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

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TABLE OF CONTENTS

1.0 INTRODUCTION

The objective of a California Accidental Release Prevention (CalARP) Program seismic assessment is to provide reasonable assurance that a release of Regulated Substances (RS) as listed in California Code of Regulations (CCR) Title 19 Division 2 Chapter 4.5 (Ref. 1) having offsite consequences (caused by a loss of containment or pressure boundary integrity) would not occur as a result of an earthquake. Since 1998, the seismic assessment study has been part of the mandated State's CalARP program. The purpose of this document is to provide guidance regarding criteria to be used in such assessments. This guidance document is an update of the CalARP seismic document published in January of 2004 (Ref. 2). The guidance provided is applicable to structural systems and components whose failure could result in the release of sufficient quantities of RS to be of concern.

The guidance given 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 level as defined in ASCE/SEI 7-05 (Ref. 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 Authority Having Jurisdiction (AHJ) to describe why this approach is being planned and explain differences between this approach and the deterministic method. The AHJ may also be referred to as the Administering Agency (AA).

The CalARP regulation states in Section 2760.2 (b): "The owner or operator shall work closely with AAs in deciding which PHA [Process Hazard Analysis] methodology is best suited to determine the hazards of the process being analyzed." Thus, prior to the beginning of any seismic assessment, the owner/operator needs to consult closely with the AHJ to obtain mutual understanding and agreement on the scope of the assessment, the general approach proposed by the Responsible Engineer and the schedule for the assessment.

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

1.2 Evaluation Scope – The owner/operator, in consultation with the AHJ and Responsible Engineer (see Section 1.5), should always identify the systems to be evaluated in accordance with this guidance. The systems are expected to fall into three categories. These are:

1) Covered processes as defined by CalARP Program regulations.

2) Adjacent facilities whose structural failure or excessive displacement could result in the significant release of RS.

3) Onsite utility systems and emergency systems which would be required to operate following an earthquake for emergency reaction or to maintain the facility in a safe condition, (e.g., emergency power, leak detectors, pressure relief valves, battery racks, release treatment systems including scrubbers or water diffusers, firewater pumps and their fuel tanks, cooling water, room ventilation, etc.).

1.3 Performance Criteria – In order to achieve the overall objective of preventing releases of RS, individual equipment items, structures, and systems (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 material
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.

From July 1, 1999 to December 31, 2007, all new facilities submitted for permit in California should have been designed in accordance with either the 1998 or 2001 California Building Code (CBC) (Ref. 19 and 20) which both reference the 1997 Uniform Building Code (UBC) (Ref. 3) seismic requirements. Starting on January 1, 2008, all new facilities in California are to be designed in accordance with the 2007 California Building Code (Ref. 21) which references the 2006 International Building Code (IBC) (Ref. 22) seismic requirements. The 2006 IBC in turn references American Society of Civil Engineers (ASCE) Standard ASCE/SEI 7-05 (Ref. 4) for its seismic load provisions. It is the consensus of this Committee that RS systems and components designed and properly constructed in accordance with the 1997 UBC or ASCE/SEI 7-05 (or later) provisions 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) provided that the facility in which the systems and components are contained is not located in the near field of an active earthquake fault or on a soft soil site. It should be noted that design earthquake terminology changed between the UBC and ASCE/SEI 7-05. The design earthquake ground motion level in the UBC is called the "design basis earthquake" while in ASCE/SEI 7-05 it is now called "design earthquake."

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.

1.4 Extent of Seismic Evaluations Required – All equipment and components identified in Section 1.2 are subject to the seismic assessment guidelines of this document. However, the extent of these evaluations may be limited or expanded depending on the situation. Each owner/operator will have different conditions at their facility and should consult with the AHJ to determine which of the following subsections apply to their facility.

