliminary" methods may often suffice for evaluation of their liquefaction potential. For example, gravelly deposits which can be shown to be pre-Holocene in age (older than about 11,000 years) are generally not considered susceptible to liquefaction.

- **If present, are the potentially liquefiable soils saturated or might they become saturated?**

  In order to be susceptible to liquefaction, potentially liquefiable soils must be saturated or nearly saturated. In general, liquefaction hazards are most severe in the upper 50 feet of the surface, but on a slope near a free face or where deep foundations go beyond that depth, liquefaction potential should be considered at greater depths. If it can be demonstrated that any potentially liquefiable materials present at a site: (a) are currently unsaturated (e.g., are above the water table), (b) have not previously been saturated (e.g., are above the historic-high water table), and (c) are highly unlikely to become saturated (given foreseeable changes in the hydrologic regime), then such soils generally do not constitute a liquefaction hazard that would require mitigation. Note that project development, changes in local or regional water management patterns, or both, can significantly raise the water table or create zones of perched water. Extrapolating water table elevations from adjacent sites does not, by itself, demonstrate the absence of liquefaction hazards, except in those unusual cases where a combination of uniformity of local geology and very low regional water tables permits very conservative assessment of water table depths. Screening investigations should also address the possibility of local "perched" water tables, the raising of water levels by septic systems, or the presence of locally saturated soil units at a proposed project site.

- **Is the geometry of potentially liquefiable deposits such that they pose significant risks requiring further investigation, or might they be mitigated by relatively inexpensive foundation strengthening?**

  Relatively thin seams of liquefiable soils (on the order of only a few centimeters thick), if laterally continuous over sufficient area, can represent potentially hazardous planes of weakness and sliding, and may thus pose a hazard with respect to lateral spreading and related ground displacements. Thus, the screening investigation should identify nearby free faces (cut slopes, stream banks, and shoreline areas), whether on or off-site, to determine whether lateral spreading and related ground displacements might pose a hazard to the project. If such features are found, the quantitative evaluation of liquefaction usually will be warranted because of potential life-safety concerns.

  Even when it is not possible to demonstrate the absence of potentially liquefiable soils or prove that such soils are not and will not become saturated, it may be possible to demonstrate that any potential liquefaction hazards can be adequately mitigated through a simple strengthening of the foundation of the structure, as described in the mitigation section of this chapter, or other appropriate methods.

- **Are in-situ soil densities sufficiently high to preclude liquefaction?**

If the screening evaluation indicates the presence of potentially liquefiable soils, either in a saturated condition or in a location which might subsequently become saturated, then the resistance of these soils to liquefaction and/or significant loss of strength due to cyclic pore pressure generation under seismic loading should be evaluated. If the screening investigation does not conclusively eliminate the possibility of liquefaction hazards at a proposed project site (a factor of safety of 1.5 or greater), then more extensive studies are necessary.

A number of investigative methods may be used to perform a screening evaluation of the resistance of soils to liquefaction. These methods are somewhat approximate, but in cases wherein liquefaction resistance is very high (e.g., when the soils in question are very dense) then these methods may, by themselves, suffice to adequately demonstrate sufficient level of liquefaction resistance, eliminating the need for further investigation. It is emphasized that the methods described in this section are more approximate than those discussed in the quantitative evaluation section, and so require very conservative application.

Methods that satisfy the requirements of a screening evaluation, at least in some situations, include:

1. Direct in-situ relative density measurements, such as the ASTM D 1586-92 (Standard Penetration Test [SPT]) or ASTM D3441-94 (Cone Penetration Test [CPT]).

2. Preliminary analysis of hydrologic conditions (e.g., current, historical and potential future depth(s) to subsurface water). Current groundwater level data, including perched water tables, may be obtained from permanent wells, driller's logs and exploratory borings. Historical groundwater data can be found in reports by various government agencies, although such reports often provide information only on water from production zones and ignore shallower water.

3. Non-standard penetration test data. It should be noted that correlation of non-standard penetration test results (e.g., sampler size, hammer weight/drop, hollow stem auger) with SPT resistance is very approximate, and so requires very conservative interpretation, unless direct SPT and non-standard test comparisons are made at the site and in the materials of interest.

4. Geophysical measurements of shear-wave velocities.

5. "Threshold strain" techniques represent a conservative basis for screening of some soils and some sites (National Research Council, 1985). These methods provide only a very conservative bound for such screening, however, and so are conclusive only for sites where the potential for liquefaction hazards is very low.

Quantitative Evaluation of Liquefaction Resistance

Liquefaction investigations are best performed as part of a comprehensive investigation. These Guidelines are to promote uniform evaluation of the resistance of soil to liquefaction.

Detailed Field Investigation

Engineering Geologic Investigations

Engineering geologic investigations should determine:

1. The presence, texture (e.g., grain size), and distribution (including depth) of unconsolidated deposits;

2. The age of unconsolidated deposits, especially for Quaternary Period units (both Pleistocene and Holocene Epochs);

3. Zones of flooding or historic liquefaction; and,

4. The groundwater level to be used in the liquefaction analysis, based on data from well logs, boreholes, monitoring wells, geophysical investigations, or available maps. Generally, the historic high groundwater level should be used unless other information indicates a higher or lower level is appropriate.

The engineering geologic investigations should reflect relative age, soil classification, three-dimensional distribution and general nature of exposures of earth materials within the area. Surficial deposits should be described as to general characteristics (including environment of deposition) and their relationship to present topography and drainage. It may be necessary to extend the mapping into adjacent areas. Geologic cross sections should be constrained by boreholes and/or trenches when available.

Geotechnical Field Investigation

The vast majority of liquefaction hazards are associated with sandy and/or silty soils. For such soil types, there are at present two approaches available for quantitative evaluation of the soil's resistance to liquefaction. These are: (1) correlation and analyses based on in-situ Standard Penetration Test (SPT) (ASTM D1586-92) data, and (2) correlation and analyses based on in-situ Cone Penetration Test (CPT) (ASTM D3441-94) data. Both of these methods have some relative advantages (see Table 4). Either of these methods can suffice by itself for some site conditions, but there is also considerable advantage to using them jointly.

Seed and others (1985) provide guidelines for performing "standardized" SPT, and also provide correlations for conversion of penetration resistance obtained using most of the common alternate combinations of equipment and procedures in order to develop equivalent "standardized" penetration resistance values — $(N_1)_{60}$. These "standardized" penetration resistance values can then be used as a basis for evaluating liquefaction resistance.

Table 4. Comparative advantages of SPT and CPT methods.	
SPT ADVANTAGES	CPT ADVANTAGES
1. Retrieves a sample. This permits identification of soil type ***with certainty***, and permits evaluation of fines content (which influences liquefaction resistance). Note that CPT provides poor resolution with respect to soil classification, and so usually requires some complementary borings with samples to more reliably define soil types and stratigraphy.	1. Provides ***continuous*** penetration resistance data, as opposed to averaged data over discrete increments (as with SPT), and so is less likely to "miss" thin layers and seams of liquefiable material.
2. Liquefaction resistance correlation is based primarily on field case histories, and the vast majority of the field case history database is for in-situ SPT data	2. Faster and less expensive than SPT, as no borehole is required.

Cone penetration test (CPT) tip resistance (q_c) may also be used as a basis for evaluation of liquefaction resistance, by either (a) direct empirical comparison between qc data and case histories of seismic performance (Olsen, 1988), or (b) conversion of q_c-values to "equivalent" $(N_1)_{60}$-values and use of correlations between $(N_1)_{60}$ data and case histories of seismic performance. At present, Method (b) — conversion of qc to equivalent $(N_1)_{60}$— is preferred because the field case history data base for SPT is well-developed compared to CPT correlations. A number of suitable correlations between qc and $(N_1)_{60}$ are available (e.g., Robertson and Campanella, 1985; Seed and De Alba, 1986). These types of conversion correlations depend to some extent on knowledge of soil characteristics (e.g., soil type, mean particle size (D_{50}), fines content). When the needed soil characteristics are either unknown or poorly defined, then it should be assumed that the ratio

$$\frac{q_c(kg/cm^2)}{N(blows/ft)}$$

is approximately equal to five for conversion from q_c to "equivalent" N-values.

