GUIDANCE

FOR

CALIFORNIA

ACCIDENTAL RELEASE PREVENTION (CalARP) PROGRAM

SEISMIC ASSESSMENTS

Prepared for the

UNIFIED PROGRAM AGENCY (UPA) SUBCOMMITTEE

REGION I LOCAL EMERGENCY PLANNING COMMITTEE (LEPC)

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GLOSSARY OF TERMS

Accidental Release - an unanticipated emission of a regulated substance or other extremely hazardous substance (EHS) into the ambient air from a stationary source.

Active Faults - a fault that is likely to become a source of another earthquake sometime in the future. A fault is determined to be active by the Authority Having Jurisdiction from properly substantiated data (e.g., most recent mapping of active faults by the U.S. Geological Survey).

Components - a general term used to describe architectural elements, or mechanical and electrical equipment.

Convective Component - a portion of the tank contents which responds independently of the tank shell when subjected to seismic shaking; the convective component of the liquid is in the upper portion of the tank and responds in a sloshing mode.

Covered Process - a process that has a regulated substance present in more than a threshold quantity as determined under California Code of Regulations (CCR) Title 19 Division 2 Chapter 4.5 §2770.2 (Reference 1).

Damage Mechanism - the mechanical, chemical, physical, or other process that results in equipment or material degradation.

Design Earthquake - two-thirds of the corresponding Maximum Considered Earthquake according to ASCE/SEI 7-10.

Diamond Shape Buckling - a form of buckling of the tank shell, often found in the upper courses of slender tanks, in which the shell wrinkles in diamond-shaped patterns.

Distribution Systems - an interconnected system of piping, tubing, conduit, raceway, or duct. Distribution systems include in-line components such as valves.

Drift - lateral displacement between floors or segments of a structure under earthquake loading.

Ductility - maximum deformation divided by deformation at yield; routinely utilized a measure of energy absorbing capability.

Ductility Based Reduction Factor - a factor representing a measure of energy absorbing capability of a structure.

Elephant Foot Buckling - a form of buckling of the tank shell near its connection with the bottom plate that resembles an elephant's foot, in which the shell bulges outward near the bottom, but is constrained at its base by the bottom plate.

Extremely Hazardous Substances - includes any chemicals or hazardous substances identified by the U.S. Environmental Protection Agency (EPA) on the basis of hazard or toxicity and listed under the U.S. Emergency Planning and Community Right-to-Know Act (EPCRA). See Appendix A of the following website at**:**
https://www.epa.gov/sites/production/files/2015-03/documents/list_of_lists.pdf

Freeboard - vertical distance between the free surface of liquid contained in the tank and the top of the tank shell or underside of tank roof.

Highly Hazardous Material - a flammable liquid, flammable gas, toxic or reactive substance as defined in CCR Title 8, §5194, Appendices A or B. Highly hazardous material includes all regulated substances listed in Tables 1, 2 and 3 of 19 CCR §2770.5.

Importance Factor - a factor that accounts for the degree of risk to human life, health and welfare associated with damage to property or loss of use or functionality.

Impulsive Component - portion of the tank contents which responds in unison with the tank shell when subjected to seismic shaking.

Intermediate Moment Resisting Frame - a moment resisting frame with certain design features to provide ductile post yield behavior, but not containing all of the design features of a special moment resisting frame

Moment Resisting Frame - a frame in which members and joints are capable of resisting forces primarily through flexure.

Near Field - area in close proximity to active faults usually taken as within 5 to 10 kilometers of the fault trace.

Nonbuilding Structure - A structure, other than a building, constructed of a type included in Chapter 15 and within the limits of Section 15.1.1 of ASCE/SEI 7-10.

Nonbuilding Structure Not Similar to a Building - Nonbuilding structures that do not have lateral and vertical seismic force-resisting systems that are similar to buildings.

Nonbuilding Structure Similar to a Building - A nonbuilding structure that is designed and constructed in a manner similar to buildings, will respond to strong ground motion in a fashion similar to buildings, and has a basic lateral and vertical seismic force-resisting system conforming to one of the types indicated in Tables 12.2-1 or 15.4-1 of ASCE/SEI 7-10.

Nonstructural Components - architectural, mechanical, or electrical components that are permanently attached to structures.

Offsite - areas beyond the property boundary of the stationary source, and areas within the property boundary to which the public has routine and unrestricted access during or outside business hours.

Petroleum Refinery - a stationary source engaged in activities set forth in North American Industry Classification System (NAICS) code 324110.

Process - any activity involving a regulated substance including any use, storage, manufacturing, handling, or on-site movement of such substances, or combination of these activities. For the purposes of this definition, any group of vessels that are interconnected, or separate vessels that are located such that a regulated substance could be involved in a potential release, shall be considered a single process. This definition shall not apply to Article 6.5 (Reference 1; see Program 4 Prevention Program definition).

Process - for purposes of Article 6.5 means petroleum refining activities involving a highly hazardous material, including use, storage, manufacturing, handling, piping, or on-site movement. For the purposes of this definition, any group of vessels that are interconnected, or separate vessels that are located such that an incident in one vessel could affect any other vessel, shall be considered a single process. Utilities and safety related devices shall be considered part of the process if, in the event of an unmitigated failure or malfunction, they could potentially contribute to a major incident. This definition includes processes under partial or unplanned shutdowns. Ancillary administrative and support functions, including office buildings, laboratories, warehouses, maintenance shops, and change rooms are not considered processes under this definition (Reference 1).

Process Hazard Analysis or Hazard Review - a systematic method of identifying, evaluating, and controlling the hazards associated with each covered process.

Program 4 Prevention Program (Article 6.5) - applies to all processes within petroleum refineries, as specified in §2762.0.1. The purpose of Program 4 is to prevent major incidents at petroleum refineries in order to protect the health and safety of communities and the environment (Reference 1).

Regulated Substance - any substance, unless otherwise indicated, listed in §2770.5 (Reference 1).

Response Spectrum - response of a single degree of freedom oscillator subject to vibratory motion.

Revalidation - a critical review of a hazard review or a process hazard analysis (PHA) with qualified team members of the most recent hazard review or PHA studies to verify that past studies remain valid and that changes made to the covered process are properly assessed. This critical review is to ensure that hazards are well understood, and existing safeguards are properly identified, past recommendations have been addressed, the overall risk ranking of each scenario is accurate, and relevant incidents and near misses at the stationary source and industry are evaluated. For situations when past studies cannot be readily revalidated, a new complete hazard review or PHA may be warranted.

Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion - the most severe earthquake effects considered by ASCE/SEI 7-10 determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustment for targeted risk.

Seismic Interaction - is the physical interaction, such as impact or differential displacement, between adjacent structures, systems, or components caused by relative motions from an earthquake.

Site Amplification Effects - as seismic waves travel from the rock where the fault rupture occurred to the surficial geological layers of a site, a change in the seismic wave’s amplitude and frequency will occur which could result in strong amplification if the geological conditions are unfavorable.

Sloshing - relative movement of the free surface of liquid contained in the tank as a result of seismic shaking.

Special Moment Resisting Frame - a moment resisting frame specially detailed to provide ductile post yield behavior through member proportioning and connection detailing.

SRSS - square root of the sum of the squares; a method for combining modes or force components resulting from different directions of motion.

Stationary Source - any buildings, structures, equipment, installations, or substance emitting stationary activities which belong to the same industrial group, which are located on one or more contiguous properties, which are under the control of the same person (or persons under common control), and from which an accidental release may occur. The term stationary source does not apply to transportation, including storage incident to transportation, of any regulated substance or any other extremely hazardous substance. A stationary source includes transportation containers used for storage not incident to transportation and transportation containers connected to equipment at a stationary source for loading or unloading. Transportation includes, but is not limited to, transportation subject to oversight or regulations under Part 192, 193, or 195 of Title 49 of Code of Federal Regulations (CFR) or a state or natural gas or hazardous liquid program for which the state has in effect a certification to Department of Transportation (DOT) under Section 60105 of Title 49 of USC. A stationary source does not include naturally occurring hydrocarbon reservoirs. Properties shall not be considered contiguous solely because of a railroad or pipeline right-of-way.

Threshold quantity - quantity specified for a regulated substance pursuant to §2770.5 and determined to be present at a stationary source as specified in §2770.2 (Reference 1).

Unified Program Agency (UPA) - the local agency, pursuant to Health and Safety Code (HSC) §25501, responsible to implement the CalARP Program.

Utility - for the purposes of Article 6.5 a system that provides energy or other process-related services to enable the safe operation of a petroleum refinery process. This definition includes electrical power, fire water systems, steam, instrument power, instrument air, nitrogen, and carbon dioxide (Reference 1).

Weak Story – defined as Type 5a per ASCE 7-10 Table 12.3-2 Vertical Structural Irregularity.

Acronyms

AA	Administering Agency
AHJ	Authority Having Jurisdiction
API	American Petroleum Institute
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society of Testing Materials
AWWA	American Water Works Association
CalARP	California Accidental Release Prevention
CalOES	Governor’s Office of Emergency Services
CBC	California Building Code
CCR	California Code of Regulations
CEBC	California Existing Building Code
CFR	Code of Federal Regulations
CGS	California Geological Survey
DBE	Design Basis Earthquake
DCR	Demand to Capacity Ratio
DE	Design Earthquake
DGF	Dibblee Geological Foundation
DOT	Department of Transportation
EHS	Extremely Hazardous Substance
EPA	Environmental Protection Agency
EPCRA	Emergency Planning and Community Right-to-Know Act
FEMA	Federal Emergency Management Agency
FRP	Fiber Reinforced Plastic
HR	Hazard Review
IBC	International Building Code
LEPC	Local Emergency Planning Committee
MCE	Maximum Considered Earthquake
MCE_G	Geo-Mean Maximum Considered Earthquake
MCE_R	Risk-Targeted Maximum Considered Earthquake
NAICS	North American Industry Classification System
NCHRP	National Cooperative Highway Research Program
PHA	Process Hazard Analysis
RMP	Risk Management Plan
RS	Regulated Substance
SAM	Seismic Anchor Movement
SEI	Structural Engineering Institute
UBC	Uniform Building Code
UPA	Unified Program Agency
USC	United States Code
USGS	United States Geological Survey

1.0 INTRODUCTION

The objective of the California Accidental Release Prevention (CalARP) Program is to prevent releases of regulated substances (RS) with potential offsite consequences to the public and the environment. Seismic assessments have been a required component of the state mandated CalARP Program since 1998. The purpose of this document is to provide guidance regarding applicable criteria to be used in such assessments. This guidance document is an update of the CalARP seismic document published in March 2014 (Reference 2), and is applicable to covered processes, stationary sources, structural systems and components whose failure could result in a release of RS with potential offsite impacts.

The guidance in this document provides for a deterministic evaluation of structural systems and components. This deterministic evaluation should be performed considering an earthquake which has a low probability of occurrence (code Design Earthquake as defined in ASCE/SEI 7-10 [Reference 4]). The seismic capacity of structures and components to withstand this level of earthquake should be calculated using realistic criteria and assumptions.

An acceptable alternate approach is to perform a probabilistic risk assessment which provides estimates and insights on the relative risks and vulnerabilities of different systems and components from the impact of an earthquake. These risks should be compatible with accepted practices for similar civil and industrial facilities. When a probabilistic risk assessment approach is planned, the owner/operator should consult with the local Authority(ies) Having Jurisdiction (AHJ) to describe why this approach is being planned and explain differences between this approach and the deterministic method.

The AHJ is usually a Unified Program Agency (UPA) also referred to as an Administering Agency (AA), which implements the CalARP program and enforces the CalARP regulations. The AHJ may also include local city or county Building & Safety Departments that approve plans and issue permits for renovation and/or construction/installation of structural systems/components.

The CalARP regulations state in §2735.5. The owner or operator of a stationary source shall closely coordinate with the UPA to implement the requirements of this chapter and to determine the appropriate level of documentation required for an RMP (risk management plan) to comply with Sections 2745.3 through 2745.9 of this chapter (Reference 1).

Thus, prior to beginning any seismic assessment, the owner/operator must consult closely with the UPA to obtain mutual understanding and agreement on the scope and depth of the assessment, the general approach proposed by the Responsible Engineer (see Section 1.5) and the schedule for the assessment.

The owner/operator of CalARP covered processes subject to Programs 2 and 3 requirements and stationary sources subject to Program 4 requirements must conduct a process hazard analysis (PHA) or hazard review (HR) to identify, evaluate and control hazards associated with each process. Additionally, the owner/operator must work with the UPA in selecting and using the PHA/HR methodology best suited to determine the process hazards being analyzed. The

PHAs/HRs must include a consideration of natural or manmade external events, such as earthquakes, fires, tsunamis, etc. For each external event, with a potential to create a release of a regulated substance that will reach an endpoint offsite, the owner/operator must provide specific information in the RMP, such as the date of completion of the PHA/HR, and the date of the most recent field verification that equipment is installed and maintained as designed (Reference 1).