1.4.1 Existing Facilities Which Have Not Had Previous CalARP Seismic Assessments

1) Constructed to 1985 UBC and Earlier

There is considerable uncertainty about the capacity of non-building structures and non-structural components designed and constructed prior to the 1988 UBC. This is because there were no specific seismic code requirements for non-building structures and non-structural 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 non-building structure and non-structural component designs should always be considered suspect and subject to CalARP type evaluations if they are in the evaluation scope (Section 1.2).

2) Constructed to 1988 UBC and Later

Existing facilities which are subject to the CalARP requirements and which were permitted for construction in California after mid-1990 (i.e., designed and constructed 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 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 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 or the 1997 UBC 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.

1.4.2 New Facilities Submitted for Permit After September 2009 That Are Subject to CalARP Program Requirements – Design and construction of new facilities containing RS must satisfy the seismic provisions of the 2007 California Building Code (ASCE/SEI 7-05). 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.

1.4.3 Facility Revalidation With a Previous CalARP Seismic Assessment – The CalARP program requires that facilities, which are subject to the CalARP requirements, have their process hazard analysis updated and revalidated at least every 5 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, any portion of the previous assessment maybe used for the current assessment. However, any revalidation should include the performance of a walkdown in accordance with Section 3 of this document. Equipment and systems that have been previously judged to have met the September 1998 or January 2004 Seismic Guidance requirements and for which a visual field inspection reveals adequate lateral force resisting systems may be deemed to meet the intent of these requirements without further evaluation.

1.4.4 Occurrence of Conditions That Would Trigger an Assessment Within the Revalidation Period – It is recommended that owners/operators assessing the validity of past evaluations consider conditions that may make a partial or entirely new assessment necessary. Examples of such conditions include:

1) Major increases in the estimated ground motions (new significant active fault discovered near the facility).

2) System modifications that would significantly affect the seismic behavior of the equipment or system, such as changing or addition of equipment or processes.

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 (e.g., corrosion) in equipment, piping, structural members, foundations or anchorages.

1.5 Responsible Engineer – The Responsible Engineer has responsibility for conducting and/or overseeing the evaluations and walkdowns required by this document for a given facility. 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.

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/SEI 7-05. The procedures of ASCE/SEI 7-05 should be used consistently for determination of these ground motions, including Chapter 21 of ASCE/SEI 7-05 for site-specific assessments. Values to be used in these evaluations may be obtained online from the United States Geological Survey (USGS) website at http://earthquake.usgs.gov/research/hazmaps/design. Latitude and longitude of the facility should always be used, 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. Fault maps are described and can be found online at the California Geological Survey (CGS) website at http://www.conserv.ca.gov/cgs/rghm/ap/index.htm in Special Publication 42 and the associated fault maps.

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. 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 CGS has established evaluation guidelines in Special Publication 117 (SP117) (Ref. 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

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 (Ref. 5) presents guidelines for evaluation and mitigation of earthquake-induced landslide hazards. 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 semienclosed 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.

In California, the Seismic Safety Elements of General Plans typically provide an estimate of the potential for tsunami and seiche inundation. Estimates of maximum tsunami run-up can be made using historical information or theoretical modeling.

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.

2.6.3 Ongoing Developments for Mitigating Tsunami Hazards in the United States - The National Oceanographic and Atmospheric Administration (NOAA) is currently in the process of developing an early tsunami warning system for distant tsunami sources for the west coast of the United States. When the system becomes available, facilities which are vulnerable to a tsunami should be tied into the system and they should develop emergency plans in the event there is a tsunami warning issued that would affect their area.

"Tsunami Risk Reduction for the United States: A Framework for Action" (Ref. 23), the joint report by the sub-committee on Disaster Reduction and the US Group on Earth Observations, called for development of standardized and coordinated tsunami hazard and risk assessment for all coastal regions of the United States and its territories. In response to this report, at the request of the National Tsunami Hazard Mitigation Program (NTHMP), NOAA's National Geophysical Data Center (NGDC) and the United States Geological Survey (USGS) collaborated to conduct the first tsunami hazard assessment of the United States and its territories with the following conclusion: both the frequency and the amplitudes of tsunami run-ups support a qualitative "high" hazard assessment for Washington, Oregon, California, Puerto Rico, and the Virgin Islands. The "high" value for Oregon, Washington, and northern California reflects the low frequency but the potential for very high run-ups from magnitude 9 earthquakes on the Cascade subduction zone. Updates will be required as additional knowledge is obtained of possible tsunami sources in offshore southern California.