Geotechnical Laboratory Testing

The use of laboratory testing (e.g., cyclic triaxial, cyclic simple shear, cyclic torsional tests) on "undisturbed" soil samples as the sole basis for the evaluation of in-situ liquefaction resistance is not recommended, as unavoidable sample disturbance and/or sample densification during reconsolidation prior to undrained cyclic shearing causes a largely unpredictable, and typically unconservative, bias to such test results. Laboratory testing is recommended for determining grain-size distribution (including mean grain size D_{50}, effective grain size D_{10}, and percent passing #200 sieve), unit weights, moisture contents, void ratios, and relative density.

In addition to sandy and silty soils, some gravelly soils are potentially vulnerable to liquefaction (Evans and Fragasy, 1995, Evans and Zhou, 1995). Most gravelly soils drain relatively well, but when their voids are filled with finer particles, or they are surrounded (or "capped") by less pervious soils, drainage can be impeded and they may be vulnerable to liquefaction. Gravelly soils tend to be deposited in a more turbulent environment than sands or silts, and are fairly dense, and so are generally resistant to liquefaction. Accordingly, conservative "preliminary evaluation" methods (e.g., geologic assessments and/or shear-wave velocity measurements) often suffice for evaluation of their liquefaction potential. When preliminary evaluation does not suffice, more accurate quantitative methods must be used. Unfortunately, neither SPT nor CPT provides reliable penetration resistance data in soils with high gravel content, as the large particles impede these small-diameter penetrometers. At present, the best available technique for quantitative evaluation of the liquefaction resistance of coarse, gravelly soils involves correlations and analyses based on in-situ penetration resistance measurements using the very large-scale Becker-type Hammer system (Harder, 1988).

Evaluation of Potential Liquefaction Hazards

The factor of safety for liquefaction resistance has been defined:

$$\text{Factor of Safety} = \frac{CSR_{liq}}{CSR_{eq}}$$

where CSR_{eq} is the cyclic stress ratio generated by the anticipated earthquake ground motions at the site, and CSR_{liq} is the cyclic stress ratio required to generate liquefaction (Seed and Idriss, 1982). For the purposes of evaluating the results of a quantitative assessment of liquefaction potential at a site, a factor of safety against the occurrence of liquefaction greater than about 1.3 can be considered an acceptable level of risk. This factor of safety assumes that high-quality, site-specific penetration resistance and geotechnical laboratory data were collected, and that ground-motion data from DMG (Petersen and others, 1996) were used in the analyses. If lower factors of safety are calculated for some soil zones, then an evaluation of the level (or severity) of the hazard associated with potential liquefaction of these soils should be made.

Such hazard assessment requires considerable engineering judgment. The following is, therefore, only a guide. The assessment of hazard associated with potential liquefaction of soil deposits at a site must consider two basic types of hazard:

1. Translational site instability (sliding, edge failure, lateral spreading, flow failure, etc.) that potentially may affect all or large portions of the site; and

2. More localized hazard at and immediately adjacent to the structures and/or facilities of concern (e.g., bearing failure, settlement, localized lateral movements)

As Bartlett and Youd (1995) have stated: "Two general questions must be answered when evaluating the liquefaction hazards for a given site:

(1) ‘Are the sediments susceptible to liquefaction?' and

(2) ‘If liquefaction does occur, what will be the ensuing amount of ground deformation?'"

Lateral Spreading and Site Displacement Hazards

Lateral spreading on gently sloping ground generally is the most pervasive and damaging type of liquefaction failure (Bartlett and Youd, 1995). Assessment of the potential for lateral spreading and other large site displacement hazards may involve the need to determine the residual undrained strengths of potentially liquefiable soils. If required, this should be done using in-situ SPT or CPT test data (e.g., Seed and Harder, 1990). The use of laboratory testing for this purpose is not recommended, as a number of factors (e.g., sample disturbance, sample densification during reconsolidation prior to undrained shearing, and void ratio redistribution) render laboratory testing a potentially unreliable, and, therefore, unconservative basis for assessment of in-situ residual undrained strengths. Assessment of residual strengths of silty or clayey soils may, however, be based on laboratory testing of "undisturbed" samples.

Assessment of potential lateral spread hazards must consider dynamic loading as a potential "driving" force, in addition to gravitational forces. It should again be noted, that relatively thin seams of liquefiable material, if fairly continuous over large lateral areas, may serve as significant planes of weakness for translational movements. If prevention of translation or lateral spreading is ascribed to structures providing "edge containment," then the ability of these structures (e.g., berms, dikes, sea walls) to resist failure must also be assessed. Special care should be taken in assessing the containment capabilities of structures prone to potentially "brittle" modes of failure (e.g., brittle walls which may break, tiebacks which may fail in tension). If a hazard associated with potentially large translational movements is found to exist, then either: (a) suitable recommendations for mitigation of this hazard should be developed, or (b) the proposed "project" should be discontinued.

When suitably sound lateral containment is demonstrated to prevent potential sliding on liquefied layers, then potentially liquefiable zones of finite thickness occurring at depth may be deemed to pose no significant risk beyond the previously defined minimum acceptable level of risk. Suitable criteria upon which to base such an assessment include those proposed by Ishihara (1985, Figure 88; 1996, Chapter 16).

For information on empirical models that might be appropriate to use in these analyses, see Bartlett and Youd (1995).

Localized Liquefaction Hazards

If it can be shown that no significant risk of large translational movements exists, or if suitable mitigation measures can be developed that address such risks, then studies should proceed to consideration of five general types of more localized potential hazards, including:

1. ***Potential foundation bearing failure, or large foundation settlements due to ground softening and near-failure in bearing***. To form a basis for concluding that no hazard exists, a high factor of safety (FS > 1.5) should be based on a realistic appraisal of the minimum soil strengths likely to be mobilized to resist bearing failure (including residual undrained strengths of soils considered likely to liquefy or to suffer significant strength loss due to cyclic pore pressure generation). If such hazard does exist, then appropriate recommendations for mitigation of this hazard should be developed.

2. ***Potential structural and/or site settlements***. Settlements for saturated and unsaturated clean sands can be estimated using simplified empirical procedures (e.g., Tokimatsu and Seed, 1987; Ishihara and Yoshmine, 1992). These procedures, developed for relatively clean, sandy soils, have been found to provide reasonably reliable settlement estimates for sites not prone to significant lateral spreading.

   Any prediction of liquefaction-related settlements is necessarily approximate, and related hazard assessment and/or development of recommendations for mitigation of such hazard should, accordingly, be performed with suitable conservatism. Similarly, it is very difficult to reliably estimate the amount of localized differential settlement likely to occur as part of the overall predicted settlement: localized ***differential*** settlements on the order of up to two-thirds of the total settlements anticipated should be assumed unless more precise predictions of differential settlements can be made.

3. ***Localized lateral displacement; "lateral spreading" and/or lateral compression.*** Methods for prediction of lateral ground displacements due to liquefaction-related ground softening are not yet well supported by data from case histories of field performance. As such case history data are now being developed, significant advances in the reliability and utility of techniques for prediction of lateral displacements may be expected over the next few years. Finite element models represent the most sophisticated method currently in use for calculating permanent displacements due to liquefaction lateral spreading. Like the dynamic analysis for landslide displacements, this method evaluates time histories of the stresses and strains for a strong-motion time history. This method is a state-of-the-art approach to liquefaction hazards and will likely take time to become the state-of-the-practice.

   Consultants performing liquefaction hazard assessment should do their best to keep abreast of such developments. At present, lateral ground displacement magnitudes can be predicted with reasonable accuracy and reliability only for cases wherein such displacements are likely to be "small" (e.g., on the order of 15 cm or less). Larger displacements may be predicted with an accuracy of + one meter or more; this level of accuracy may suffice for design of some structures (e.g., earth and rock-fill dams), but does not represent a sufficiently refined level of accuracy as to be of use for design of foundations for most types of structures.