A seismic evaluation, therefore, must be conducted and included as a part of the PHA/HR external events analysis with the RMP submittal for the RMP to be deemed as complete during the regulatory RMP review process. The PHA/HR, including the seismic evaluation, are required to be updated and revalidated with the RMP submittal every five years (Reference 1).

1.1 Evaluation Scope – The owner/operator, in consultation with the AHJ and Responsible Engineer (see Section 1.4), should always identify the scope to be evaluated in accordance with the CalARP Program regulations and this guidance. The scope is expected to fall into three categories, as follows:

1) Components of a covered process subject to Program Level 2 and 3 requirements and a stationary source subject to Program 4 requirements, as defined by §2735.4 (Reference 1).
2) Adjacent structures, distribution systems, equipment, etc. whose failure or excessive displacement could result in a release of RS with potential offsite consequences.
3) Onsite utilities and emergency systems which would be required to operate following an earthquake for emergency response or to maintain the facility in a safe condition, (e.g., emergency power, detection and alarm systems, pressure relief devices, flare systems, battery racks, firewater systems, steam, instrument power, instrument air, cooling water, ventilation and diffusion systems, etc.).

1.2 Performance Criteria – In order to achieve the overall objective of preventing releases of RS, individual equipment items, structures, and utilities (e.g., power, water, etc.) may need to achieve varied performance criteria. These criteria may include one or more of the following:

1) Maintain structural integrity
2) Maintain position
3) Maintain containment of RS
4) Function immediately following an earthquake

Note that an owner/operator may choose to set more stringent performance requirements dealing with continued function of the facilities both during and after an earthquake. These are individual business decisions and are not required for compliance with the CalARP Program

Prior to the 2007 California Building Code (CBC), the CBC was based on adopted versions of the Uniform Building Code (UBC). Starting in 2008, all new facilities in California should have been designed in accordance with 2007 CBC which was based on the 2006 International Building Code (IBC). The 2006 IBC in turn referenced the American Society of Civil Engineers (ASCE) and Structural Engineering Institute (SEI) Standard ASCE/SEI 7-05 for its seismic load provisions. This system of codes and referencing has continued since and the current 2016 CBC (Reference 22) is based on the 2015 IBC (Reference 19) which in turn references ASCE/SEI 7-10 including Supplements 1 and 2 (hereinafter referred to as ASCE 7-10) for seismic load provisions.

It should be noted that design earthquake terminology changed between the UBC and ASCE/SEI 7-05 and between ASCE/SEI 7-05 and ASCE 7-10. The design earthquake ground motion in the UBC is called the "design basis earthquake" while in ASCE/SEI 7-05 and ASCE 7-10, it is called "design earthquake" (DE). Also, the maximum earthquake ground motion considered in ASCE/SEI 7-05 was called the "Maximum Considered Earthquake" (MCE) while the maximum earthquake ground motion considered in ASCE 7-10 is called the "Risk-Targeted Maximum Considered Earthquake" (MCE_R).

It is the consensus of this Committee that RS systems and components designed and properly constructed in accordance with the 1997 UBC (Reference 3) or ASCE/SEI 7-05 (or later) provisions, and which have not been subjected to detrimental modifications or significant deterioration, provide reasonable assurance of withstanding design/evaluation basis earthquake effects without either structural failure or a release of RS having offsite consequences.

It is also the consensus of this Committee that RS systems and components that were designed and constructed in accordance with the 1988, 1991 or 1994 UBC also provide reasonable assurance of withstanding design/evaluation basis earthquake effects without either structural failure or a release of RS (caused by a loss of containment or pressure boundary integrity). This consensus does not apply to systems and components that meet any of the following:

1) The facility in which systems and components are contained is located in the near field of an active earthquake fault.
2) The facility in which the systems and components are contained is located on a soft soil site.
3) The steel structures supporting equipment and/or piping that contain RS and utilize pre-Northridge type special moment connections (see Figure 1).
4) Reinforced concrete chimneys or stacks with large rectangular breach openings.

State and national policies have generally established performance objectives for new facilities that are more restrictive than those for existing facilities. This guidance document recognizes this to be appropriate. However, it should be recognized that any regular inspection and repair of systems containing RS should make them significantly safer than similar systems for which these steps are not taken.

1.3 Extent of Seismic Evaluations Required – All equipment and components identified in Section 1.1 are subject to the seismic assessment guidelines of this document. However, the extent of these evaluations may be limited or expanded depending on the unique features, conditions, age and complexity of the facility and/or processes. Given these wide ranging facility and/or process variables, the owner/operator should consult with the AHJ to determine which of the following subsections would be applicable.

1.3.1 Existing Facilities Which Have Not Had Previous CalARP Seismic Assessments

1) Constructed to 1985 UBC and Earlier or Unknown Standard

There is considerable uncertainty about the capacity of nonbuilding structures and nonstructural components designed and constructed prior to the 1988 UBC. This is because there were no specific seismic code requirements for nonbuilding structures and nonstructural components in heavy industrial applications and they were rarely reviewed and inspected by building departments. Starting with the 1988 UBC, seismic code requirements were provided and designs were much more consistent. Therefore, pre-1988 UBC nonbuilding structure and nonstructural component designs should be given closer scrutiny.

2) Constructed to 1988 UBC and Later

Existing facilities which are subject to the CalARP requirements and which were permitted for construction in California in accordance with the 1988 or later version of the UBC may generally be deemed to meet the intent of the requirements of Section 4 of this Guidance, provided the following conditions are met and documented:

a. The near field requirements of either ASCE/SEI 7-05, ASCE 7-10 or the 1997 UBC, either using the near field maps or a site-specific spectrum, are satisfied or the facility is not located in the near field zone (i.e., where per ASCE/SEI 7-05/10 S_S is not greater than 1.5 and S_1 is not greater than 0.6 or per the 1997 UBC the facility is not within 15 km of an active fault).
b. The soft soil site conditions of ASCE/SEI 7-05, ASCE 7-10 (ASCE Site Classes E and F) or the 1997 UBC (UBC Soil Type S_E and S_F) were considered in the design of the facility or the facility is not located on a soft soil site.
c. A walkdown in accordance with Section 3 reveals adequate lateral force resisting systems.

The recommended contents of the initial report are given in Section 9.

1.3.2 New Facilities That Are Subject to CalARP Program Requirements – Design and construction of new facilities containing RS must satisfy the seismic provisions of the 2016 California Building Code (ASCE/SEI 7-10). In general, such facilities are deemed to satisfy the analytical evaluation requirements of the guidance document. However, a walkdown should always be performed in accordance with Section 3 after construction has been completed. The recommended contents of the initial report are given in Section 9.

1.3.3 Facility Revalidation With a Previous CalARP Seismic Assessment – The CalARP Program regulations requires that owner/operators update and revalidate their PHA/HR at least every five years. The extent of a seismic assessment revalidation depends on many factors that need to be coordinated and agreed to by the AHJ. If deemed appropriate by the Responsible Engineer (see Section 1.4), any portion of the previous assessment may be used for the current assessment. However, any revalidation should include the performance of a walkdown in accordance with Section 3 of this document. As part of the revalidation process the equipment population being assessed should be discussed with the process engineer responsible for defining the scope of the assessment. It is possible that process conditions have changed since the initial screening of equipment having offsite consequences was first performed.

Additionally, it is important to note that the scope of PHAs and/or PHA revalidations has expanded for stationary sources subject to Program 4 requirements based on changes to the CalARP Program regulations (Reference 1) to include applicable processes within a petroleum refinery.

The recommended contents of the revalidation report are given in Section 9. Facilities which are unable to locate original CalARP Seismic Assessment documents require a more in-depth reassessment process, in lieu of the standard revalidation report and as recommended by the Responsible Engineer.

1.3.4 Occurrence of Conditions That Would Trigger an Assessment within the Revalidation Period – It is recommended that the owner/operator assessing the validity of past evaluations considers conditions that may make a partial or entirely new assessment necessary. Examples of such conditions include:

1) New significant active fault discovered near the facility which causes a major increase in the estimated ground motions, or other seismic hazards.
2) System modifications that would significantly affect the seismic behavior of the equipment or system, such as changing or addition of process equipment.
3) The occurrence of an earthquake that has caused significant damage in the local vicinity of the facility since the latest assessment.
4) The occurrence of other events (e.g., fire or explosion) that have caused structural damage.
5) Significant deterioration and/or damage mechanisms in equipment, piping, structural members, foundations or anchorages.

1.3.5 Changes to This Guidance Document and/or Ground Motions That May Expand Revalidation Requirements - This guidance document is updated at 5-year intervals to incorporate needed technical and administrative revisions. At times, minor revisions are made to Q values, which the committee does not believe should trigger a reassessment of previously qualified or retrofitted items. However at times substantive revisions are made, which may require reassessment of some items previously qualified. In such cases, it is recommended that the affected items be identified and listed in the report during the first subsequent revalidation. The rigor of the new assessment should not be less than what is otherwise required by this document. Walkdown, drawing review, scaling of prior analysis results and/or a new analysis can be utilized as directed by the Responsible Engineer.

The committee recognizes that the detailed requirements for calculating ground motions in ASCE 7 are typically modified with each version of the document, and the USGS ground motion estimates may also change from time to time as they update their data and methods. As a result, the design earthquake ground motions for a site may go up or down from the same values calculated 5 years earlier. Given the variability of these estimates, the committee recommends that previous assessments and upgrades do not need to be redone during the revalidation only because of changes in the ground motions due to reasons mentioned above, with the exception of circumstances such as those mentioned in Section 1.3.4 (e.g. discovery of a new fault near the site, etc.).

However, previous evaluations and upgrades should be reviewed if significant physical changes have been made during the 5 year period that could adversely affect seismic performance (e.g. additional weight, change of load path), if those changes were implemented without verifying compliance with the CalARP seismic guidance criteria or current building code in force at that time.

1.4 Responsible Engineer – The Responsible Engineer has the responsibility for conducting and/or overseeing the evaluations and walkdowns required by this document for a given facility. All applicable engineering work associated with seismic evaluations should be performed or supervised by California Registered Professionals in accordance with the Business and Professions Code, Chapter 7, §§6700-6799 and CCR, Title 16, Division 5, §§400-476. It is strongly recommended that the Responsible Engineer be registered in California as a Civil, Structural or Mechanical Engineer with experience in seismic design and/or evaluations of facilities within the scope of this document.

1.5 Limitations – The guidance provided is intended to reduce the likelihood of a release of RS. Conformance to this document does not guarantee or assure that a release of RS will not occur in the event of strong earthquake ground motions.

Conformance to this document does not guarantee or assure that the intent of the CBC Section 101.3 will be met.

2.0 DETERMINATION OF SEISMIC HAZARDS

When a seismic hazard assessment is performed, it should address and, where appropriate, quantify the following site-specific seismic hazards:

1) Ground shaking, including local site amplification effects
2) Fault rupture
3) Liquefaction and lateral spreading
4) Seismic settlement
5) Landslides
6) Tsunamis and seiches

Each of these site-specific seismic hazards is discussed in the following sections. Attachment A presents guidance for geotechnical reports that may be necessary to perform these evaluations.

2.1 Ground Shaking – It is the consensus of the Seismic Guidance Committee that the same ground motion hazard used in the design of new facilities be used as the basis for evaluating existing facilities. (i.e., the “Design Earthquake Response Spectrum” as per Section 11.4.5 of ASCE 7-10). The procedures of ASCE 7-10 should be used consistently for determination of these ground motions, including Chapter 21 of ASCE 7-10 for site-specific assessments. Values to be used in these evaluations may be obtained online from the links provided by the United States Geological Survey (USGS) website at https://earthquake.usgs.gov/designmaps. Latitude and longitude of the facility should always be used, as opposed to zip codes, along with the appropriate soil classification.

2.2 Fault Rupture – Fault rupture zones which pass near or under the site should be identified. A fault is a fracture in the earth's crust along which the separated sections have moved or displaced in relation to each other. The displacement can be in either a horizontal or vertical direction. A ground rupture involving more than a few inches of movement can cause major damage to structures sited on the fault or pipelines that cross the fault. Fault displacements produce forces so great that the best method of limiting damage to structures is to avoid building in areas close to ground traces of active faults.