As the result of this review, the *National Tsunami Research Plan* (Ref. 24) has been developed. The Plan has identified the following high priority research areas for improving the knowledge essential to tsunami risk reduction:

1) Enhance and sustain tsunami education
2) Improve tsunami warnings
3) Understand the impacts of tsunami at the coasts
4) Develop effective mitigation and recovery tools
5) Improve characterization of tsunami sources
6) Develop a tsunami data acquisition, archival, and retrieval system

3.0 WALKDOWN CONSIDERATIONS

A critical feature of the evaluation methodology is the onsite review of the existing facility by a qualified engineer 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 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 process can be found in ASCE guidelines (Ref. 6). 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 NON-BUILDING STRUCTURES

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

4.2 Analysis Methodology and Acceptance Criteria – Acceptance for existing ground supported building and non-building 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 through7.

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 equation is satisfied:

DEMAND*		CAPACITY BASED ON
$D + L + \frac{E_e}{Q}$	$\leq$	$øR_n$

* using Load Factors of unity for all loads

Where,

D = Dead load

L = Live and/or operating load

E_e = Unreduced elastic earthquake load based upon ground motion determined in Section 2

$ø$ = Capacity reduction factor (per ACI) or resistance factor (per AISC)

Q = Ductility based reduction factor per Table 1

R_n = Nominal capacity 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 amplification. For spectra that have flattened peaks (e.g. ASCE/SEI 7-05 Figure 11.4-1), the smallest period of the flattened peak (T_0) 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 accelerations 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 accelerations 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 frames 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 Q should be multiplied by the factor 0.5Q, [where the value of 0.5Q should not be taken as less than one (1.0)], to determine displacements to be used in an evaluation.

a. Generally, the drift (relative horizontal displacement) should be less than 0.01H, 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 non-structural 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 should be evaluated. When evaluating overturning, a minimum of 10 percent reduction in dead load should be assumed to account for vertical acceleration effects. This reduction factor may be higher for facilities close to active faults that may

be subject to higher vertical acceleration. The factor of safety against overturning and sliding should be larger than or equal to 1.0.

5) The capacity of anchor bolts embedded in concrete may be evaluated in accordance with the strength design provisions of Section 1923 of the 1997 UBC with inspection load factors specified in Section 1923.2 taken as unity. The capacity of post-installed anchors should be determined in accordance with the latest International Code Conference Evaluation Services (ICCES) Evaluation Reports published in 2000 or later. Where the anchorage capacity is greater than 1.25 times the minimum yield strength (but need not exceed the ultimate strength of the bolts) the Q value of the structure may be used to determine the bolt load. Where the anchorage capacity is less than 1.25 F_y, the Q value for determining bolt loads should be taken as 1.5.

6) The directional effects of an earthquake should be considered either using the Square Root of the Sum of the Square (SRSS) rule or the 100%-30%-30% rule.

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 7-05 (Ref. 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 and piping and vessel designs where working stress allowable design is standard practice, capacity may be taken as 1.6x 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 7-05 (Ref. 4) Section 16.2 or ASCE/SEI 41-06 (Ref. 25). For significant structures, where these types of analyses are preferred, a peer review should be done.

4.2.3 Recommended Guidelines for Seismic Evaluation and Design of Petrochemical Facilities – ASCE (Ref. 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 NON-STRUCTURAL COMPONENTS

Permanent equipment and non-structural components supported within or by structures 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 non-building structure per Section 4.