It may be possible to demonstrate that localized lateral displacements will be 0.5 meter or less based on: (a) evaluation of soil stratigraphy, residual undrained strengths, and duration and severity of seismic loading, or (b) simplified empirical methods. Bartlett and Youd's (1995) empirical procedure uses an existing field case history database of lateral spread occurrences. Other empirical methods or more complex analyses, may yield somewhat different results but should be allowed if the methods are documented and the results justified. When likely maximum lateral displacements can be shown to be less than 0.5 meter (e.g., Bartlett and Youd, 1995), it may be possible to design foundations with sufficient strength to withstand the expected movements without complete failure. In all other cases, more extensive recommendations are needed for mitigation of the hazard associated with potential lateral displacements.

4. ***Floatation of light structures with basements, or underground storage structures***. Light structures which extend below the groundwater table and contain large void spaces may "float" or rise out of the ground during, or soon after an earthquake. Structures that are designed for shallow groundwater conditions typically rely on elements, such as cantilevered walls or tie-downs, that resist the buoyant or uplift forces produced by the water. If the material surrounding these elements liquefies, the resisting forces can be significantly reduced and the entire structure may be lifted out of the ground.

5. ***Hazards to Lifelines***. To date, most liquefaction hazard investigations have focused on assessing the risks to commercial buildings, homes, and other occupied structures. However, liquefaction also poses problems for streets and lifelines—problems that may, in turn, jeopardize lives and property. For example, liquefaction locally caused natural gas pipelines to break and catch fire during the Northridge earthquake, and liquefaction-caused water line breakage greatly hampered firefighters in San Francisco following the 1906 earthquake. Thus, although lifelines are not explicitly mentioned in the Seismic Hazards Mapping Act, cities and counties may wish to require investigation and mitigation of potential liquefaction-caused damage to lifelines.

Mitigation of Liquefaction Hazards

The hazard assessment required for project sites within zones of required investigation should (a) demonstrate that liquefaction at a proposed project site poses a sufficiently low hazard as to satisfy the defined acceptable level of risk criteria, or (b) result in implementation of suitable mitigation recommendations to effectively reduce the hazard to acceptable levels (CCR Title 14, Section 3721). Mitigation should provide suitable levels of protection with regard to the two general types of liquefaction hazards previously discussed (1) potential large lateral spread failures, and (2) more localized problems including potential bearing failure, settlements, and lateral displacements.

Potentially suitable methods for mitigation of lateral spread hazards may include the following:

1. Edge containment structures (e.g., berms, dikes, sea walls, retaining structures, compacted soil zones);

2. Removal or treatment of liquefiable soils to reduce liquefaction potential;

3. Modification of site geometry to reduce the risk of translational site instability; and/or

4. Drainage to lower the groundwater table below the level of the liquefiable soils.

Mitigation techniques may be applied individually or in combination. Mitchell and others (1995) summarize the performance of some mitigation techniques for past earthquakes. Hryciw (1995) includes several articles with additional information about the success of specific soil improvement techniques.

Once problems related to potentially large lateral spread failures have been resolved, the remaining "localized" potential hazards should be addressed and resolved. Suitable mitigation alternatives may include one or more of the following:

1. Excavation and removal or recompaction of potentially liquefiable soils;

2. In-situ ground densification (e.g., compaction with vibratory probes, dynamic consolidation, compaction piles, blasting densification, compaction grouting);

3. Other types of ground improvement (e.g., permeation grouting, columnar jet grouting, deep mixing, gravel drains or other drains, surcharge pre-loading, structural fills, dewatering);

4. Deep foundations (e.g., piles, piers), that have been designed to accommodate liquefaction effects;

5. Reinforced shallow foundations (e.g., grade beams, combined footings, reinforced or post-tensioned slabs, rigid raft foundations); and

6. Design of the proposed structures or facilities to withstand predicted ground softening and/or predicted vertical and lateral ground displacements to an acceptable level of risk.

The scope and type(s) of mitigation required depend on the site conditions present and the nature of the proposed project. Individual mitigation techniques may be used, but the most appropriate solution may involve using them in combination.

In general, only removal and/or densification of potentially liquefiable soils, or drainage of groundwater can ***fully*** eliminate all liquefaction hazards. In many cases, other methods may achieve the desired acceptable level of risk. For example, in areas where liquefaction may potentially cause displacements of one-third meter or less, design of the foundation to withstand displacements of one-half meter can significantly reduce future damage from liquefaction. The Northridge earthquake caused liquefaction in a number of locations. Insurers reported that losses equal to two-thirds of the value of damaged structures were not uncommon—structures that took many months, if not years, to again make habitable. Youd (personal communication, 1996) and other engineers indicate that by adding adequate reinforcing steel to properly designed concrete slabs or grade beams to resist fracture during ground displacement (very inexpensive for a single-family dwelling), 80 percent or more of this damage would have been avoided and repairs (patching, re-leveling of homes, etc.) would have been expedited. Such improved foundations will also reduce damage from expansive soils, settling, minor landslide movement, and similar ground-related problems (Federal Emergency Management Agency, in press). Based on these conclusions, the Liquefaction Working Group strongly recommends that, if the consultant determines that the project site will be affected by small lateral spreading, lead agencies should consider waiving detailed site investigations in lieu of foundation and structure designs that safely withstand up to two times the estimated deformations without fracturing the foundation. In the Liquefaction Working Group's opinion, the money required for detailed site investigations in areas not subject to lateral spread displacement would be better spent on mitigation than on investigation. This mitigation measure should provide adequate protection to the structure but will leave buried utilities unprotected and subject to damage, particularly at connections to the improved structures. In zones of required investigation for liquefaction, developers and utility companies should use types of pipe and flexible connections that are resistant to earthquake damage, thereby increasing the likelihood that the utilities will be functional after an earthquake (Federal Emergency Management Agency, in press).

Development of appropriate recommendations for mitigation of liquefaction hazards requires considerable judgment, as does the review and evaluation of such recommendations. Accordingly, the importance of the lead agency technical reviewer is emphasized. Technical reviewers are reminded to consider that the intent of the State's Seismic Hazard Zone program is to provide an adequate minimum level of protection for projects in the zone of required investigation, based on the acceptable level of risk. Owners/developers are, however, also hereby encouraged to implement a higher level of mitigation, in order to protect their investment and/or to minimize their potential future exposure and that of future occupants or users of the project structures or facilities.

CHAPTER 7

GUIDELINES FOR REVIEWING SITE-INVESTIGATION REPORTS

The purpose of this chapter is to provide general guidance to regulatory agencies that have approval authority over projects and to engineering geologists and civil engineers who review reports of seismic hazard investigations. These Guidelines recognize that effective mitigation ultimately depends on the professional judgment and expertise of the developer's engineering geologist and/or civil engineer in concert with the lead agency's engineering geologist and/or civil engineer.

The required technical review is a critical part of the evaluation process of approving a project. The reviewer ensures compliance with existing laws, regulations, ordinances, codes, policies, standards, and good practice, helping to assure that significant geologic factors (hazards and geologic processes) are properly considered, and potential problems are mitigated prior to project development. Under the Seismic Hazards Mapping Act, the reviewer is responsible for determining that each seismic hazard site investigation, and the resulting report, reasonably address the geologic and soil conditions that exist at a given site. The reviewer acts on behalf of a governing agency—city, county, regional, state, or federal—not only to protect the government's interest but also to protect the interest of the community at large. Examples of the review process in a state agency are described by Stewart and others (1976). Review at the local level has been discussed by Leighton (1975), Hart and Williams (1978), Berkland (1992), and Larson (1992). Grading codes, inspections, and the review process are discussed in detail by Scullin (1983).

The Reviewer

Qualifications

CCR Title 14, Section 3724(c) states that the reviewer must be a licensed engineering geologist and/or civil engineer having competence in the field of seismic hazard evaluation and mitigation. California's Business and Professions Code limits the practice of geology and engineering to licensed geologists and engineers, respectively, thereby requiring that reviewers be licensed, or directly supervised by someone who is licensed, by the appropriate State board. Local and regional agencies may have additional requirements. Nothing in these Guidelines is intended to sanction or authorize the review of engineering geology reports by engineers or civil engineering reports by geologists.