Under the Alquist-Priolo Special Studies Zones Act of 1972, the State Geologist is required to delineate "Earthquake Fault Zones" along known active faults in California. Interactive fault maps can be found online at the California Geological Survey (CGS) website at https://maps.conservation.ca.gov/cgs/EQZApp/. Editions of CGS Special Publication 42 prior to the 2018 edition provided these hazard maps, but the document now presents guidelines for practitioners for assessing fault rupture hazards in California and provides additional means to access the fault maps online..

2.3 Liquefaction and Lateral Spreading – Liquefaction is the transformation of soil from solid to a liquid state caused by an increase in pore water pressure and a reduction of effective stress within the soil mass during an earthquake. The potential for liquefaction is greatest when loose saturated cohesionless (sandy) soils or silty soils of low plasticity are subjected to a long duration of seismically induced strong ground shaking.

The assessment of hazards associated with potential liquefaction of soil deposits should consider two basic types of hazards:

1) One type of hazard associated with liquefaction is translational site instability more commonly referred to as lateral spreading. Lateral spreading occurs on gently sloping ground with free-face (stream banks, and shorelines), when seams of liquefiable material are continuous over large lateral areas and serve as significant planes of weakness for translational movements.
2) Localized liquefaction hazards may include large liquefaction-induced settlements/differential settlements and foundation bearing failures.

The current 2016 CBC, 2015 IBC and ASCE 7-10 require the liquefaction hazards be evaluated for the Maximum Considered Earthquake (MCE_G) geo-mean earthquake ground motions. Previous editions of the IBC and ASCE/SEI 7 required the liquefaction hazards be evaluated for the Design Earthquake (DE) ground motions.

It should be noted that although the current codes have changed their requirements regarding the seismic hazard level, those changes are also associated with different performance expectations for the design of new structures (i.e., non-collapse) in the MCE. It is the consensus of this committee that changing the seismic hazard levels for CalARP assessments of existing facilities to be consistent with philosophical changes in new design codes would add a level of complexity that is not justified and inconsistent with the approach used throughout this document. As such, this document continues to use the DE ground motions to evaluate liquefaction hazards for existing facilities

The CGS has established evaluation guidelines in Special Publication 117 (SP117) (Reference 5). Preliminary screening investigations for liquefaction hazards should include the following:

1) Check the site against the liquefaction potential zone identified on the CGS Seismic Hazard Zones Maps where available.
2) Check for susceptible soil types. Most susceptible soil types include sandy soils and silty soils of low plasticity. Also susceptible are cohesive soils with low clay content (less than 15% finer than 0.005mm), low liquid limit (less than 35%), and high moisture content (greater than 0.9 times the liquid limit). The latter may be designated as “quick” or “sensitive” clays.

3) Check for groundwater table. Liquefaction can only occur in susceptible soils below the groundwater table. Liquefaction hazards should be evaluated only if the highest possible groundwater table is shallower than 50 feet from the ground surface.

4) Check for in-situ soil densities to determine if they are sufficiently low to liquefy. Direct in-situ relative density measurements, such as the ASTM D 1586 (Standard Penetration Test) or ASTM D 3441 (Cone Penetration Test) or geophysical measurements of shear-wave velocities can provide useful information for screening evaluation. This information will usually need to be evaluated by a geotechnical engineer.

The issue of liquefaction may be discounted if the geotechnical report or responsible engineer, using one or more of the above screening approaches, concludes that the likelihood of liquefaction is low.

A site-specific investigation and liquefaction evaluation may be omitted if a screening investigation can clearly demonstrate the absence of liquefaction hazards at site. Where the screening investigation indicates a site may be susceptible to liquefaction hazard, a more extensive site-specific investigation and liquefaction evaluation should be performed by a geotechnical engineer.

If liquefaction potential is identified at the facility, the assessment should focus on the consequences of potential liquefaction including the potential for seismic settlement, lateral spreading, kinematic and downdrag loads on deep foundations, loss of bearing capacity/lateral resistance, and increased lateral loads on retaining walls.

2.4 Seismic Settlement – In addition to the effects of liquefaction, foundation settlement may occur due to soil compaction in strong ground shaking. A geotechnical engineer can determine the potential for this settlement.

2.5 Landslides – Facilities that are in close proximity to natural hillside terrain or man-made slopes (cut or fill slopes) are potentially susceptible to earthquake-induced landslide hazards. SP117 (Reference 5) presents guidelines for evaluation and mitigation of earthquake-induced landslide hazards. NCHRP 611 (Reference 20) also provides analytical methods for the evaluation of slopes and embankments. Information can also typically be obtained from the Seismic Safety Element of the General Plan. Preliminary screening investigation for such hazards should include the following:

1) As part of the site reconnaissance, the engineer should observe whether there are any existing slopes (natural or man-made) in the immediate vicinity of the facility.

2) If there are no slopes of significant extent within a reasonably adequate distance from the facility, then the potential for landslide may be dismissed as a likely seismic hazard. Engineering judgment may be used to assess what constitutes an "adequate distance." For example, generally level alluvial valleys can be reasonably excluded from the potential for seismically induced landslide.

3) If the facility is in close proximity to existing slopes which could pose a significant hazard, a certified engineering geologist or a registered geotechnical engineer should perform the following screening investigation steps.
   a. Check the site against the Seismic Slope Stability Hazard maps where available prepared by the CGS. Also check other similar maps from the USGS, Dibblee Geological Foundation (DGF), and Seismic Safety Elements of local cities and counties.
   b. Check the site against available published and unpublished geologic and landslide inventory maps.
   c. Review stereoscopic pairs of aerial photographs for distinctive landforms associated with landslides (steep slopes, scarps, troughs, disrupted drainages, etc.).

2.6 Tsunamis and Seiches

2.6.1 Background - Tsunamis, or tidal waves, are generated by distant earthquakes and undersea fault movement. Traveling through the deep ocean, a tsunami is a broad and shallow, but fast moving, wave that poses little danger to most vessels. When it reaches the coastline however, the waveform pushes upward from the ocean bottom to make a swell of water that breaks and washes inland with great force.

A seiche occurs when resonant wave oscillations form in an enclosed or semi-enclosed body of water such as a lake or bay. Seiches may be triggered by moderate or larger local submarine earthquakes and sometimes by large distant earthquakes. A tsunami or seiche may result in flooding of low-lying coastal areas. The greatest hazard results from the inflow and outflow of water, where strong currents and forces can erode foundations and sweep away structures and equipment. The rupture of storage tanks from debris impact and foundation erosion can result in fires and explosions.

The California Geologic Survey (CGS) and the Governor's Office of Emergency Services (CalOES) provides official Tsunami Inundation Maps for most populated areas along the states coastline. These maps can be accessed at https://www.conservation.ca.gov/cgs/geohazards/tsunami/maps. Where mapping is not available, estimates of maximum tsunami run-up can be made using historical information or theoretical modeling

Methodologies for tsunami design are incorporated in ASCE/SEI 7-16 (Reference 18) which will be adopted in CBC 2019. These procedures are intended for the design of tsunami resistant structures and protective barriers, and include site-specific design maps, and procedures for analysis for tsunami loads and effects. An evaluation of a process facility subject to tsunami inundation is complex. Such assessment would need to include the potential effects of flooding, debris, and numerous other unique considerations.

2.6.2 Administrative Mitigation Measures - Due in part to a lack of specific tsunami likelihood and/or probability of occurrence data, administrative mitigation measures are valuable. These include:

1) Early Warning System
2) Evacuation Planning
3) Hazardous Materials Area Plans and Regional Plans
4) Emergency Plant Shutdown Procedures
5) Coordination Emergency Drills

These measures would also be more achievable and timely than attempts to strengthen plant tankage and equipment from the effects of a large tsunami event.

3.0 WALKDOWN CONSIDERATIONS

A critical feature of the evaluation methodology is the onsite review of the existing facility under the direction of the Responsible Engineer. This is primarily a visual review that considers the actual condition of each installation in a systematic manner. It is generally referred to as a "walkdown" or "walkthrough" review because the engineers performing the review systematically walk down each equipment item, building, or system to look for potential seismic vulnerabilities.

The walkdown scope should include all structures, equipment, piping, utilities, and other systems as determined in Section 1.1. The CalARP Program regulations mandate that the walkdown occur for initial and revalidation seismic assessments. PHA/HR revalidations submitted without the walkdown are typically not acceptable to the AHJ.

The basis for assessment may include observed failure modes from past earthquake experience, basic engineering principles, and engineering judgment. The walkdown review emphasizes the primary seismic load resisting elements and the potential areas of weakness due to design, construction, or modification practices, as well as deterioration or damage. A special emphasis is placed on details that may have been designed without consideration of seismic loads. Specific guidance for ground supported tanks is discussed in Section 6. Specific guidance for piping systems is discussed in Section 7.

In many cases, the walkdown review should be supplemented by a review of related drawings. This may be done, for example, to check adequacy of older reinforced concrete structures, to verify anchorage details, or to identify configurations that cannot be visually reviewed due to obstructions, fireproofing, insulation, etc. Note that drawings may not always be available, in which case the engineer should document assumptions made and the basis for those assumptions.

The walkdown review is also used to identify whether or not calculations are needed to complete the evaluation and for what items. The amount of calculations will depend on several factors including the experience of the reviewer, the size, age and condition of the facility, the type of construction, etc. The engineer may choose to evaluate several "bounding cases" or "questionable items" and use those as a basis for further assessments. The calculations should use the guidelines in Section 4 or other appropriate methods.

A detailed description of the walkdown methodology can be found in ASCE Guidelines for Seismic Evaluation and Design of Petrochemical Facilities (Reference 6). Note that at the time of updating this document, Reference 6 is also being updated to the 3rd Edition. That updated version should be used when available. One significant change that will be included in the 3rd Edition of Reference 6 regards the evaluation of reinforced concrete chimneys or stacks with large rectangular breach openings. This is in response to a major collapse of a 350ft high chimney in a refinery in the 1999 M7.4 Turkey earthquake, and the related investigations and ongoing building code changes that have occurred as a result of that failure.

Examples of walkdown evaluation sheets are provided in Figure 6.1 of Reference 6 for equipment and References 7 and 8 for piping (see Attachment B). Items of concern identified in the walkdown should be addressed in the seismic report.

4.0 EVALUATION OF GROUND SUPPORTED BUILDING AND NONBUILDING STRUCTURES

4.1 Ground Motion – Define ground motion and response spectrum as outlined in Section 2.

4.2 Analysis Methodology and Acceptance Criteria – Acceptance for existing ground supported building and nonbuilding structures (including pressure vessels), and their foundations may be accomplished by one of the following methods. Analysis methods described below may also be used in Sections 5 through 7.

4.2.1 Linear Static and Linear Dynamic Analyses – Perform an appropriate linear dynamic analysis or equivalent static analysis.

The evaluation consists of demonstrating that capacity exceeds demand for identified systems. Acceptance is presumed if the following equations are satisfied:

DEMAND*		CAPACITY BASED ON
$D + L \pm \frac{E_{horiz}}{Q} + E_{vert}$	$\leq$	ϕR_n
$D \pm \frac{E_{horiz}}{Q} - E_{vert}$	$\leq$	ϕR_n

* using Load Factors of unity for all loads

Where,

D	=	Dead load including operating loads
L	=	Any sustained live load expected to be present during an earthquake
E_{horiz}	=	Unreduced elastic earthquake horizontal load based upon ground motion determined in Section 2
Q	=	The lowest applicable ductility based reduction factor per Table 1. The Notes to Table 1 are an integral part of the Table and must be adhered to.
E_{vert}	=	Unreduced elastic earthquake vertical load based on vertical accelerations of $0.2S_{DS}$ as defined in ASCE 7-10
ϕ	=	Strength reduction factor (per ACI) or resistance factor (per AISC)
R_n	=	Nominal strength (per ACI or AISC)

And subject to the following considerations:

1) For systems whose fundamental period (T) is less than the period at which the peak spectral acceleration occurs (T_{peak}), one of the following approaches should be used to determine the appropriate level of seismic acceleration for the fundamental and higher modes. [Note: T_{peak} is the period at which the ground motion has the greatest spectral acceleration. For spectra that have flattened peaks (e.g., ASCE/SEI 7-10 Figure 11.4-1), the smallest period of the flattened peak (T_o) should be used.]

   a. The peak spectral acceleration should be used for the fundamental mode of the structure. When considering higher modes, either the peak or actual spectral acceleration values may be used.

   b. For a structure that has a fundamental period less than $0.67xT_{peak}$, the maximum spectral acceleration in the range of 0.5xT to 1.5xT may be used in lieu of the peak spectral acceleration. When considering higher modes, either the peak or actual spectral acceleration values may be used.