The supported permanent equipment and non-structural components should be considered subsystems if their total weight is less than 25% of the total weight of the supporting structure and subsystems. For these subsystems, the anchorage and attachments may be evaluated in accordance with the equivalent static force provisions of Chapter 13 of ASCE/SEI 7-05. The equipment or the non-structural component itself should be checked for the acceleration levels based on the above referenced sections. Alternatively, 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/SEI 7-05 if the equivalent static force provisions of Chapter 13 result in excessive demand. Also, nonlinear dynamic analysis is permitted of combined non-structural systems in accordance with Section 4.2.2.

If the permanent equipment or non-structural component weight is greater than 25% of the weight of the supporting structure, Section 4 with Q values equal to the smaller of the values for the equipment or the supporting structure from Table 1 can be used for the entire system. Alternatively, 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 value to obtain the design values.

Where an approved national standard provides a basis for the earthquake-resistant design of a particular type of non-building 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 TANKS

6.1 Scope – Vertical liquid storage tanks with supported bottoms should be addressed 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.

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, detailed methodology for analytical evaluation as well as suggested modifications to mitigate seismic hazards.

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 (Ref. 9) - This method is a standard for the design of new tanks for the petrochemical industry. Its provisions are accepted by the UBC and ASCE/SEI 7-05 and it addresses both anchored and unanchored tanks.

2) AWWA D100 (Ref. 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 (Ref. 11) - This method is primarily for anchored tanks.

4) Manos (Ref. 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 (Ref. 13) - This method is primarily for the analysis of anchored tanks, but is often used for both anchored and unanchored tanks.

6) ACI 350.3-01, (Ref. 14) - Applies to Concrete Tanks (both round and rectangular)

7) API 620 Appendix L (Ref. 26)

8) “Nuclear Reactors and Earthquakes”. United States Atomic Energy Commission, TID-7024, August 1961 (Ref. 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 factors 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 (Ref. 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 3-14 of ASCE/SEI 41-06 (Ref. 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.

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/SEI 41-06), 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/SEI 41-06), the seismic force in the specific element at target displacement may be reduced by a "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 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.

7.0 EVALUATION OF PIPING SYSTEMS

7.1 Aboveground Piping Systems – Evaluation of piping systems should be primarily accomplished by field walkdowns. One reason this method is recommended is because some piping is field routed and, in some instances, piping and supports have been modified from that shown on design drawings.

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

1) Identify piping systems to be evaluated. The list should include piping systems that can directly, or indirectly, lead to a significant release of RS as discussed in Section 1.2. The list should also include piping downstream of relief valves and other safety systems used to remove RS to a safe location.

2) Perform a walkdown of the piping systems for seismic capability. Document the walkdown and identify areas for detailed evaluation.

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 flanged 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 (Ref. 15). Evaluation of other piping material is discussed in Section 7.1.7.

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. 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, 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.

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.

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 B31.1 Table 121.5 (Ref. 28)
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

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
9) Corrosion Under Insulation (CUI)
10) Inadequate vertical supports and/or insufficient lateral restraints
11) Welded attachments to thin wall pipe
12) Excessive seismic displacements of expansion joints
13) Brittle elements such as cast iron pipes
14) Sensitive equipment impact (e.g., control valves)
15) Potential for fatigue of short to medium length rod hangers that are restrained against rotation at the support end

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 following general rules and the inertial force determination procedures of Section 7.1.6 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 pipelines 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 7.1.4 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 Seismic Anchor Movement – The recommended procedures for seismic anchor movement (SAM) evaluation of piping are as follows:

1) Use the relative seismic anchor displacements as determined in Section 4.2.1

2) Piping stress due to seismic anchor displacement should meet the following criteria:

$$\frac{iM_{SAM}}{Z} \leq 3.0\ S_h$$

Where,

i = stress intensification factor from ASME B31.3 or other appropriate reference

M_{SAM} = moment amplitude due to seismic anchor movement using nominal pipe wall thickness from Section 13.3.2 of ASCE/SEI 7-05 (Ref. 4)

Z = elastic section modulus of pipe = $\pi r^2 t$

S_h = basic material allowable stress at pipe operating temperature from ASME B31.3 or other appropriate reference

r = mean cross-sectional radius

t = design nominal wall thickness minus design corrosion/erosion allowances or actual wall thickness minus future anticipated corrosion/erosion

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

1) RS 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 RS piping being investigated. Those interactions that could cause unacceptable damage to RS piping should be mitigated. Note that falling hazards should be considered in this evaluation.