The reviewer should be familiar with the investigative methods employed and the techniques available to these professions (see Chapters 3 through 6). The opinions and comments made by the reviewer should be competent, prudent, objective, consistent, unbiased, pragmatic, and reasonable. The reviewer should be professional and ethical. The reviewer should have a clear understanding of the criteria for approving and not approving reports. Reviews should be based on logical, defensible criteria.

Reviewers must recognize their limitations. They should be willing to ask for the opinions of others more qualified in specialty fields.

If there is clear evidence of incompetence or misrepresentation in a report, this fact should be reported to the reviewing agency or licensing board. California Civil Code Section 47 provides an immunity for statements made "in the initiation or course of any other proceedings authorized by law." Courts have interpreted this section as providing immunity to letters of complaint written to provide a public agency or board, including licensing boards, with information that the public board or agency may want to investigate (see *King v. Borges*, 28 Cal. App. 3d 27 [1972]; and *Brody v. Montalbano*, 87 Cal. App 3d 725 [1978]). Clearly, reviewers need to have the support of their agency in order to carry out these duties.

The primary purpose of the review procedure should always be kept in mind: to determine compliance with the regulations, codes, and ordinances that pertain to the development. The reviewer should demand that minimum standards are met. The mark of a good reviewer is the ability to sort out the important from the insignificant, to list appropriate requirements for compliance, and to assist the applicant and their consultants in meeting the regulations without doing the consultant's job.

Conflict of Interest

In cases where reviewers also perform geologic or engineering investigations, they should **never** be placed in the position of reviewing their own report, or that of their own agency or company.

Reviewing Reports

The Report

A report that is incomplete or poorly written should be **not** approved. The report should demonstrate that the project complies with applicable regulations, codes, and ordinances, or local functional equivalents, in order to be approved.

The reviewer performs four principal functions in the technical review:

1. Identify any known potential hazards and impacts that are not addressed in the consultant's report. The reviewer should require investigation of the potential hazards and impacts,

2. Determine that the report contains sufficient data to support and is consistent with the stated conclusions,

3. Determine that the conclusions identify the potential impact of known and reasonable anticipated geologic processes and site conditions during the lifespan of the project; and,

4. Determine that the recommendations are consistent with the conclusions and can reasonably be expected to mitigate those anticipated earthquake-related problems that could have a significant impact on the proposed development. The included recommendations also should address the

need for additional geologic and engineering investigations (including any site inspections to be made as site remediation proceeds).

Report Guidelines and Standards

Investigators may save a great deal of time (and the client's money), and possibly misunderstandings, if they contact the reviewing geologist or engineer at the initiation of the investigation. Reviewers typically are familiar with the local geology and sources of information and may be able to provide additional guidance regarding their agency's expectations and review practices. Guidelines for geologic or geotechnical reports have been prepared by a number of agencies and are available to assist reviewers in their evaluation of reports (for example, DMG Notes 42, 44, 48, and 49). Distribution of copies of written policies and guidelines adopted by the agency, usually alerts the applicants and consultants about procedures, report formats, and levels of investigative detail that will expedite review and approval of the project.

If a reviewer determines that a report is not in compliance with the appropriate requirements, this fact should be stated in the written record. After the reviewer is satisfied that the investigation and resulting conclusions and recommendations are reasonable and meet local requirements, approval of the project should be recommended to the reviewing agency.

Review of Submitted Reports

The review of submitted reports constitutes professional practice and should be conducted as such. The reviewer should study the available data and site conditions in order to determine whether the report is in compliance with local requirements. A field reconnaissance of the site should be conducted, preferably after the review of available stereoscopic aerial photographs, geologic maps, and reports on nearby developments.

For each report reviewed, a clear, concise, and logical written record should be developed. This review record may be as long or short as is necessary, depending upon the complexity of the project, the geology, the engineering analysis, and the quality and completeness of the reports submitted. At a minimum, the record should:

1. Identify the project, pertinent permits, applicant, consultants, reports and plans reviewed,

2. Include a clear statement of the requirements to be met by the parties involved, data required, and the plan, phase, project, or report being approved or denied;

3. Contain summaries of the reviewer's field observations, associated literature and air photo review, and oral communications with the applicant and the consultant; and,

4. Contain copies of any pertinent written correspondence.

5. The reviewer's name and license number(s), with any associated expiration dates.

The report, plans, and review record should be kept in perpetuity to document that compliance with local requirements was achieved and for reference during future development, remodeling, or rebuilding. Such records also can be a valuable resource for land-use planning and real-estate disclosure.

Report Filing Requirements

PRC Section 2697 requires cities and counties to submit one copy of each approved site-investigation report, including mitigation measures, if any, that are to be taken, to the State Geologist within 30 days of report approval. Section 2697 also requires that if a project's approval is not in accordance with the policies and criteria of the State Mining and Geology Board (CCR Title 14, Chapter 2, Division 8, Article 10), the city or county must explain the reasons for the differences in writing to the State Geologist, within 30 days of the project's approval. Reports should be sent to:

California Department of Conservation
Division of Mines and Geology
Attn: Seismic Hazard Reports
801 K Street, MS 12-31
Sacramento, CA 95814-3531

Waivers

PRC Section 2697 and CCR Title 14, Section 3725 outline the process under which lead agencies may determine that information from studies conducted on sites in the immediate vicinity may be used to waive the site-investigation report requirement. CCR Title 14, Section 3725 indicates that when a lead agency determines that "geological and geotechnical conditions at the site are such that public safety is adequately protected and no mitigation is required," it may grant a waiver. CCR Title 14, Section 3725 also requires that such a finding be based on a report presenting evaluations of sites in the immediate vicinity having similar geologic and geotechnical characteristics. Further, Section 3725 stipulates that lead agencies must review waiver requests in the same manner as it reviews site-investigation reports; thus, waiver requests must be reviewed by a licensed engineering geologist and/or civil engineer, competent in the field of seismic hazard evaluation and mitigation. Generally, in addition to the findings of the reports that are presented in support of the waiver request, reviewers should consider:

1. The proximity of the project site to sites previously evaluated;

2. Whether the project sites previously evaluated adequately "surround" the project site to preclude the presence of stream channel deposits, historically higher water table, stream channels and other types of free faces that may present an opportunity for lateral spread failures; and,

3. Whether the supporting reports do, in fact, conclude that no hazard exists.

Waiver Filing Requirements

CCR Title 14, Section 3725 provides that "All such waivers shall be recorded with the county recorder and a separate copy, together with the report and commentary, filed with the State Geologist within 30 days of the waiver." These materials should be sent to:

> California Department of Conservation
> Division of Mines and Geology
> Attn: Seismic Hazard Reports
> 801 K Street, MS 12-31
> Sacramento, CA 95814-3531

Appeals

In cases where the reviewer is not able to approve a site-investigation report, or can accept it only on a conditional basis, the developer may wish to appeal the review decision. However, every effort should be made to resolve problems informally prior to making a formal appeal. Appeal procedures are often specified by a city or county ordinance or similar instrument. An appeal may be handled through existing legal procedures, such as a hearing by a County Board of Supervisors, a City Council, or a specially appointed Technical Appeals and Review Panel. Several administrators note that the Technical Appeals and Review Panel, comprised of geoscientists, engineers, and other appropriate professionals, benefits decision makers by providing additional technical expertise for especially complex and/or controversial cases. Adequate notice should be given to allow time for both sides to prepare their cases. After an appropriate hearing, the appeals decision should be made promptly and in writing as part of the permanent record.

Another way to remedy conflicts between the investigator and the reviewer is by means of a third party review. Such a review can take different paths ranging from the review of existing reports to in-depth field investigations. Third party reviews are usually done by consultants; not normally associated with the reviewing/permitting agency.

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APPENDIX A

SEISMIC HAZARDS MAPPING ACT

CALIFORNIA PUBLIC RESOURCES CODE

Division 2. Geology, Mines and Mining

CHAPTER 7.8. SEISMIC HAZARDS MAPPING

2690. This chapter shall be known and may be cited as the Seismic Hazards Mapping Act.

2691. The Legislature finds and declares all of the following:

(a) The effects of strong ground shaking, liquefaction, landslides, or other ground failure account for approximately 95 percent of economic losses caused by an earthquake.