2) For redundant structural systems, (e.g., multiple frames or multiple bracing systems), in which seismic loads can be redistributed without failure, the demand (from the previous equation) on an individual frame or member may exceed its capacity by up to 50 percent, provided that the structure remains stable. In addition, the total seismic demand on the structure should not exceed the capacity of the overall structure.

3) Relative displacements should be considered and should include torsional and translational deformations. Structural displacements that are determined from an elastic analysis that was based on seismic loading reduced by the Q-factor, should be multiplied by the Q-factor to determine displacements to be used in an evaluation.

   a. Generally, the drift (relative horizontal displacement) should be less than 0.02H, where H is the height between levels of consideration. This drift limit may be exceeded if it can be demonstrated that greater drift can be tolerated by structural and nonstructural components or the equipment itself.

   b. To obtain relative displacements between different support points, absolute summation of the individual displacements can conservatively be used. Alternatively, the Square Root of the Sum of Squares (SRSS) method for combining displacements may be used where appropriate.

4) The potential for overturning and sliding of the foundation should be evaluated.
   The factor of safety against overturning and sliding should be larger than or equal to 1.0, considering the appropriate Q-factor from Table 1.F.

5) The capacity of existing concrete anchorage may be evaluated in accordance with the strength design provisions of Section 1923 of the 1997 UBC with inspection load factors specified in Section 1923.3 taken as unity. Alternatively, and for post-installed

anchors, the capacity of existing concrete anchorage may be in accordance with the strength provisions of ACI 318-14 (Reference 24) Chapter 17 excluding the requirements of 17.2.3.1, and using Ω_o = 1.0.

6) The directional effects of horizontal earthquake loads should be considered per the requirements of ASCE/SEI 7-10 (Reference 4) Section 12.5.4.

7) Structures that do not pass these evaluation criteria can be reassessed using a more rigorous approach to determine if structural retrofit is actually required.

8) Note that the importance factor (I), as defined in the ASCE/SEI 7-10 (Reference 4) base shear equation for design of new facilities, should be set to unity (1.0) for evaluation of existing facilities, unless an importance factor greater than 1.0 is requested by the owner of the facility.

9) For soil bearing, deep foundations and piping and pressure vessel designs where working stress allowable design is standard practice, the strength level capacity may be taken as 1.6 times working stress allowable (without the 1/3 increase).

4.2.2 Nonlinear Static and Nonlinear Dynamic Analyses – Alternative procedures using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these recommendations. Methods such as nonlinear time history and nonlinear static pushover analyses would be acceptable. The resulting inelastic deformations should be within appropriate levels to provide reasonable assurance of structural integrity. Acceptable methods include those provided in ASCE/SEI 7-10 (Reference 4) Section 16.2 or ASCE 41-13 (Reference 25). For significant structures, where these types of analyses are performed, a peer review should be done.

4.2.3 Recommended Guidelines for Seismic Evaluation and Design of Petrochemical Facilities – ASCE (Reference 6), Section 4.0, including appendices, provides a summary of analytical approaches as well as detailed examples for the evaluation of structural period, base shear and other pertinent topics.

5.0 EVALUATION OF EQUIPMENT AND NONSTRUCTURAL COMPONENTS

Permanent equipment and nonstructural components supported within or by structures as indicated in Section 1.1 should be assessed together with the supporting structure. If the equipment or component is directly founded on soil or ground, it should be treated separately as a nonbuilding structure per Section 4.

The supported permanent equipment and nonstructural components should be considered subsystems if their total weight is less than 25% of the total weight of the supporting structure and subsystems. Design forces should be determined using one of the following methods:

1) The anchorage and attachments may be evaluated in accordance with the equivalent static force provisions of Chapter 13 of ASCE 7-10. The equipment or the nonstructural component itself should be checked for the acceleration levels based on the above referenced sections.
2) A modal dynamic analysis using the evaluation basis spectra as defined in Section 2 of this document, may be performed in accordance with Equation 13.3-4 of ASCE 7-10. Nonlinear dynamic analysis of combined nonstructural systems in accordance with Section 4.2.2 is permitted.

If the permanent equipment or nonstructural component weight is greater than 25% of the weight of the supporting structure, design forces should be determined using one of the following methods:

1) Section 4 with Q-factors equal to the smaller of the values for the equipment or the supporting structure from Table 1 can be used for the entire system.
2) A dynamic analysis of the equipment coupled with the supporting structure may be performed to determine the elastic response of the equipment. The elastic responses should then be reduced by the smaller Q-factors to obtain the design values.

Where an approved national standard provides a basis for the earthquake-resistant design of a particular type of nonbuilding structure, such a standard may be used, provided the ground motion used for analysis is in conformance with the provisions of Section 2.

6.0 EVALUATION OF GROUND SUPPORTED STORAGE TANKS

6.1 Scope – Vertical liquid storage tanks (commonly referred to as aboveground storage tanks or flat bottom storage tanks) with supported bottoms at ground level should be addressed using the approaches provided in this section when they meet one of the criteria in Section 1.2. These are tanks which either (a) contain an RS, (b) contain fluids (firewater being the most common example) which are required in an emergency, or (c) are located sufficiently close to a tank in one of the two previous categories so as to pose a threat to the covered process or its emergency shutdown. Horizontal vessels (bullets), vertical vessels and spherical tanks which are supported at ground level are addressed in Section 4.0. Elevated tanks and vessels are addressed in Section 5.0.

Section 7.0 of Reference 6 provides a thorough overview of tank failure modes during a seismic event, seismic vulnerabilities to look for during a seismic walkdown, and the detailed methodology for analytical evaluation as well as suggested modifications to mitigate seismic hazards. See Figure 7.7 of that document for valuable illustrations of some of the items of concern, which typically include over-constrained piping, stairway and walkway attachments to the tank.

6.2 Tank Damage in Past Earthquakes – Vertical liquid storage tanks with supported bottoms have often failed, sometimes with loss of contents during strong ground shaking. The response of such tanks, unanchored tanks in particular, is highly nonlinear and much more complex than that generally implied in available design standards. The effect of ground shaking is to generate an overturning force on the tank, which in turn causes a portion of the tank bottom plate to lift up from the foundation. While uplift, in and of itself, may not cause serious damage, it can be accompanied by large deformations and major changes in the tank shell stresses. It can also lead to damage and/or rupture of the tank shell at its connection with any attachments (e.g., piping, ladders, etc.) that are over-constrained and cannot accommodate the resulting uplift. Tanks have been observed to uplift by more than 12 inches in past earthquakes.

The following are typical of the failure (or damage) modes of tanks that have been observed during past earthquakes:

1) Buckling of the tank shell known as "elephant foot" buckling. This typically occurs near grade around the perimeter of unanchored tanks. Another less common (and less damaging) buckling mode of the tank shell, normally associated with taller tanks, is "diamond shape" buckling.
2) Weld failure between the bottom plate and the tank shell as a result of high-tension forces during uplift.
3) Fluid sloshing, thus potentially causing damage to the tank's roof and/or top shell course followed by spillage of fluid.
4) Buckling of support columns for fixed roof tanks.
5) Breakage of piping connected to the tank shell or bottom plate primarily due to lack of flexibility in the piping to accommodate the resulting uplift.

6) Tearing of tank shell or bottom plate due to over-constrained stairway, ladder, or piping anchored at a foundation and at the tank shell. Tearing of tank shell due to over-constrained walkways connecting two tanks experiencing differential movement.

7) Non-ductile anchorage connection details (anchored tanks) leading to tearing of the tank shell or failure of the anchorage.

8) Splitting and leakage of tank shells due to high tensile hoop stress in bolted or riveted tanks.

6.3 Recommended Steps for Tank Evaluation – When evaluating existing ground supported tanks for seismic vulnerabilities, the following steps should be followed:

1) Quantification of site-specific seismic hazard as outlined in Section 2.

2) Walkdown inspection to assess piping, staircase and walkway attachments, and other potential hazards.

3) Analytical assessment of tanks to evaluate the potential for overturning and shell buckling. Such analysis may usually be limited to tanks having a height-to- diameter ratio of greater than 0.33.

Engineering judgment of the evaluating engineer should be relied upon to determine the need for analytical evaluations. Considerations such as presence of ductile anchorage, plate thickness, favorable aspect ratio of the tank, operating height, ductile tank material, weld/bolting detail, etc. are important in determining whether an analytical assessment is required. Two evaluation methods are provided below in Sections 6.3.1 and 6.3.2.

6.3.1 Linear Static Analysis of Tanks - Linear static analysis procedures are provided in the following industry standards. These include:

1) API 650 Appendix E (Reference 9) - This method is a standard for the design of new tanks for the petrochemical industry. Its provisions are accepted by the CBC and ASCE/SEI 7-10 and it addresses both anchored and unanchored tanks.

2) AWWA D100 (Reference 10) - This method is very similar to the API 650 method and is used primarily for design of water storage tanks. It addresses both anchored and unanchored tanks.

3) Veletsos and Yang (Reference 11) - This method is primarily for anchored tanks.

4) Manos (Reference 12) - This method was primarily developed to evaluate the stability of unanchored tanks and is based on correlation between empirical design approach and observed performance of tanks during past earthquakes. It is generally less conservative than API 650.

5) Housner and Haroun (Reference 13) - This method is primarily for the analysis of anchored tanks, but is often used for both anchored and unanchored tanks.

6) ACI 350.3, (Reference 14) - Applies to Concrete Tanks (both round and rectangular)
7) API 620 Appendix L (Reference 26)
8) “Nuclear Reactors and Earthquakes”. United States Atomic Energy Commission, TID-7024, August 1961 (Reference 27)

Alternatively, the Q-factor given in Table 1 for tanks in conjunction with the demand equation in Section 4.2.1 may be used to determine the lateral seismic loads for tanks. As a guidance, the Q factor method may be used for non-metallic as well as smaller less significant tanks whereas the more traditional methods in the literature as listed above may be used for larger tanks (metallic and concrete). It should be noted that in References 9 and 10 listed above, Q-factor reductions are inherently included in the determination of seismic forces. In References 11 to 14 listed above, the Q-factor should only be applied to impulsive or structural modes (not sloshing modes).

6.3.2 Nonlinear Static Analysis of Tanks - Section 4.2.2 allows that nonlinear static analysis is an alternative procedure that can be used to evaluate existing structures. Although there are no published guidelines on how to apply this methodology to bottom-supported liquid storage tanks, the following is a suggested approach that can be deemed as acceptable if other methods do not result in demonstrating adequate seismic resistance.

A vertical liquid storage tank may be evaluated using a nonlinear static analysis procedure such as the following:

The loading should be composed of both static fluid pressures, which are constant, plus the effects of fluid inertia forces which are simulated by monotonically increasing two pressure profiles on the tank walls and bottom. The fluid inertia force profiles may be taken from Appendix F of TID 7024 (Reference 27), which contains the original derivation of seismic-induced fluid inertial forces as derived by Housner. The two pressure profiles are (a) those for the portion of the fluid which moves with the tank (termed the impulsive portion), and (b) those for the portion of the fluid which “sloshes” (termed the convective portion). Both portions contain horizontal pressure profiles on the sides of the tank and a vertical pressure profile on the tank bottom.

The pressure profiles are to be monotonically increased until a horizontal “target displacement” for the design earthquake is exceeded at the maximum fluid level. The target displacement may be calculated using Equation 7-28 of ASCE 41-13 (Reference 25). When using this empirical equation for the calculation of the target displacement, in lieu of specific data, the product of the three “C” coefficients need not exceed 1.5.

For thin walled tanks, diamond and elephant foot buckling are potential limit states which can be evaluated by using either recognized equations for storage tank wall stress state at incipient buckling (Reference 29 and 30) or by detailed nonlinear finite element analysis.

The analysis is typically a nonlinear pushover analysis where the fluid inertial loads are increased until a post peak in the load-displacement curve is observed.

The acceptance criteria for the seismic-resisting elements of the tank, including anchor bolts and foundation, should be as follows. For deformation-controlled elements (as defined in ASCE 41-13), the plastic deformation of these elements should not exceed deformations consistent with a "collapse prevention" level of performance. For force-controlled elements (again as defined in ASCE 41-13), the seismic force in the specific element at target displacement may be reduced by the Q-factor as per Section 4.2.1 of this document. However, for such force-controlled elements (such as shell buckling and anchor bolts whose ultimate load is governed by concrete failure), the Q-factor should not exceed 2.5.

6.4 Mitigation Measures for Tanks – If the walkdown and the evaluation of the tank identify potential seismic vulnerabilities, mitigation measures should be considered. These mitigations may include measures such as increasing the tank wall section (e.g., ribs), addition of flexibility to rigid attachments, reduction of safe operating height or, as a last resort, anchorage of the tank.