7.1.6 Inertia Evaluation –The seismic force to be used shall be determined from Section 13.3.1 of ASCE/SEI 7-05 (Ref. 4) substituting Q from Table 1 for R_p and using $I_p = 1.0$.

7.1.7 Allowable Stress – Piping made from materials other than ductile steel accepted by ASME B31.3 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.

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: $S_h = \sigma_t / 40$ 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

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.

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.

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.

For “building-like” non-building structures (those with framing systems that are specifically listed in the building codes), the procedures and analysis methods outlined in documents such as ASCE/SEI 41-06 (Ref. 25) may be useful in determining appropriate strengthening measures.

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 (Ref. 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 bring them up to current code." In many instances, ”to bring them up to current code” may not be practical. 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 Section 3403.2.3 of the 2007 California Building Code. That section of the code basically sets out conditions whereby the entire structure need not be brought up to current code when additions or alterations 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 addition or alteration cannot increase the seismic load in the remainder of the structure by more than 10% or decrease the capacity by more than 10%. The consensus of the Committee is that allowing this type of "grandfathering" of existing structures is appropriate.

If the intent of any retrofit construction associated with the CalARP programs is to do enough work to satisfy the CalARP Program requirements but not meet the current code requirements, it behooves the owner and/or the engineer to discuss the proposed work with the local Building Official to ensure the Building Official is in agreement.

9.0 RECOMMENDED REPORT CONTENTS

The CalARP seismic assessment report should contain the items listed below as applicable.

9.1 Report Contents – CalARP seismic reports should at least contain the following information:

1) Provide the reason for performing the seismic evaluation. For example, is it an evaluation of a facility not previously reviewed? Or is it a revalidation of a previous study? If a revalidation, it is necessary to reference the previous report, and indicate how that previous report was used and the extent to which it was relied upon.

2) Provide a description of the scope of the structural/seismic evaluation as determined in Section 1.2. 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/or 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 the basis for that characterization. For example, reference to a geotechnical report (e.g. see Attachment A for recommended contents of a Geotechnical report), including its date of issue, if such report serves as the basis for the site soil profile characterization (as per the guidelines in Section 2). In addition, if the geotechnical report serves as the basis for assessing the potential for any of the seismic hazards in Section 2, this should be noted. Depending on the extent to which the geotechnical report is relied upon, it may be appropriate to append a copy of this geotechnical report, or at least key excerpts from it, to the CalARP seismic report.

4) Provide a discussion of the determination of each of the seismic hazards listed in Section 2, 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).

5) 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 previous seismic evaluation reports, drawing reviews and/or structural/seismic calculations.

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.

8) Provide an assessment of existing detection and mitigation systems and, when necessary, recommendations for new mitigation systems such as seismically triggered safe shut-off systems.

9) The CalARP report should be signed and stamped by the Responsible Engineer (see Section 1.5).

10) The CalARP report should discuss all deficiencies and recommendations identified during this evaluation regardless of whether or not they were contained in previous evaluation findings.

11) 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.

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

9.2 Initial Walkdown versus Revalidation Walkdown – When the Responsible Engineer intends on using portions of previous work, the prior report needs to be attached to the new report for reference. It is also expected that all walkdown sheets and photographs be included in color print.

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

1. California Code of Regulations (CCR) Title 19, Division 2 Chapter 4.5.

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, January 2004, Approved by Region 1 LEPC, 12/10/03.