(b) Areas subject to these processes during an earthquake have not been identified or mapped statewide, despite the fact that scientific techniques are available to do so.

(c) It is necessary to identify and map seismic hazard zones in order for cities and counties to adequately prepare the safety element of their general plans and to encourage land use management policies and regulations to reduce and mitigate those hazards to protect public health and safety.

2692.

(a) It is the intent of the Legislature to provide for a statewide seismic hazard mapping and technical advisory program to assist cities and counties in fulfilling their responsibilities for protecting the public health and safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure and other seismic hazards caused by earthquakes.

(b) It is further the intent of the Legislature that maps and accompanying information provided pursuant to this chapter be made available to local governments for planning and development purposes.

(c) It is further the intent of the Legislature that the Division of Mines and Geology, in implementing this chapter, shall, to the extent possible, coordinate its activities with, and use existing information generated from, the earthquake fault zones mapping program pursuant to Chapter 7.5 (commencing with Section 2621), the landslide hazard identification program pursuant to Chapter 7.7 (commencing with Section 2670), and the inundation maps prepared pursuant to Section 8589.5 of the Government Code.

2692.1. The State Geologist may include in maps compiled pursuant to this chapter information on the potential effects of tsunami and seiche when information becomes available from other sources and the State Geologist determines the information is appropriate for use by local government. The State Geologist shall not be required to provide this information unless additional funding is provided both to make the determination and to distribute the tsunami and seiche information.

2693. As used in this chapter:

(a) "City" and "County" includes the City and County of San Francisco.

(b) "Geotechnical" report means a report prepared by a certified engineering geologist or a civil engineer practicing within the area of his or her competence, which identifies seismic hazards and recommends mitigation measures to reduce the risk of seismic hazard to acceptable levels.

(c) "Mitigation" means those measures that are consistent with established practice and that will reduce seismic risk to acceptable levels.

(d) "Project" has the same meaning as in Chapter 7.5 (commencing with Section 2621),except as follows:

- A single-family dwelling otherwise qualifying as a project may be exempted by the city or county having jurisdiction of the project.

- "Project" does not include alterations or additions to any structure within a seismic hazard zone which do not exceed either 50 percent of the value of the structure or 50 percent of the existing floor area of the structure.

(e) "Commission" means the Seismic Safety Commission.

(f) "Board" means the State Mining and Geology Board.

2694.

(a) A person who is acting as an agent for a seller of real property that is located within a seismic hazard zone, as designated under this chapter, or the seller, if he or she is acting without an agent, shall disclose to any prospective purchaser the fact that the property is located within a seismic hazard zone, if the maps prepared pursuant to this chapter or the information contained in the maps are reasonably available.

(b) In all transactions that are subject to Section 1102 of the Civil Code, the disclosure required by subdivision (a) of this section shall be provided by either of the following means:
- The Local Option Real Estate Transfer Disclosure Statement as provided in Section 1102.6a of the Civil Code.
- The Natural Hazard Disclosure Statement as provided in Section 1102.6c of the Civil Code.

(c) Disclosure is required pursuant to this section only when one of the following conditions is met:
  - The seller, or seller's agent, has actual knowledge that the property is within a seismic hazard zone.
  - A map that includes the property has been provided to the city or county pursuant to Section 2622,and a notice has been posted at the offices of the county recorder, county assessor, and county planning agency that identifies the location of the map and any information regarding changes to the map received by the county.

(d) If the map or accompanying information is not of sufficient accuracy or scale that a reasonable person can determine if the subject real property is included in a seismic hazard zone, the agent shall mark "Yes" on the Natural Hazard Disclosure Statement. The agent may mark "No"on the Natural Hazard Disclosure Statement if he or she attaches a report prepared pursuant to subdivision (c) of Section 1102.4 of the Civil Code that verifies the property is not in the hazard zone. Nothing in this subdivision is intended to limit or abridge any existing duty of the seller or the seller's agents to exercise reasonable care in making a determination under this subdivision.

(e) For purposes of the disclosures required by this section, the following persons shall not be deemed agents of the seller:
  (a) Persons specified in Section 1102.11 of the Civil Code.
  (b) Persons acting under a power of sale regulated by Section 2924 of the Civil Code.

(f) For purposes of this section, Section 1102.13 of the Civil Code applies.

(g) The specification of items for disclosure in this section does not limit or abridge any obligation for disclosure created by any other provision of law or that may exist in order to avoid fraud, misrepresentation, or deceit in the transfer transaction.

2695.

(a) On or before January 1, 1992, the board, in consultation with the director and the commission, shall develop all of the following:
  (1) Guidelines for the preparation of maps of seismic hazard zones in the state.

  (2) Priorities for mapping of seismic hazard zones. In setting priorities, the board shall take into account the following factors:

    - The population affected by the seismic hazard in the event of an earthquake.
    - The probability that the seismic hazard would threaten public health and safety in the event of an earthquake.
    - The willingness of lead agencies and other public agencies to share the cost of mapping within their jurisdiction.
    - The availability of existing information.

(3) Policies and criteria regarding the responsibilities of cities, counties, and state agencies pursuant to this chapter. The policies and criteria shall address, but not be limited to, the following:

(4)
- Criteria for approval of a project within a seismic hazard zone, including mitigation measures.
- The contents of the geotechnical report.
- Evaluation of the geotechnical report by the lead agency.

(5) Guidelines for evaluating seismic hazards and recommending mitigation measures.

(6) Any necessary procedures, including, but not limited to, processing of waivers pursuant to Section 2697, to facilitate the implementation of this chapter.

(b) In developing the policies and criteria pursuant to subdivision (a), the board shall consult with and consider the recommendations of an advisory committee, appointed by the board in consultation with the commission, composed of the following members:

(1) An engineering geologist registered in the state.
(2) A seismologist.
(3) A civil engineer registered in the state.
(4) A structural engineer registered in the state.
(5) A representative of city government, selected from a list submitted by the League of California Cities.
(6) A representative of county government, selected from a list submitted by the County Supervisors Association of California.
- A representative of regional government, selected from a list submitted by the Council of Governments.
- A representative of the insurance industry.
- The Insurance Commissioner

(c) All of the members of the advisory committee shall have expertise in the field of seismic hazards or seismic safety.

(d) At least 90 days prior to adopting measures pursuant to this section, the board shall transmit or cause to be transmitted a draft of those measures to affected cities, counties, and state agencies for review and comment.

2696.

(a) The State Geologist shall compile maps identifying seismic hazard zones, consistent with the requirements of Section 2695. The maps shall be compiled in accordance with a time schedule developed by the director and based upon the provisions of Section 2695 and the level of funding available to implement this chapter.

(b) The State Geologist shall, upon completion, submit seismic hazard maps compiled pursuant to subdivision (a) to the board and all affected cities, counties, and state agencies for review and comment. Concerned jurisdictions and agencies shall submit all comments to the board for review and consideration within 90 days. Within 90 days of board review, the State Geologist shall revise the maps, as appropriate, and shall provide copies of the official maps to each state agency, city, or county, including the county recorder, having jurisdiction over lands containing an area of seismic hazard. The county recorder shall record all information transmitted as part of the public record.

(c) In order to ensure that sellers of real property and their agents are adequately informed, any county that receives an official map pursuant to this section shall post a notice within five days of receipt of the map at the office of the county recorder, county assessor, and county planning agency, identifying the location of the map and any information regarding changes to the map and the effective date of the notice.

2697.

(a) Cities and counties shall require, prior to the approval of a project located in a seismic hazard zone, a geotechnical report defining and delineating any seismic hazard. If the city or county finds that no undue hazard of this kind exists, based on information resulting from studies conducted on sites in the immediate vicinity of the project and of similar soil composition to the project site, the geotechnical report may be waived. After a report has been approved or a waiver granted, subsequent geotechnical reports shall not be required, provided that new geologic datum, or data, warranting further investigation is not recorded. Each city and county shall submit one copy of each approved geotechnical report, including the mitigation measures, if any, that are to be taken, to the State Geologist within 30 days of its approval of the report.