6.5 Sloshing Effects – The height of the convective (sloshing) wave (d_s) may be calculated by the following equation:

$$d_s = 0.42\, D_i\, S_a$$

Where,

D_i = the diameter of a circular tank, or the longer plan dimension of a rectangular tank

S_a = the spectral acceleration, as a fraction of g, at the convective (sloshing) period

The period (T) of the convective (sloshing) mode in a circular tank may be calculated by the methods in ASCE 7-10 or API 650 Appendix E, which utilize the following formula:

$$T = 2\pi \sqrt{\frac{D_i}{3.68 \cdot g \cdot tanh\left(\frac{3.68 \cdot H}{D_i}\right)}}$$

Where,

H = the height of the fluid

g = the acceleration due to gravity in consistent units

The above equation for amplitude of a sloshing wave is appropriate for fixed roof tanks. However, in lieu of a detailed analysis, the above equation may be used for a floating roof tank if the weight of the floating roof is replaced by an equivalent height of fluid. For fixed roof tanks, the effects of sloshing may be addressed by having sufficient freeboard to accommodate the wave slosh height. However, when this is not possible, then the following steps should be incorporated into the tank evaluation (or the design of mitigation measures):

1) The geometry of the wave (both unconfined and confined by the roof) should be defined. The geometry of the unconfined wave may conveniently be taken as a trapezoid or a parabola.
2) The fluid head of the freeboard deficit (the unconfined wave height less the available freeboard) should be considered to act as an upward load on the roof. The roof live load should not be considered as assisting to resist this upward fluid pressure.
3) The mass of the fluid that is in the sloshing wave but within the portion confined by the roof should be considered to act laterally at the period of the structural (or impulsive) mode, rather than at the period of the sloshing mode.

For floating roof tanks, the key concern is that the slosh height will be sufficient to lift the bottom of the floating roof onto the top of the shell, potentially leading to a release of contents. Since most tank shells cannot sustain such a weight, this could also result in a major risk of buckling or other failure of the shell at the top of the shell.

It should be noted that the long period transition period (T_L) in the seismic hazard formulation (See ASCE 7-10, Chapter 22) defines the long period response that affects sloshing. The engineer should be aware that significant sloshing can occur even at low seismicity sites. There are numerous documented instances of sloshing related damage at sites over 100 miles from the epicenter that had negligible short period ground shaking.

7.0 EVALUATION OF PIPING SYSTEMS

7.1 Aboveground Piping Systems – The evaluation of aboveground piping systems should be primarily accomplished by a field walkdown. This method is recommended because some piping is field routed and, piping and supports may have been modified from that shown on design drawings. This section does not encompass guidelines for the evaluation of pipe racks or similar piping support systems, which should be evaluated in accordance with the recommendations for support structures.

The procedure for evaluating aboveground piping systems should be as follows:

1) Identify piping systems to be evaluated per Section 1.1.
2) Perform a walkdown of the piping systems for seismic capability. Document the walkdown and identify areas for detailed evaluation, if any.
3) Complete the detailed evaluation of any identified areas and recommend remedial actions, if required.

Damage to or failure of pipe supports should not be construed as a piping failure unless it directly contributes to a pressure boundary failure. The intention here is to preserve the essential pressure containing integrity of the piping system but not necessarily leak tightness. Therefore, this procedure does not preclude the possibility of small leaks at bolted flange joints.

The guidance provided in Sections 7.1.1 through 7.1.6 is primarily intended for ductile steel pipe constructed to a national standard such as the American Society of Mechanical Engineers (ASME) B31.3 (Reference 15) or B31.8 (Reference 23). Evaluation of other piping material is discussed in Section 7.1.7. The basis for certain provisions in this section and further discussion can be found in Reference 8.

7.1.1 Historical Piping Earthquake Performance – Ductile piping systems have, in general, performed adequately in past earthquakes. Where damage has occurred, it has been related to the following aspects of piping systems:

1) Excessive seismic anchor movement (SAM). Seismic anchor movements could be the result of relative displacements between points of support/attachment of the piping systems. Such movements include relative displacements between vessels, coolers and other similar process structures, pipe supports, or main headers for branch lines.
2) Interaction with other elements. Interaction is defined as the seismically induced impact of piping systems with adjacent structures, systems, or components, including the effects of falling hazards.
3) Extensive corrosion effects. Corrosion could result in a weakened pipe cross section that could fail during an earthquake.

4) Non-ductile materials such as cast iron, fiberglass, glass, etc., combined with high stress or impact conditions.
5) Failure of pipe supports.
6) Geohazard issues.

7.1.2 Walkdown – The walkdown is the essential element for seismic evaluations of piping systems. Careful consideration needs to be given to how the piping system will behave during a seismic event, how nearby items will behave during a seismic event (if they can interact with the piping system) and how the seismic capacity will change over time. The walkdown should be performed in accordance with Section 3. Some guidance on how to perform a walkdown can be found in Reference 6. An example of a piping walkdown form is shown in Attachment B.

Additional aspects of piping systems which should also be reviewed during the walkdown for seismic capability are:

1) Large unsupported segment of pipe (see ASME B31E (Reference 21) Table 2)
2) Brittle elements
3) Threaded connections, flange joints, and special fittings
4) Inadequate supports, where an entire system or portion of piping may lose its primary support
5) Potential for geohazard issues
6) Connections to components that are susceptible to high seismic displacements

Special features or conditions to illustrate the above concerns include:

1) Inadequate anchorage of attached equipment
2) Short/rigid spans that cannot accommodate the relative displacement of the supports (e.g., piping spanning between two structural systems)
3) Damaged supports including corrosion
4) Long vertical runs subject to inter level drift
5) Large unsupported masses (e.g., valves) attached to the pipe
6) Flanged and threaded connections in high stress locations
7) Existing leakage locations (flanges, threads, valves, welds)
8) Significant external corrosion such as Corrosion Under Insulation (CUI)
9) Inadequate vertical supports or insufficient lateral restraints (pipe could fall off support)

10) Welded attachments to thin wall pipe

11) Excessive seismic displacements of expansion joints

12) Brittle elements such as cast iron pipes

13) Sensitive equipment impact (e.g., control valves)

14) Potential for fatigue of short to medium length rod hangers that are restrained against rotation at the support end

These lists are intended to be illustrative and not comprehensive. Other features may be governing.

7.1.3 Analysis Considerations – Detailed analysis of piping systems should not be the focus of this evaluation. Rather it should be on finding and strengthening weak elements. However, after the walkdown is performed and if an analysis is deemed necessary, the procedures in ASME B31E (Reference 21) and the following general rules should be followed.

1) Friction resistance should not be considered for seismic restraint, except for the following condition: for long straight piping runs with numerous supports, friction in the axial direction may be considered.

2) Spring supports (constant or variable) should not be considered as seismic supports.

3) Unbraced piping with short rod hangers can be considered as effective lateral supports if justified.

4) Appropriate stress intensification factors ("i" factors) should be used.

5) Allowable piping stresses should be reduced to account for fatigue effects due to significant cyclic operational loading conditions. In this case the allowables presented in Section 7.1.7 may need to be reduced.

6) Flange connections should be checked to ensure that high moments do not result in significant leakage.

7.1.4 Inertia Evaluation – The recommended procedure for seismic inertia evaluation of piping is discussed in Section 3.4 of ASME B31E (Reference 21) including allowable stress values. Both horizontal and vertical loading should be considered. Seismic loading should be determined following Section 13.3.1 of ASCE 7-10 (Reference 4) with the value of a_p = 2.5, R_p should be substituted with Q from Table 1 and the value of I_p = 1.0. Since the basis for design or analysis of piping is allowable stress design, the calculated seismic inertia loads should be multiplied by 0.7 as per Section 2.4.1 of ASCE 7-10 (Reference 4). Restraints and bracing of piping are typically designed using strength design procedures and therefore if piping loads are multiplied by 0.7 for pipe stress analysis, they should be increased by a factor of 1.4 to convert back to strength level. Furthermore, calculated forces for restraints and bracing should be multiplied by an additional factor of 1.5 so that restraints and bracing are not the weak link in the seismic load path. It is permissible to perform a seismic inertia evaluation using a properly substantiated dynamic analysis. The methodology discussed in this section should be used for evaluating a piping system, if needed, with the exception of seismic anchor movement.

7.1.5 Seismic Anchor Movement – The recommended procedure for seismic anchor movement evaluation of piping is discussed in this section. The relative seismic anchor displacements should be calculated following the methodology in Section 4.2.1(3) of this document. If the location of interest includes a flange connection, then it should be demonstrated that the flange bolt stress for the moment amplitude is less than or equal to $\mathrm{S_y}/2$ for the bolting at temperature.

$$\frac{0.75 \times i \times M_{SAM}}{Z} \leq S_y$$

Where,

i	=	Stress Intensification Factor (SIF) as discussed in ASME B31.3 (Reference 15) Appendix D but 0.75i cannot be less than 1.0.
M_{SAM}	=	Moment amplitude from seismic anchor movement with the displacements determined per Section 4.2.1(3) of this document. The relative displacement of two points on the piping system may be determined with the SRSS if the two points are attached to two independent structures.
Z	=	Section modulus of piping including corrosion allowance but not the mill tolerance.
Sy	=	Specified minimum yield strength of the piping material at temperature.

7.1.6 Interaction Evaluation – The recommended procedures for interaction evaluation of piping are as follows:

1) Piping should be visually inspected to identify potential interactions with adjacent structures, systems, or components. Those interactions which could cause unacceptable damage to piping, piping components (e.g., control valves), or adjacent critical items should be mitigated.

   Note that restricting piping seismic movement to preclude interaction may lead to excessive restraint of thermal expansion or inhibit other necessary operational flexibility.

2) The walkdown should also identify the potential for interaction between adjacent structures, systems or components, and the piping being investigated. Those interactions that could cause unacceptable damage to piping should be mitigated. Note that falling hazards should be considered in this evaluation.

3) Displacements used when considering seismic interaction should be those calculated per Section 4.2.1 (3).

7.1.7 Allowable Stress – Piping made from materials other than ductile steel accepted by ASME B31E may be required to withstand seismic loading. The criteria outlined above for ductile steel piping should be followed for piping made from other materials with the following allowable stress values:

1) When ductile material piping is designed and constructed to a national standard with basic allowable stresses given, then those values should be used multiplied by the appropriate factor in Section 3.4 of ASME B31E.

2) When piping materials meet a national standard with a minimum specified tensile strength, σ_t, then the basic allowable stress at operating temperature should be:

   a. Ductile Materials: $S_h = \sigma_t / 3$ at temperature

   b. Brittle Materials: $S_h = \sigma_t / 10$ at temperature

3) When piping materials cannot be identified with a national standard with a minimum specified tensile strength, then one should be estimated from published literature or a testing program. The basic allowable stress at temperature should be determined using the appropriate equation in (2) above, unless a higher allowable can be justified by seismic testing.

7.2 Underground Piping Systems – Piping that is underground should be identified as such on walkdown reports and other documentation prepared for this evaluation. The evaluator can use the technical guidance provided in the aboveground piping section or other technical guidance appropriate for underground piping seismic evaluations. Concerns unique to underground piping that should be considered by the engineer include:

1) Liquefaction and lateral spreading
2) Seismic settlement
3) Surface faulting
4) Landsliding

Additional evaluation guidance for underground piping systems can be found in Reference 28.

8.0 STRENGTHENING CRITERIA

A strengthening and/or management program should be developed to correct deficiencies. If strengthening is required, appropriate strengthening criteria should be developed to provide a confidence level that retrofitted items will perform adequately when subjected to strong earthquake ground motions.

The intent of any retrofit construction associated with the CalARP programs is to do enough work to satisfy the CalARP Program requirements (to mitigate the risk of an accidental release of the regulated substance), but not meet the current code requirements. It is beneficial for the owner and/or the Responsible Engineer, as appropriate; to discuss the proposed work with the local Building Code Official to ensure the Building Code Official is in agreement.

For “building-like” nonbuilding structures (those with framing systems that are specifically listed in the building codes), the procedures and analysis methods outlined in documents such as ASCE 41-13 (Reference 25) may be useful in determining appropriate strengthening measures.

Often, the largest category of structural/seismic deficiencies in an existing facility will involve equipment which is not anchored or braced and thus has no lateral restraint. This may include equipment or structures for which bracing has been omitted or removed, or it may include structural bolts or anchor bolts, including their nuts, which were never installed. Another deficiency might be structural elements that are severely corroded or damaged. For such items, the strengthening measures may be obvious, or at least straightforward. Minor deficiencies associated with material deterioration (e.g. metal corrosion, or concrete carbonization) or missing hardware could be addressed by proper management programs.