(b) In meeting the requirements of this section, cities and counties shall consider the policies and criteria established pursuant to this chapter. If a project's approval is not in accordance with the policies and criteria, the city or county shall explain the reasons for the differences in writing to the State Geologist, within 30 days of the project's approval.

2698.

Nothing in this chapter is intended to prevent cities and counties from establishing policies and criteria which are more strict than those established by the board.

2699.

Each city and county, in preparing the safety element to its general plan pursuant to subdivision (g) of Section 65302 of the Government Code, and in adopting or revising land use planning and permitting ordinances, shall take into account the information provided in available seismic hazard maps.

2699.5

There is hereby created the Seismic Hazards Identification Fund, as a special fund in the State Treasury. Notwithstanding Section 13340 of the Government Code, the moneys in the fund are continuously appropriated to the division for the purposes of this chapter. Notwithstanding Section 5001 of the Insurance Code, one-half of 1 percent of the earthquake surcharge moneys received by the California Residential Earthquake Recovery Fund in any calendar year shall be transferred to the Seismic Hazards Identification Fund for the purposes of carrying out this chapter. This subdivision shall become operative only if Assembly Bill 3913 or Senate Bill 2902 of the 1989-90 Regular Session of the Legislature is enacted and takes effect.

2699.6.
This chapter shall become operative on April 1, 1991.

APPENDIX B

California Code of Regulations
Title 14. Natural Resources
Division 2. Department of Conservation
Chapter 8. Mining and Geology
Article 10. Seismic Hazards Mapping

3720. Purpose

These regulations shall govern the exercise of city, county and state agency responsibilities to identify and map seismic hazard zones and to mitigate seismic hazards to protect public health and safety in accordance with the provisions of Public Resources Code, Section 2690 et seq. (Seismic Hazards Mapping Act).

Authority cited: Public Resources Code Section 2695
Reference: Public Resources Code Section 2695(a)(1)and (3)-(5)

3721. Definitions

(a) "Acceptable Level" means that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project.

(b) "Lead Agency" means the city, county or state agency with the authority to approve projects.

(c) "Registered civil engineer" or "certified engineering geologist" means a civil engineer or engineering geologist who is registered or certified in the State of California.

Authority cited: Public Resources Code Section 2695
Reference: Public Resources Code Sections 2690-2696.6

3722. Requirements for Mapping Seismic Hazard Zones

(a) The Department of Conservation, Division of Mines and Geology, shall prepare one or more State-wide probabilistic ground shaking maps for a suitably defined reference soil column. One of the maps shall show ground shaking levels which have a 10% probability of being exceeded in 50 years. These maps shall be used with the following criteria to define seismic hazard zones:

(1) Amplified shaking hazard zones shall be delineated as areas where historic occurrence of amplified ground shaking, or local geological and geotechnical conditions indicate a potential for ground shaking to be amplified to a level such that mitigation as defined in Public Resources Code Section 2693(c)would be required.

(2) Liquefaction hazard zones shall be delineated as areas where historic occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

(3) Earthquake-induced landslide hazard zones shall be delineated as areas where Holocene occurrence of landslide movement, or local slope of terrain, and geological, geotechnical and ground moisture conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

(b) Highest priority for mapping seismic hazard zones shall be given to areas facing urbanization or redevelopment in conjunction with the factors listed in Section 2695(a)(2)(A), (B), (C) and (D) of the Public Resources Code.

Authority cited: Public Resources Code Section 2695
Reference: Public Resources Code Section 2695(a)(1)

3723. Review of Preliminary Seismic Hazard Zones Maps

(a) The Mining and Geology Board shall provide an opportunity for receipt of public comments and recommendations during the 90-day period for review of preliminary seismic hazard zone maps provided by the Public Resources Code Section 2696. At least one public hearing shall be scheduled for that purpose.

(b) Following the end of the review period, the Board shall forward its comments and recommendations, with supporting data received, to the State Geologist for consideration prior to revision and official issuance of the maps.

Authority cited: Public Resources Code Section 2696
Reference: Public Resources Code Section 2696

3724. Specific Criteria for Project Approval

The following specific criteria for project approval shall apply within seismic hazard zones and shall be used by affected lead agencies in complying with the provisions of the Act:

(a) A project shall be approved only when the nature and severity of the seismic hazards at the site have been evaluated in a geotechnical report and appropriate mitigation measures have been proposed.

(b) The geotechnical report shall be prepared by a registered civil engineer or certified engineering geologist, having competence in the field of seismic hazard evaluation and mitigation. The geotechnical report shall contain site-specific evaluations of the seismic hazard affecting the project, and shall identify portions of the project site containing seismic hazards. The report shall also identify any known off-site seismic hazards that could adversely affect the site in the event of an earthquake. The contents of the geotechnical report shall include, but shall not be limited to, the following:

(1) Project description.

(2) A description of the geologic and geotechnical conditions at the site, including an appropriate site location map.

(3) Evaluation of site-specific seismic hazards based on geological and geotechnical conditions, in accordance with current standards of practice.

(4) Recommendations for appropriate mitigation measures as required in Section 3724(a), above.

(5) Name of report preparer(s), and signature(s) of a certified engineering geologist and/or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation.

(c) Prior to approving the project, the lead agency shall independently review e geotechnical report to determine the adequacy of the hazard evaluation and proposed mitigation measures and to determine the requirements of Section 3724(a), above, are satisfied. Such reviews shall be conducted by a certified engineering geologist or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation.

Authority cited: Public Resources Code Section 2695
Reference: Public Resources Code Section 2695(a)(3)(A), (B), and (C)

3725. Waivers of Geotechnical Report Requirements

For a specific project, the lead agency may determine that the geological and geotechnical conditions at the site are such that public safety is adequately protected and no mitigation is required. This finding shall be based on a report presenting evaluations of sites in the immediate vicinity having similar geologic and geotechnical characteristics. The report shall be prepared by a certified engineering geologist or register civil engineer, having competence in the field of seismic hazard evaluation and mitigation. The lead agency shall review submitted reports in the same manner as in Section 3724(c) of this article. The shall also provide a written commentary that addresses the report conclusions and the justification for applying the conclusions contains in the report to the project site. When the lead agency makes such a finding, it may waive the requirement of a geotechnical report for the project. All such waivers shall be recorded with the county recorder and a separate copy, together with the report and commentary, filed with the State Geologist within 30 days of the waiver.

Authority cited: Public Resources Code Section 2695
Reference: Public Resources Code Section 2697(a)(5)

APPENDIX C

TECHNICAL TERMS AND DEFINITIONS

ASTM	American Society for Testing and Materials
CPT	Cone Penetration Test (ASTM D3441-94).
CSR	Cyclic stress ratio— a normalized measure of cyclic load severity, expressed as equivalent uniform cyclic deviatoric load divided by some measure of initial effective overburden or confining stress.
CSR_{eq}	The equivalent uniform cyclic stress ratio representative of the dynamic loading imposed by an earthquake.
CSR_{liq}	The equivalent uniform cyclic stress ratio required to induce liquefaction within a given number of loading cycles [that number of cycles considered representative of the earthquake under consideration].
DSHA	Deterministic seismic hazard analysis
FS	Factor of safety— the ratio of the forces available to resist failure divided by the driving forces.
Ground Loss	Localized ground subsidence.
k	Seismic coefficient used in a pseudo-static slope stability analysis
Liquefaction	Significant loss of soil strength due to pore pressure increase.
N	Penetration resistance measured in SPT tests (blows/ft).
N_1	Normalized SPT N-value (blows/ft); corrected for overburden stress effects to the N-value which would occur if the effective overburden stress was 1.0 tons/ft^2.
$(N_1)_{60}$	Standardized, normalized SPT-value; corrected for both overburden stress effects and equipment and procedural effects (blows/ft).
PI	Plasticity Index; the difference between the Atterberg Liquid Limit (LL) and the Atterberg Plastic Limit (PL) for a cohesive soil. [PI(%) = LL(%) - PL(%)].
PSHA	Probabilistic seismic hazard analysis
q_c	Tip resistance measured by CPT probe (force/length^2).
$q_{c,1}$	Normalized CPT tip resistance (force/length^2); corrected for overburden stress effects to the qc value which would occur if the effective overburden stress was 1.0 tons/ ft^2.
Sand Boiling	Localized ejection of soil and water to relieve excess pore pressure.
SPPV	Simple prescribed parameter values
SPT	Standard Penetration Test (ASTM D1586-92).
UBC	The Uniform Building Code, published by the International Conference of Building Officials (ICBO, 1997), periodically updated.

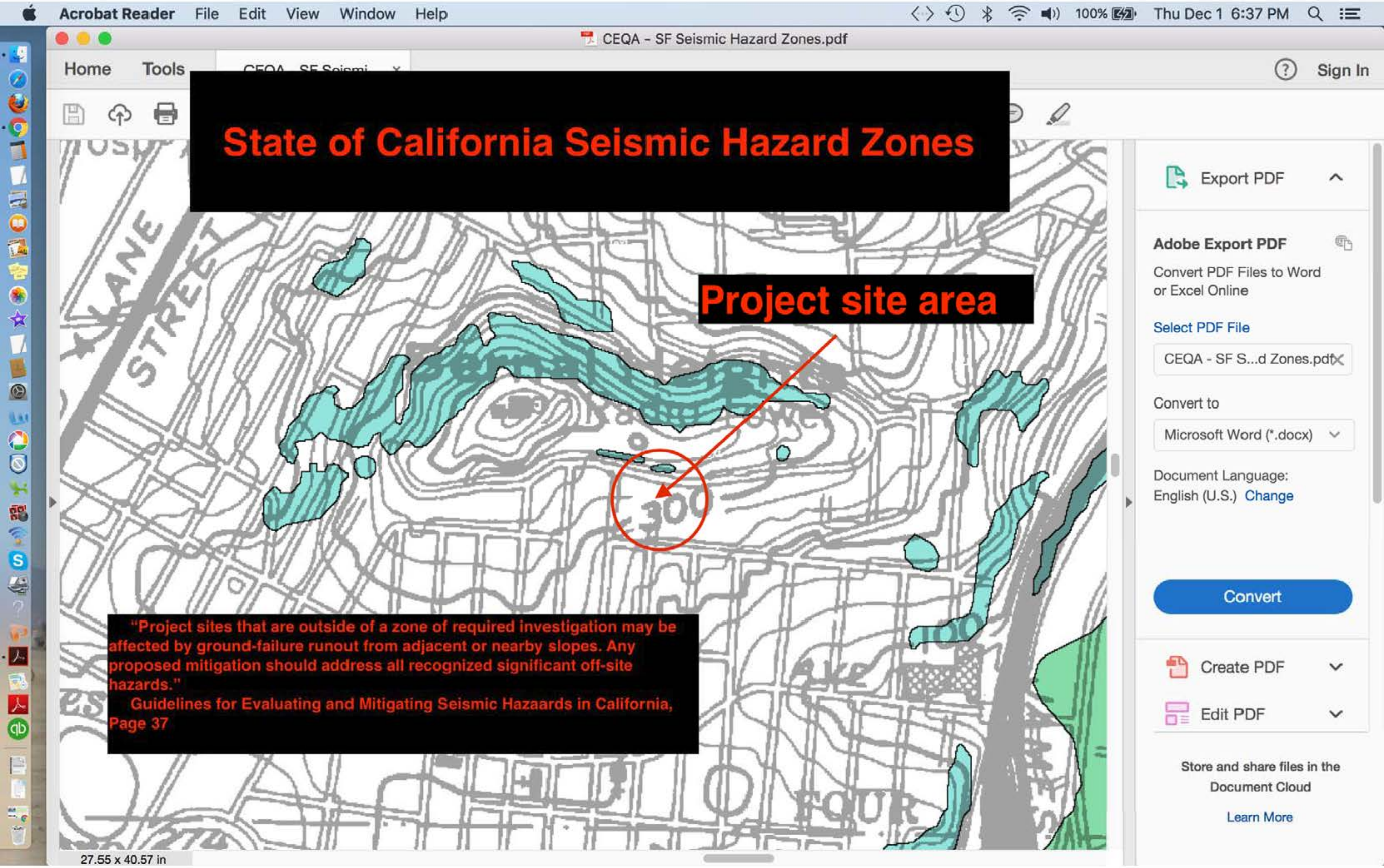
APPENDIX D

GEOLOGIC ENVIRONMENTS LIKELY TO PRODUCE EARTHQUAKE-INDUCED LANDSLIDES

Landslide Type	Type of Material	Minimum Slope	Remarks
Rock falls	Rocks weakly cemented, intensely fractured, or weathered; contain conspicuous planes of weakness dipping out of slope or contain boulders in a weak matrix.	40° 1.7:1	Particularly common near ridge crests and on spurs, ledges, artificially cut slopes, and slopes undercut by active erosion.
Rock slides	Rocks weakly cemented, intensely fractured, or weathered; contain conspicuous planes of weakness dipping out of slope or contain boulders in a weak matrix.	35° 1.4:1	Particularly common in hillside flutes and channels, on artificially cut slopes, and on slopes undercut by active erosion. Occasionally reactivate preexisting rock slide deposits.
Rock Avalanches	Rocks intensely fractured and exhibiting one of the following properties: significant weathering, planes of weakness dipping out of slope, weak cementation, or evidence of previous landsliding.	25° 2.1:1	Usually restricted to slopes of greater than 500 feet (150 m) relief that have been undercut by erosion. May be accompanied by a blast of air that can knock down trees and structures beyond the limits of the deposited debris
Rock slumps	Intensely fractured rocks, preexisting rock slump deposits, shale, and other rocks containing layers of weakly cemented or intensely weathered material.	15° 3.7:1	
Rock block slides	Rocks having conspicuous bedding planes or similar planes of weakness dipping out of slopes.	15° 3.7:1	
Soil falls	Granular soils that are slightly cemented or contain clay binder	40° 1.7:1	Particularly common on stream-banks, terrace faces, coastal bluffs, and artificially cut slopes.
Disrupted soil slides	Loose, unsaturated sands.	15° 3.7:1	
Soil avalanches	Loose, unsaturated sands.	25° 2.1:1	Occasionally reactivate preexisting soil avalanche deposits.
Soil slumps	Loose, partly to completely saturated sand or silt; uncompacted or poorly compacted manmade fill composed of sand, silt, or clay, preexisting soil slump deposits.	10° 11:1	Particularly common on embankments built on soft, saturated foundation materials, in hillside cut-and-fill areas, and on river and coastal flood plains.
Soil block slumps	Loose, partly or completely saturated sand or silt; uncompacted or slightly compacted manmade fill composed of sand or silt, bluffs containing horizontal or subhorizontal layers or loose, saturated sand or silt.	5° 11:1	Particularly common in areas of preexisting landslides along river and coastal flood plains, and on embankments built of soft, saturated foundation materials.
Slow earth	Stiff, partly to completely saturated clay	10°	

Landslide Type	Type of Material	Minimum Slope	Remarks
flows	and preexisting earth-flow deposits.	5.7:1	
Soil lateral spreads	Loose, partly or completely saturated silt or sand, uncompacted or slightly compacted manmade fill composed of sand.	0.3° 190:1	Particularly common on river and coastal flood plains, embankments built on soft, saturated foundation materials, delta margins, sand dunes, sand spits, alluvial fans, lake shores and beaches.
Rapid soil flow	Saturated, uncompacted or slightly compacted manmade fill composed of sand or sandy silt (including hydraulic fill earth dams and tailings dams); loose, saturated granular soils.	2.3° 25:1	Includes debris flows that typically originate in hollows at heads of streams and adjacent hillsides; typically travel at tens of miles per hour or more and may cause damage miles from the source area.
Subaqueous landslides	Loose, saturated granular soils.	0.5° 110:1	Particularly common on delta margins.