Seismic restraining of minor mechanical/electrical/piping systems and equipment as defined in ASCE 7-10 Section 13.1.4.6 may not require engineering calculations and details as long as reasonable seismic bracing is provided. For these types of items FEMA has several guideline publications for seismic restraints of mechanical, electrical and piping systems that could be used as reference:

1) FEMA 412 Installing Seismic Restraints for Mechanical Equipment (Reference 32)
2) FEMA P-414 Installing Seismic Restraints for Duct and Pipe (Reference 33)
3) FEMA 413 Installing Seismic Restraints for Electrical Equipment (Reference 34)

For “building-like” nonbuilding structures (those with framing systems that are specifically listed in the building codes), the procedures and analysis methods outlined in documents such as ASCE 41-13 (Reference 25) may be useful in determining appropriate strengthening.

An important point to consider when retrofitting is that over-strengthening areas of the structure that are currently deficient in strength can force the weak link(s) to occur in other elements that are perhaps more brittle. This can have a negative impact on overall structural performance during a major earthquake. In other words, a structure that is presently weak, but ductile, should not be strengthened to the point that its failure mode becomes brittle with a lower energy absorbing capacity.

When seismic hazards such as liquefaction or seismically induced landslide can potentially affect a site, it is recommended that a geotechnical engineer be consulted. The basic reference for assessing these seismic hazards is SP117 (Reference 5). However, Section 12 of Reference 16, developed by the Los Angeles Section of ASCE, gives additional guidelines for mitigating landslide hazards. Section 8 of Reference 17, also developed by the Los Angeles Section of ASCE, gives additional guidelines for mitigating liquefaction hazards at a site.

When any retrofit construction work associated with the CalARP program is to be undertaken, a Building Permit is normally required; thus the local Building Department is involved automatically. It should always be kept in mind that the intent of retrofitting these structures, systems, or components is not to fully comply with the current building code as in many instances it is not practical to bring them up to current code. The retrofit design criteria should be consistent with this proposed guidance. However, it is always advisable to meet code requirements to the extent practical. If the retrofit construction does not meet the current Building Code, the detail drawings should clearly state that the retrofit is a voluntary seismic upgrade and may not meet current Building Code requirements for new construction.

The concept of "grandfathering" of existing structures is addressed specifically in Sections 402 to 404 of the 2016 California Existing Building Code (CEBC) (Reference 31) Those sections of the code basically set out conditions whereby the entire structure need not be brought up to current code when additions, alterations or repairs are made. In addition to requiring that the newly designed portion itself meet the current code, the primary requirements for "grandfathering" the unaltered portion of the structure are that the change cannot increase the gravity load in existing elements by more than 5% without meeting new code requirements for the gravity loads, and that the seismic demand-capacity ratios (DCRs) in existing elements cannot increase by more than 10% without meeting new code requirements. Note that the basis of comparison is the structure with the alteration versus the structure with the alteration ignored. The original design code is not relevant. Additional conditions are provided in Section 403 of the 2016 CEBC and restrictions for “voluntary seismic improvements” are provided in Section 403.9. The consensus of the Committee is that allowing this type of "grandfathering" of existing structures is appropriate.

Although the code allows DCRs greater than 1.0 in existing members when applying these provisions, it is prudent to apply some caution when DCRs are very high, or when the “grandfathering” clauses are applied multiple times over a period of years, potentially incrementally increasing loads significantly without an individual increase triggering an upgrade requirement. The engineer may want to consider factors such as the extent of high DCRs (local vs. global), redundancy, whether the controlling DCRs are for ductile vs. brittle failure modes, the time associated with the seismic masses (normal operating loads vs. rarely occurring maximum loads), etc.

9.0 RECOMMENDED REPORT CONTENTS

The CalARP seismic assessment report should contain the items listed below as applicable. As discussed in several of the paragraphs below, revalidation reports should include additional information related to prior CalARP seismic assessments.

1) Provide a statement that the report is either an initial CalARP seismic assessment report or a revalidation CalARP seismic assessment report.

2) Provide a description of the scope of the structural/seismic evaluation as determined in Section 1.1. This description may be in terms of the RS present at the facility and where in the facility those RS are located (area, building, floor, etc.). The scope description should include a listing or a tabulation of the items in the facility that were reviewed including structures, equipment and piping. Key items which were specifically excluded and therefore were not reviewed should also be noted.

3) Provide a characterization of the soil profile at the site and a geotechnical assessment describing each of the seismic hazards listed in Section 2 in accordance with Attachment A, and the basis for the determination of each. In particular, where ground response spectra are used as the basis for the CalARP seismic assessment, they should be referenced along with the basis for determining the ground response spectra (See Section 2.1). For a revalidation report, compare the current CalARP seismic hazards to the seismic hazards from prior evaluations and comment.

4) For each reviewed item, provide an assessment of its structural adequacy to resist the estimated seismic ground shaking for the site.

   a. The assessment should include a noting of any deterioration in the physical condition of the reviewed item that was observed in the field walkdown, such as excessive corrosion, concrete spalling, etc.

   b. The assessment should indicate the basis used. This would include visual observations made during a walkdown and corroborating photographs. Depending on the circumstances, the assessment may also be based on drawing reviews and structural/seismic calculations.

5) For a revalidation report, provide a discussion of items with a recommendation for remediation or additional evaluation from a prior evaluation and list the status of these prior recommendations. Prior recommendations should be categorized as having been sufficiently addressed, partially addressed with further action still required, or not addressed. Prior recommendations should always be completed unless the reviewer can demonstrate in writing that the prior recommendation is not needed anymore and the basis for this determination.

6) Provide recommendations for conceptual measures that will alleviate seismic deficiencies. These recommendations may include:

   a. Strengthening of structural elements
   b. Addition of new structural elements
   c. Reduction or redistribution of the seismic forces
   d. Measures for reducing the effects of a seismic hazard as identified in Section 2, etc.

7) Provide a recommendation for further study or detailed design for items that appear to be seismically deficient or for items which are clearly deficient but for which an adequate seismic risk-reduction measure is not obvious. Such further study may involve a structural issue or it may involve a study on how to address a seismic hazard in Section 2. For revalidation reports include prior recommendations that were not addressed or which were not addressed adequately since the last evaluation.

8) The CalARP report should be signed and stamped by the Responsible Engineer (see Section 1.4) and include the date of the field walkdown.

9) The CalARP report should discuss all deficiencies and recommendations identified during this evaluation regardless of whether or not they were contained in previous findings. Provide a photograph showing the identified deficiency if possible.

10) A list of the drawings that were reviewed should be included (including date and revision number) when drawing reviews form part of the basis for determining the seismic adequacy of structures or equipment.

11) Supplementary documentation of the observations made and the assessments performed. These may include photographs (where permissible) and copies of walkdown sheets.

10.0 REFERENCES

References may be obtained from:

Engineering Societies Library (Linda Hall Library), a private library located on the campus of the University of Missouri
5109 Cherry Street
Kansas City, Missouri 64110-2498
1-800-662-1545

Thomson Reuters website:
https://govt.westlaw.com/calregs/Browse/Home/California/CaliforniaCodeofRegulations?guid=I421E3AF0330549D9AE54D12AC2D8F349&originationContext=documenttoc&transitionType=Default&contextData=(sc.Default)

1. California Code of Regulations (CCR) Title 19, Division 2 Chapter 4.5, *California Accidental Release Prevention (CalARP) Program (October 1, 2017)*

2. Guidance for California Accidental Release Prevention (CalARP) Program Seismic Assessments, Prepared for the Administering Agency (AA) Subcommittee Region I Local Emergency Planning Committee (LEPC), Prepared by the CalARP Program Seismic Guidance Committee, December 2013, Approved by Region I LEPC, 03/12/2104.

3. Uniform Building Code, 1997 Edition, International Conference of Building Officials, Whittier, California, 1997.

4. ASCE/SEI 7-10 with Supplements No.1 and 2, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Reston, Virginia, 2010.

5. California Department of Conservation, California Geological Survey, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, 2008.

6. Guidelines for Seismic Evaluation and Design of Petrochemical Facilities, Second Edition, Task Committee on Seismic Evaluation and Design of Petrochemical Facilities, American Society of Civil Engineers, Reston, Virginia, 2011.

7. Gurbuz, O., Lopez, A., Summers, P., April 1992, "Implementation of the Proposed RMPP Seismic Assessment Guidance to Perform a Structural Seismic Evaluation of Existing Facilities" in Proceedings of HAZMACON '92, Session on "New Developments in Earthquake Caused Hazardous Materials Releases," Long Beach, CA.

8. Saunders, K.L. and Hau, G., "Seismic Evaluation Acceptance Guideline for Existing Above Ground Piping Systems", ASME PVP Volume 256-1, 1993.

9. API Standard 650, Welded Tanks for Oil Storage, 12th Edition, American Petroleum Institute, Washington, D.C., 2013.

10. AWWA D100-11, Welded Carbon Steel Tanks for Water Storage, American Water Works Association, Denver, Colorado, 2011.

11. Veletsos, A.S., Contributor, "Guidelines for the Seismic Design of Oil and Gas Pipeline Systems", ASCE, Committee on Gas and Liquid Fuel Lifelines, NY, NY, 1984.

12. Manos, G.W., August 1986, "Earthquake Tank-Wall Stability of Unbraced Tanks," American Society of Civil Engineers, Journal of Structural Engineering, Vol. 112, No. 8, including Erratum in Journal of Structural Engineering, Vol. 113, No.3, March 1987.

13. Housner, G.W., and Haroun, M.A., 1980, "Seismic Design of Liquid Storage Tanks," ASCE Convention and Exposition, Portland, Oregon, April 14-18, 1980.

14. ACI 350.3-06, Seismic Design of Liquid - Containing Concrete Structures and Commentary, American Concrete Institute, Farmington Hills, Michigan, 2006.

15. ASME B31.3 - 2012, Process Piping, ASME Code for Pressure Piping, B31, American Society of Mechanical Engineers, New York, New York, 2013.

16. ASCE, Los Angeles Section Geotechnical Group, "Recommended Procedures for Implementation of Division of Mines and Geology Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California", Published by Southern California Earthquake Center (SCEC), June 2002.

17. ASCE, Los Angeles Section Geotechnical Group, "Recommended Procedures for Implementation of Division of Mines and Geology Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California", Published by Southern California Earthquake Center (SCEC), March 1999.

18. ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, American Society of Civil Engineers, Reston, Virginia, 2016. (See Section 2.6 changes).

19. 2015 International Building Code, International Code Council, Inc., Country Club Hills, Illinois, June 2011.

20. National Cooperative Highway Research Program, NCHRP Report 611, "Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments," Anderson, Martin, Lam, and Wang, Transportation Research Board, Washington D.C., 2008.

21. ASME B31Ea – 2010, Addenda to ASME B31E – 2008 Standard for the Seismic Design and Retrofit of Above – Ground Piping Systems, American Society of Mechanical Engineers, New York, New York, 2010.

22. 2016 California Building Code Title 24, Part 2, Volumes 1 and 2, California Building Standards Commission, Sacramento, California, 2013.

23. ASME B31.8 - 2014, Gas Transmission and Distribution Piping Systems, ASME Code for Pressure Piping, B31, American Society of Mechanical Engineers, New York, New York, 2014.

24. ACI 318-14, Building Code Requirements for Structural Concrete, American Concrete Institute, Farmington Hills, Michigan, Errata as of 2015.

25. ASCE/SEI 41-13, Seismic Evaluation and Retrofit of Existing Buildings, American Society of Civil Engineers, Reston, Virginia, 2013

26. API Standard 620, Design and Construction of Large, Welded, Low-pressure Storage Tanks, Twelfth Edition, American Petroleum Institute, Washington, D.C., 2013 including Addendum 1 (2104) and Addendum 2 (2018).

27. US Department of Energy, Nuclear Reactors and Earthquakes, Report No. TID-7024, January 1961.

28. Guidelines for the Design of Buried Steel Pipe, American Lifelines Alliance, July 2001.

29. Priestley, M.J.N (Editor & Chairman) et. al, “Seismic Design of Storage Tanks,” Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering, December 1986.

30. Eurocode 8, (1998), “Design provisions for earthquake resistance of structures, Part 1-General rules and Part 4 - Silos, tanks and pipelines,” European committee for Standardization, Brussels.

31. California Existing Building Code, California Code of Regulations Title 24, Part 10, California Code of Building Standards Commission, Sacramento, 2016,

32. FEMA 412 , “Installing Seismic Restraints for Mechanical Equipment”, Prepared by Vibration Isolation and Seismic Control Manufacturers Association under a cooperative agreement between the Federal Emergency Management Agency and the American Society of Civil Engineers,1801 Alexander Bell Drive, Reston, Virginia, December 2002

33. FEMA P-414, “Installing Seismic Restraints for Duct and Pipe”, Prepared by Vibration Isolation and Seismic Control Manufacturers Association under a cooperative agreement

between the Federal Emergency Management Agency and the American Society of Civil Engineers,1801 Alexander Bell Drive, Reston, Virginia, January 2004

34. FEMA 413, “Installing Seismic Restraints for Electrical Equipment”, Prepared by Vibration Isolation and Seismic Control Manufacturers Association under a cooperative agreement between the Federal Emergency Management Agency and the American Society of Civil Engineers,1801 Alexander Bell Drive, Reston, Virginia, January 2004

TABLE 1

DUCTILITY-BASED REDUCTION FACTORS (Q) FOR EXISTING STRUCTURES AND SYSTEMS

A. STRUCTURES SUPPORTING EQUIPMENT This covers structures whose primary purpose is to support equipment, such as air coolers, spheres, horizontal vessels, exchangers, heaters, vertical vessels, etc.	**Q**
<u>1. Steel Structures</u> **Ductile Moment Frame** A value of Q = 8 is usually indicative of a moment frame that satisfies the seismic detailing provisions of AISC Seismic Provisions for Structural Steel Buildings(1997) Supplement 2 (Reference 4) or later for special moment frames. All other moment frames should be treated as ordinary moment frames.	8
Ordinary Moment Frame (See Note 3) A value of Q = 2 (also see Note 7) is usually indicative of a moment frame which have one or more of the following structural characteristics a. There is a significant strength discontinuity in any of the vertical lateral force resisting elements, i.e., a weak story. b. There are partial penetration welded splices in the columns of the moment resisting frames. c. The structure exhibits "strong girder-weak column" behavior, i.e., under combined lateral and vertical loading, hinges occur in a significant number of columns before occurring in the beams. d. The moment frame does not satisfy the seismic detailing provisions of Chapter 27 of the 1988 UBC. A value of Q = 3 (also see Note 7) is usually indicative of a moment frame that satisfies the seismic detailing provisions of Chapter 27 of the 1988 UBC or later for ordinary moment-resisting frames but has one or more of the following structural characteristics: e. Any of the moment frame elements are not compact. f. Any of the beam-column connections in the lateral force resisting moment frames does not have both: (1) full penetration flange welds; and (2) a bolted or welded web connection. g. There are bolted splices in the columns of the moment resisting frames that do not connect both flanges and the web. h. Moment connections made of Pre-Northridge Standard Ductile Moment Connections See Note 8 Figure 1 A value of Q = 4 is usually indicative of a moment frame that satisfies the seismic detailing provisions of Chapter 27 of the 1988 UBC but does not satisfy the seismic detailing provisions of AISC Seismic Provisions for Structural Steel Buildings(1997) Supplement 2 for ordinary moment frames. A value of Q = 5 is usually indicative of a moment frame that satisfies the seismic detailing provisions of AISC Seismic Provisions for Structural Steel Buildings(1997) Supplement 2 for ordinary moment frames.	2, 3, 4 or 5

TABLE 1
(Continued)

<table>
<tr><th>A. STRUCTURES SUPPORTING EQUIPMENT
(Continued)</th><th>Q</th></tr>
<tr><td>Braced frame (See Note 3)

A value of Q = 2 (also see Note 7) is usually indicative of a braced frame that has one or more of the following characteristics:

a. A significant strength discontinuity in any of the vertical lateral force resisting elements, i.e., a weak story (see ASCE 7-10 Table 12.3-2).
b. A bracing system that includes "K" braced bays. Note: "K" bracing is permitted for frames of two stories or less by using Q=2. For frames of more than two stories, "K" bracing must be justified on a case-by-case basis.
c. Brace connections that are not able to develop the capacity of the braces.
d. Column splice details that do not develop the column capacity
e A braced frame that does not satisfy the seismic detailing provisions of Chapter 27 of the 1988 UBC or later.

A value of Q = 3 (also see Note 7) is usually indicative of a concentrically braced frame that satisfies the seismic detailing provisions of Chapter 27 of the 1988 UBC or later but has one or more of the following structural characteristics:

f. Tension rod only bracing with connections that are able to develop the rod strength.
g. The bracing system has the working point of diagonal braces not located at the intersection of the centerlines of beams and columns unless accounted for in the evaluation.

A value of Q = 4 is usually indicative of a concentrically braced frame that satisfies the seismic detailing provisions of Chapter 27 of the 1988 UBC or later but has one or more of the following structural characteristics:

h. Diagonal elements designed to carry compression have (kl/r) greater than 120.
i. The bracing system includes chevron ("V" or inverted "V") bracing that was designed to carry gravity load and/or beams not designed to resist unbalanced load effects due to compression buckling and brace yielding.
j. Does not satisfy the seismic detailing provisions of AISC Seismic Provisions for Structural Steel Buildings(1997) Supplement 2 for ordinary concentrically braced frames

A value of Q = 5 is usually indicative of concentrically braced frame that satisfies the seismic detailing provisions of AISC Seismic Provisions for Structural Steel Buildings(1997) Supplement 2 for ordinary concentrically braced frames.</td><td>2, 3, 4 or 5</td></tr>
<tr><td>Cantilever Column / Inverted Pendulum (See Note 3)

A value of Q = 2 is usually indicative of a cantilever column which has the one or more of the following characteristics:

a. Column splice details cannot develop the column capacity.
b. Axial load demand represents more than 20% of the axial load capacity.

A value of Q = 3.5 may be used otherwise.</td><td>2 or 3.5</td></tr>
</table>

TABLE 1
(Continued)

<table>
<tr><th>A. STRUCTURES SUPPORTING EQUIPMENT (Continued)</th><th>Q</th></tr>
<tr><td><u>2. Concrete Structures</u>

Ductile Moment Frame

A value of Q = 8 is usually indicative of a moment frame that satisfies all the seismic design provision of Section 2625 of the 1988 UBC or similar provision of later codes for special moment resisting frames.</td><td>8</td></tr>
<tr><td>Intermediate Moment Frame

A value of Q = 4 is usually indicative of a moment frame that satisfies the seismic design provisions of Section 2625 (k) of the 1988 UBC or similar provisions of later codes for intermediate moment resisting frame.

Any moment frame that could have a shear failure occurring before a flexural failure in a beam or column should be considered an ordinary moment frame.</td><td>4</td></tr>
<tr><td>Ordinary Moment Frame (See Notes 3 and 7)

A value of Q = 1.5 is usually indicative of a moment frame that has one or more of the following structural characteristics:

a. There is a significant strength discontinuity in any of the vertical lateral force resisting elements, i.e., a weak story.
b. The structure exhibits "strong girder - weak column" behavior, i.e., under combined lateral and vertical loading, hinges occur in a significant number of columns before occurring in the beams.
c. There is visible deterioration of concrete or reinforcing steel in any of the frame elements, and this damage may lead to a brittle failure mode.
d. Shear failure occurs before flexural failure in a significant number of the columns.

A value of Q = 2.5 is usually indicative of a moment frame that has one or more of the following structural characteristics:

e. The lateral resisting frames include prestressed (pretensioned or post-tensioned elements).
f. The beam stirrups and column ties are not anchored into the member cores with hooks of 135º or more.
g. Columns have ties spaced at greater than d/4 throughout their length.
h. Beam stirrups are spaced at greater than d/2.
i. Any column bar lap splice is less than $35d_b$ long. Any column bar lap splice is not enclosed by ties spaced $8d_b$ or less.
j. Development length for longitudinal bars is less than $24d_b$.
k. Shear failure occurs before flexural failure in a significant number of beams.
l. Comprised of beams that do not have at least 2 continuous bars top and bottom that are developed into the supporting columns.
m. The columns do not have the shear strength to resist the shear associated with the development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending.

A value of Q = 3.5 may be used otherwise</td><td>1.5, 2.5 or 3.5</td></tr>
</table>

TABLE 1
(Continued)

<table>
<tr><th>A. STRUCTURES SUPPORTING EQUIPMENT (Continued)</th><th>Q</th></tr>
<tr><td>
Shear Wall (See Notes 3 and 7)
A value of Q = 1.5 is usually indicative of a shear wall that has one more of the following structural characteristics.
a. There is visible deterioration of concrete or reinforcing steel in any of the frame elements, and this damage may lead to a brittle failure mode.
b. There is a significant strength discontinuity in any of the vertical lateral force resisting elements, i.e., a weak story.
c. Any wall that is not continuous to the foundation.
A value of Q = 3 is usually indicative of a shear wall that has one more of the following structural characteristics.
d. The reinforcing steel for concrete walls is not greater than 0.0025 times the gross area of the wall along both the longitudinal and transverse axes. The spacing of reinforcing steel along either axis exceeds 18 inches.
e. For shear walls with H/D greater than 2.0, the boundary elements are not confined with either: (1) spirals; or (2) ties at spacing of less than $8d_b$.
f. For coupled shear wall buildings, stirrups in any coupling beam are spaced at greater than $8d_b$ or are not anchored into the core with hooks of 135º or more.
g. Shear walls that do not satisfy the seismic design provisions of Section 2625 of the 1988 UBC or satisfies seismic special shear wall provisions of later codes
A value of Q = 5 is usually indicative of a shear wall that satisfies the seismic design provisions of Section 2625 of the 1988 UBC or satisfies seismic special shear wall provisions of later codes.
</td><td>1.5, 3 or 5</td></tr>
<tr><td>
Cantilever Pier/Column (See Notes 3 and 7)
A value of Q = 1.5 is usually indicative of a cantilever pier/column that has one or more of the following structural characteristics:
a. There is visible deterioration of concrete or reinforcing steel in any of the elements, and this damage may lead to a brittle failure mode.
b. Axial load demand represents more than 20% of the axial load capacity.
A value of Q = 2.5 is usually indicative of a cantilever pier/column that has one or more of the following structural characteristics:
c. The ties are not anchored into the member cores with hooks of 135º or more.
d. Columns have ties spaced at greater than d/4 throughout their length. Piers have ties spaced at greater than d/2 throughout their length.
e. Any pier/column bar lap splice is less than $35d_b$ long. Any pier/column bar lap splice is not enclosed by ties spaced at $8d_b$ or less.
f. Development length for longitudinal bars is less than $24d_b$.
g. Cantilever pier/column that does not satisfy the seismic design provisions of Section 2625 of the 1988 UBC or later
h. Cantilever pier/column that has a natural period greater than 0.1 seconds in the direction being evaluated.
A value of Q = 3.5 is usually indicative of a cantilever pier/column that satisfies the seismic design provisions of Section 2625 of the 1988 UBC or later.
</td><td>1.5, 2.5 or 3.5</td></tr>
</table>

TABLE 1
(Continued)

<table>
<tr><th>B. EQUIPMENT BEHAVING AS STRUCTURES WITH INTEGRAL SUPPORTS</th><th>Q</th></tr>
<tr><td><u>1. Vertical Vessels/Heaters or Spheres supported by:</u>

Steel Skirts (See Notes 3 and 7)

A value of Q = 2 is usually indicative of a skirt that is sensitive to buckling as defined below:

a. The diameter (D) divided by the thickness (t) of the skirt is greater than $0.441*E/F_y$, where E and F_y are the Young's modulus and yield stress of the skirt, respectively. (see also Note 10)

A value of Q = 4 is usually indicative of a skirt that is <u>not</u> sensitive to buckling as defined above</td><td>2 or 4</td></tr>
<tr><td>Steel Braced Legs Without Top Girders or Stiffener Ring
(See Notes 3, 7 and 11)

A value of Q = 1.5 is usually indicative of a bracing layout that has one or more of the following characteristics:

a. Asymmetrical bracing causing stiffness irregularity
b. The bracing system includes "K" braced bays.
c. Brace connections are not able to develop the capacity of the diagonals.
d. Column splice details cannot develop the column capacity.

A value of Q = 3 is usually indicative of a symmetrical concentric bracing layout with bracing and column connections that are able to develop the brace and column capacities <u>but</u> has one or more of the following characteristics.

e. Diagonal elements designed to carry compression have (kl/r) greater than 120.
f. The bracing system includes chevron ("V" or inverted "V") bracing that was designed to carry gravity load and/or beams not designed to resist unbalanced load effects due to compression buckling and brace yielding.
g. Tension rod only bracing with connections which develop rod strength.

A value of Q = 4 is usually indicative of a symmetrical concentric bracing layout with bracing and column connections that are able to develop the brace and column capacities.</td><td>1.5, 3 or 4</td></tr>
<tr><td>Steel Unbraced Legs Without Top Girders or Stiffener Ring
(see also Note 3, 7 and 11)

A value of Q = 1.5 is usually indicative of unbraced legs that have the following characteristics:

a. End connections cannot develop the nominal flexural capacity of the legs.
b. Column splice details cannot develop the column capacity.
c. Axial load demands represent more than 20% of the axial load capacity.