Modified from Keefer (1984).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CEQA - SF Seismic Hazard Zones.pdf
Home Tools
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State of California Seismic Hazard Zones
Project site area
"Project sites that are outside of a zone of required investigation may be affected by ground-failure runout from adjacent or nearby slopes. Any proposed mitigation should address all recognized significant off-site hazards."
Guidelines for Evaluating and Mitigating Seismic Hazaards in California, Page 37
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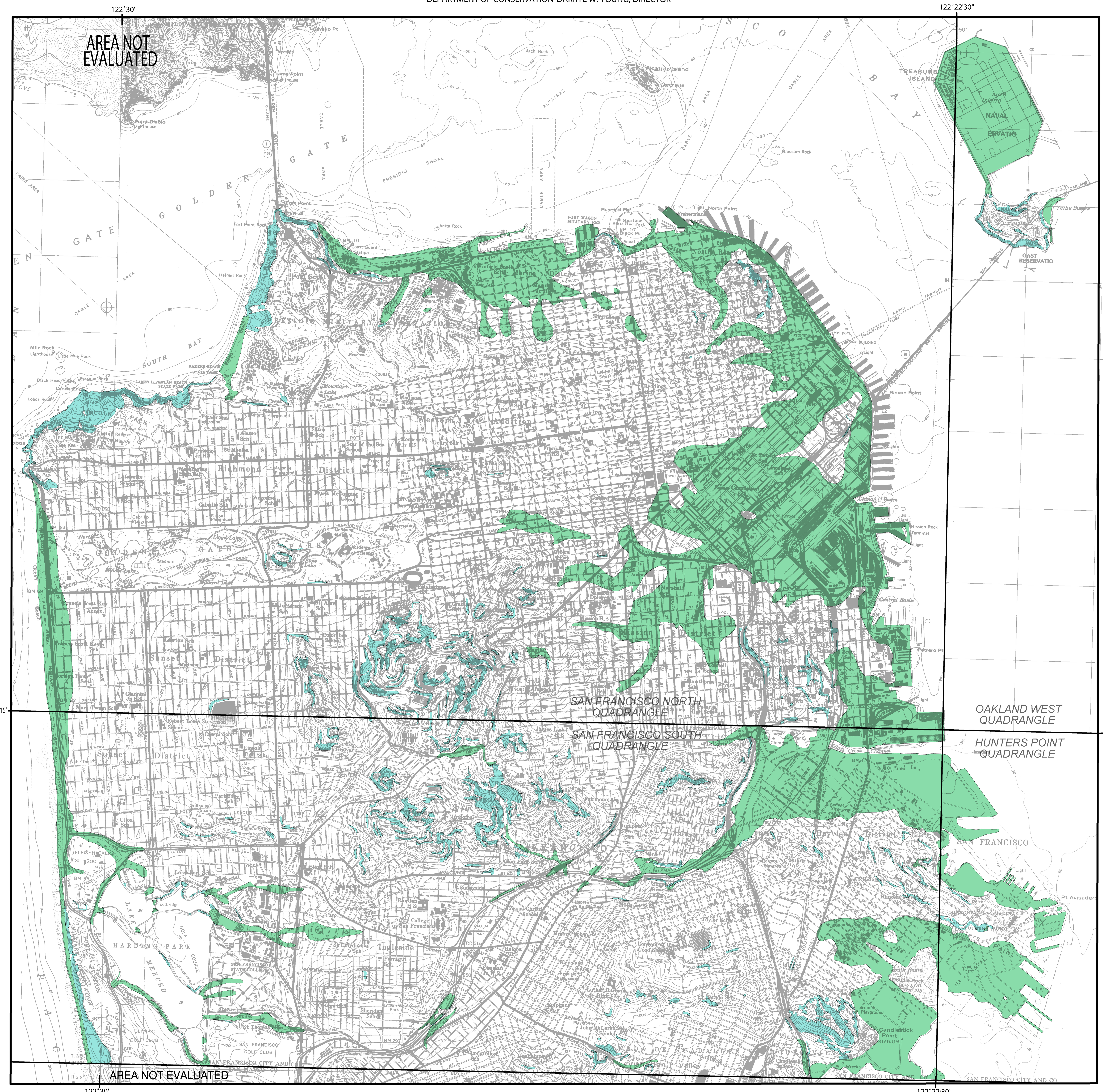
DIVISION OF MINES AND GEOLOGY
JAMES F. DAVIS, STATE GEOLOGIST

STATE OF CALIFORNIA-GRAY DAVIS, GOVERNOR
THE RESOURCES AGENCY-MARY D. NICHOLS, SECRETARY
DEPARTMENT OF CONSERVATION-DARRYL W. YOUNG, DIRECTOR

SEISMIC HAZARD ZONES
City and County of San Francisco



Base Map prepared by U.S. Geological Survey, various dates

SCALE 1:24,000

PURPOSE OF MAP

This map will assist cities and counties in fulfillng their responsibilities for protecting the public safety from the effects of earthquake-triggered ground failure as required by the Seismic Hazards Mapping Act (Public Resources Code Sections 2690-2699.6).

For information regarding the scope and recommended methods to be used in conducting the required site investigations, see DMG Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

For a general description of the Seismic Hazards Mapping Program, the Seismic Hazards Mapping Act and regulations, and related information, please refer to www.conservation.ca.gov/cgs.

Production of this map was funded by the Federal Emergency Management Agency's Hazard Mitigation Program and the Department of Conservation in cooperation with the Governor's Office of Emergency Services.

IMPORTANT - PLEASE NOTE

1) This map may not show all areas that have the potential for liquefaction, landsliding, strong earthquake ground shaking or other earthquake and geologic hazards. Also, a single earthquake capable of causing liquefaction or triggering landslide failure will not uniformly affect the entire area zoned.

2) Liquefaction zones may also contain areas susceptible to the effects of earthquake-induced landslides. This situation typically exists at or near the toe of existing landslides, downslope from rockfall or debris flow source areas, or adjacent to steep stream banks.

3) This map does not show Alquist-Priolo earthquake fault zones, if any, that may exist in this area. Please refer to the latest official map of earthquake fault zones for disclosures and other actions that are required by the Alquist-Priolo Earthquake Fault Zoning Act. For more information on this subject and an index to available maps, see DMG Special Publication 42.

4) Landslide zones on this map were determined, in part, by adapting methods originally developed by the U.S. Geological Survey (USGS). Landslide hazard maps prepared by the USGS typically use experimental approaches to assess earthquake-induced and other types of landslide hazards. Although aspects of these new methodologies may be incorporated in future CDMG seismic hazard zone maps, USGS maps should not be used as substitutes Official SEISMIC HAZARD ZONES maps.

5) U.S. Geological Survey base map standards provide that 90 percent of cultural features be located within 40 feet (horizontal accuracy) at the scale of this map. The identification and location of liquefaction and earthquake-induced landslide zones are based on available data. However, the quality of data used is varied. The zone boundaries depicted have been drawn as accurately as possible at this scale. Zone boundaries reflect digital topographic data that may differ slightly from the shorelines shown on the base map.

6) Information on this map is not sufficient to serve as a substitute for the geologic and geotechnical site investigations required under Chapters 7.5 and 7.8 of Division 2 of the Public Resources Code.

7) DISCLAIMER: The State of California and the Department of Conservation make no representations or warranties regarding the accuracy of the data from which these maps were derived. Neither the State nor the Department shall be liable under any circumstances for any direct, indirect, special, incidental or consequential damages with respect to any claim by any user or any third party on account of or arising from the use of this map.

STATE OF CALIFORNIA

SEISMIC HAZARD ZONES

Delineated in compliance with
Chapter 7.8, Division 2 of the California Public Resources Code
(Seismic Hazards Mapping Act)

CITY AND COUNTY OF SAN FRANCISCO

OFFICIAL MAP

Released: November 17, 2000

STATE GEOLOGIST

MAP EXPLANATION

Zones of Required Investigation:

Liquefaction
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

DATA AND METHODOLOGY USED TO DEVELOP
THIS MAP ARE PRESENTED IN THE FOLLOWING:

Seismic Hazard Evaluation of the City and County of San Francisco, California:
California Division of Mines and Geology, Open-File Report 2000-009.

For additional information on seismic hazards in this map area, the rationale used for zoning, and additional references consulted, refer to www.conservation.ca.gov/cgs