A value of Q = 2.5 may be used otherwise.</td><td>1.5 or 2.5</td></tr>
</table>

TABLE 1
(Continued)

B. EQUIPMENT BEHAVING AS STRUCTURES WITH INTEGRAL SUPPORTS (Continued)	Q
2. Chimneys or Stacks	
Steel Guyed	4
Steel Cantilever	4
Concrete (See also Note 12) A value of Q = 1.5 is usually indicative of concrete chimneys or stacks that have large rectangular openings with detailing that does not conform to ASCE 7-10 Section 15.6.2. A value of Q = 4 may be used otherwise	1.5 or 4
3. Cooling Towers (Concrete, Steel, FRP or Wood Framed) A value of Q = 2 is usually indicative of wooden cooling towers with eccentric bracing connections. Eccentricities should be accounted for in the evaluation A value of Q = 3.5 may be used otherwise	2 or 3.5

C. PIPEWAYS	Q
1. Pipeways supporting equipment that weighs more than 25% of the other dead loads.	Use Q values per Section A
2. Pipeways not supporting equipment that weighs more than 25% of the other dead loads. Included in this are piperacks and miscellaneous supports that carry pipe, electrical conduits and trays.	Use Q values per Section A multiplied by 1.2.

TABLE 1
(Continued)

D. GROUND SUPPORTED TANKS (See Notes 4 and 9)	Q
1. Anchored	4
2. Unanchored	3
E. FOUNDATIONS (See Note 5)	**Q**
1. Piled	6
2. Spread Footings	6
F. ANCHORAGE TO CONCRETE (See Notes 6 and 9)	**Q**
1. Anchorage in tension and/or shear when there is a ductile force transfer mechanism between structure and foundation.	Same as for the Structure
2. Anchorage in tension and/or shear when there is a non-ductile force transfer mechanism between structure and foundation. A value of Q = 1.5 may be used when the concrete-governed strength is evaluated using Section 1923 of the 1997 UBC or when there is some other non-ductile force transfer mechanism, A value of Q = 2 may be used when the concrete governed strength is evaluated using Section 17 of ACI 318-14 and with Ω_o =1.0.	1.5 or 2
G. PIPING	**Q**
1. Piping in accordance with ASME B31, including in-line components with joints made by welding or brazing.	4.5
2. Piping in accordance with ASME B31, including in-line components, constructed of high- or limited-deformability materials, with joints made by threading, bonding, compression couplings, grooved couplings or flanges.	4
3. Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-deformability materials, with made by welding or brazing.	4
4. Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high- or limited-deformability materials, with joints made by threading, bonding, compression couplings, grooved couplings or flanges.	3.5
5. Piping and tubing constructed of low-deformability materials, such as cast iron, glass, and nonductile plastics.	3

TABLE 1
(Continued)

NOTES:

1. The use of the highest Q-factors in each category requires that the elements of the primary load path of the lateral force resisting system have been proportioned to assure ductile rather than brittle system behavior. This can be demonstrated by showing that each connection in the primary load path has an ultimate strength of at least equal to 150% of the load capacity (governed by either yielding or stability) of the element to which the load is transferred. Alternatively, Q-factors should be reduced consistent with the limited ductility of the governing connection and/or the governing connection should be modified as required.

2. A Q-factor different from the tabulated values (higher or lower) may be justified on a case-by-case basis. This document provides guidance for structures and components that support or contain RS materials and the Q-factors are deemed appropriate for structures with an ASCE 7-10 Importance Factor of 1.25.

3. If more than one of the conditions specified in the table applies, the lowest Q-factor associated with those conditions should be used.

4. Other approved national standards for the seismic assessment of tanks may be used in lieu of these guidelines.

5. These values of Q apply to overturning checks, soil bearing, and pile capacities. For the remaining items including connection between piles and pile caps, use the Q factor for the supported structure.

6. For anchorage in tension or shear, a ductile force transfer mechanism occurs when the concrete-governed strength is greater than 1.2 times the anchorage steel strength or when there are properly detailed concrete reinforcing bars being provided that prevent a concrete failure for 1.2 times the anchorage steel strength. When this is the case, then the Q-factor to be used for the evaluation of the anchorage and the rest of the structural system corresponds to that for the structural system itself. The anchorage should also be a ductile steel element with a tensile test elongation of at least 14 percent and reduction in area of at least 30 percent.

   For anchorage in tension or shear, a non-ductile force transfer mechanism occurs when a concrete-governed strength controls the evaluation of anchorage (as opposed to anchorage steel and in situations where inadequate reinforcement is provided) or when there is some other non-ductile force transfer mechanism between the structure and its support.

   Tension and shear interaction may be checked for steel failure independent of concrete cone failure. Combining failures in different materials is overly conservative when evaluating existing facilities. If either tension or shear reinforcing is provided to prevent concrete failure then no interaction effects need to be considered for the concrete strength evaluation.

TABLE 1
(Continued)

NOTES: (Continued)

Additionally for skirt supported vessels, flat bottom tanks or other structural systems where the anchorage is the primary source for ductility, the Q-factor determined for the anchorage shall also be used for the evaluation of vessel or tank itself or structural system. Also see Note 7.

Where anchorage corrosion is found, the effective area of the anchorage shall be reduced accordingly and taken into account in determining the anchorage strength. If the anchorage corrosion is severe enough to prevent adequate ductile yielding of the anchorage then a Q-factor of 1.5 shall be used for the anchorage evaluation.

7. Alternatively, for structures that may contain localized/single features with limited ductility, such as limiting connections or splices, non-compact steel members, high (Kl/r) members and non-ductile anchor bolts, that do not occur at a significant number of locations, the load capacity of the specific limiting feature(s) may be evaluated and/or improved in lieu of using system-wide lower Q-factors that tend to generically penalize all elements of the structural system. The evaluation for these localized features may be performed using a Q-factor equal to 0.4 times the Q-factor normally recommended (i.e., unreduced) for the system. The evaluation for the remainder of the system may then be performed using the Q-factor normally recommended without consideration of the localized feature with limited ductility

8 Figure 1 below shows a common connection detail which has been used in the building industry. In the aftermath of the January, 1994 Northridge, California earthquake, over 100 buildings were found, where cracks occurred in connections based on this detail.

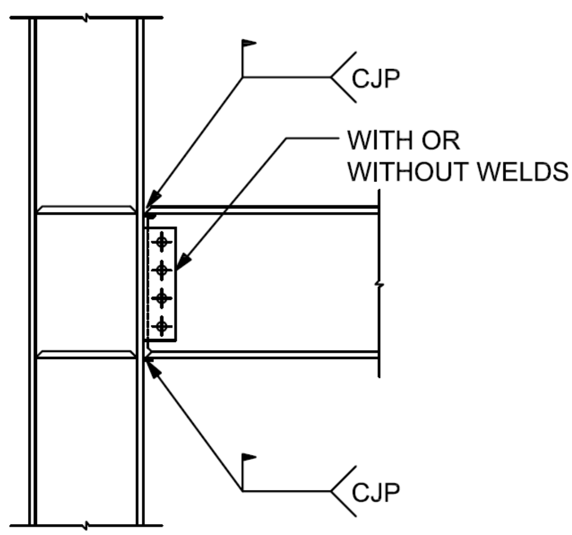

Figure 1: Former Standard Ductile Moment Connection Detail. (As a result of the Northridge Earthquake, this connection was shown to have major problems.)

TABLE 1
(Continued)

NOTES: (Continued)

9. For tanks made of fiberglass or similar materials, non-ductile anchorage and its attachments should be evaluated for a Q equal to 1.5.

10 An alternative approach to evaluate skirt buckling would be to use ASCE 7-16 (Reference 18) Commentary Section C15.7.10.5 with Q=1 in place of R=1 and with an Importance Factor of I=1.

11 Vertical vessels/heaters or spheres supported by braced legs with a top girder or stiffener ring can be treated as a braced frame. Vertical vessels/heaters or spheres supported by unbraced legs with a top girder or stiffener ring can be treated as a moment frame.

ATTACHMENT A

RECOMMENDED GEOTECHNICAL REPORT CONTENTS

A proper assessment of the above earthquake hazard effects will generally require, as a prerequisite, knowledge of the underlying soil profile at the facility. Therefore, a specialized geotechnical assessment should be prepared as part of the CalARP report. At a minimum this assessment should provide the following information:

1) A characterization of the soil profile at the site and the basis for the characterization. The characterization should indicate the Site Class for the facility.

2) Description of the ground shaking, including local site amplification effects. Include recommendations pertaining to seismic design parameters based on ASCE/SEI 7, or the latest California Building Code adopted by the local jurisdiction. Parameters such as Ground Motion Parameters S_S and S_1, Site Coefficients F_a and $F_{v,}$ and site DE parameters $S_{DS,}$ S_{D1} and T_L should be provided. The basis for the seismic design parameters should be provided.

3) Identification of fault rupture zones which pass near or under the site and the basis for the identification and recommendations for horizontal and vertical offsets to be considered for evaluations.

4) Description of the potential for liquefaction at the site, including the potential magnitude of settlement and/or spreading in the design event, the risks associated with lateral spreading and localized liquefaction-induced settlements/differential settlements and potential foundation bearing failures. The basis for the hazards due to liquefaction should be provided.

5) Description of the potential for non-liquefaction related seismic settlements and the basis for the level of predicted seismic settlements.

6) Description of the potential for other seismic hazards at the site, including but not limited to landslides, failures of slopes and embankments, tsunamis, and seiches. Include the basis for each described hazard.

7) Provide geotechnical properties appropriate for evaluating existing structures as requested by the engineer. This may include, but is not limited to, water table depths, soil bearing capacities, pile capacities, and passive and active pressures for use in evaluating retaining walls.

There are several possible methods for collecting the information necessary to perform the assessment. One method is to have the owner provide reference geotechnical reports prepared for the facility. Alternatively, if the soil profile is known to be uniform over the entire area, a geotechnical report developed for an adjacent facility may be adequate as a reference. It is preferable if the adjacent site having a geotechnical report is within 300 feet of the facility in question. Consultation with the AA and with the local Building Official may also provide some information in this regard.

ATTACHMENT A
(Continued)

If the owner cannot provide an adequate reference geotechnical report, then the options are as follows:

1 The owner or engineer may engage a licensed geotechnical engineer to provide a geotechnical investigation report that is adequate for the seismic assessment.

2) The engineer may make a series of conservative (essentially “worst case”) assumptions in determining the effects of the underlying soil profile on the various seismic hazards. Such assumptions may be based on the soil characteristics known for the general area. Alternatively, the site class may be assumed which gives the largest evaluation forces. Depending on the situation, this option may or may not be the most cost-effective approach for the owner (e.g., for a single small item, it is generally not cost effective to prepare a geotechnical investigation report).

ATTACHMENT B

WALKDOWN FORMS FOR EQUIPMENT PIPING

CalARP FIELD DATA SHEET FOR EQUIPMENT

EQUIPMENT ID:
DESCRIPTION:
LOCATION:

SCREENING EVALUATION: SUMMARY

Summary of Evaluation: ____ Adequate ____ Not Adequate

____ Further Evaluation Required

Recommendations:

SCREENING EVALUATION: ANCHORAGE

Noted Anchorage Concerns:

_____ Installation Adequacy	_____ Weld Quality
_____ Missing or Loose Bolts	_____ Corrosion
_____ Concrete Quality	_____ Other Concerns
_____ Spacing/Edge Distance	

Comments:

SCREENING EVALUATION: LOAD PATH

Noted Load Path Concerns:

_____ Connections to Components	_____ Missing or Loose Hardware
_____ Support Members	_____ Other Concerns

Comments:

ATTACHMENT B
(Continued)

Seismic Evaluation

CalARP Walkdown Review Sheet

Piping

Line Number:		Date:	
Drawing Number:		By:	

Evaluation Summary (Circle one)

Adequate Not Adequate Further Evaluation Required

Inspection Attributes				
	Yes	No	N/A	Comments
Piping				
Damaged				
Corrosion				
Flanged/Threaded Joints				
Buried Run				
Adequate Branch Flexibility				
Rigidly Spans Components				
Supports				
Piping Spans OK				
Missing Hardware				
Corrosion				
Hardware Damaged/Loose				
Seismic Interaction				
Adequate Clearance				
Adjacent Comps. Secure				
Clearance at AOVs/MOVs				

ATTACHMENT B
(Continued)

Line Number:		Date:	

Notes and Sketches