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

4. ASCE/SEI 7-05, *Minimum Design Loads for Buildings and Other Structures,* American Society of Civil Engineers, 2006.

5. California Department of Conservation, Division of Mines and Geology, “*Guidelines for Evaluating and Mitigating Seismic Hazards in California,”* Special Publication 117, 1997.

6. ASCE 1997, "Guidelines for Seismic Evaluation and Design of Petrochemical facilities", Task Committee on Seismic Evaluation and Design of Petrochemical Facilities, American Society of Civil Engineers. New York, 1997.

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 Steel Tanks for Oil Storage,* 10th Edition, Addendum 2, November 2001.

10. ANSI/AWWA D100-96, 1996, *AWWA Standard for Welded Steel Tanks for Water Storage,* American Water Works Association.

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 350.3-01, Seismic Design of Liquid - Containing Concrete Structures, and Commentary, January 2002.

15. ANSI/ASME B31.3, *ASME Code for Pressure Piping Chemical Plant and Petroleum Refinery Piping*, 2002 Edition.

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. FEMA 350 "Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings," June 2000.

19. *1998 California Building Code,* California Building Standards Commission.

20. *2001 California Building Code,* California Building Standards Commission.

21. *2007 California Building Code,* California Building Standards Commission.

22. *International Building Code,* 2006 Edition, International Code Council, Inc.

23. National Science and Technology Council, *Tsunami Risk Reduction for the United States: A Framework for Action,* December 2005.

24. Bernard B, Dengler L, and Yim S (editors), 2007, *National Tsunami Research Plan: Report of a Workshop Sponsored by NSF/NOAA.*

25. ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings,* American Society of Civil Engineers, 2006.

26. API Standard 620, *Design and Construction of Large, Welded, Low-Pressure Storage Tanks,* Eleventh Edition, 2009.

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

28. ASME B31-1a-2002, *Power Piping,* American Society of Mechanical Engineers, 2003.

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 and reactors, etc.	**Q**
1. Steel structures	
Ductile moment frame (see Note 8) Use Q=6 if there is a significant departure from the intent of the 1988 (or later) UBC for special moment-resisting frames.	6 or 8
Ordinary moment frame (see Note 8) The following structural characteristics are usually indicative of a Q=2 value (also see Note 6): 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. The following structural characteristics are usually indicative of a Q=4 value (also see Note 6): d. Any of the moment frame elements is not compact. e. 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. f. There are bolted splices in the columns of the moment resisting frames that do not connect both flanges and the web.	2, 4 or 5
Braced frame The following structural characteristics are usually indicative of a Q=2 value (also see Note 6): a. There is a significant strength discontinuity in any of the vertical lateral force resisting elements, i.e., a weak story (see SEAOC, 1996 Section C104.9). b. The bracing system 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 are not able to develop the capacity of the diagonals. d. Column splice details cannot develop the column capacity. The following structural characteristics are usually indicative of a Q=4 value (also see Note 6): 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. g. Tension rod bracing with connections which develop rod strength.	2, 4 or 5
Cantilever column The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 6): a. Column splice details cannot develop the column capacity. b. Axial load demand represents more than 20% of the axial load capacity.	1.5 or 2.5

TABLE 1
(Continued)

A. STRUCTURES SUPPORTING EQUIPMENT (Continued)	Q
2. Concrete structures	
Ductile moment frame Use Q=6 if there is a significant departure from the intent of the 1988 (or later) UBC for special moment-resisting frames. If shear failure occurs before flexural failure in either beam or column, the frame should be considered an ordinary moment frame.	6 or 8
Intermediate moment frame	4
Ordinary moment frame The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 6): 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. The following structural characteristics are usually indicative of a Q=2.5 value (also see Note 6): 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 135^o or more. g. Columns have ties spaced at greater than d/4 throughout their length. Beam stirrups are spaced at greater than d/2. h. 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. i. Development length for longitudinal bars is less than $24d_b$. j. Shear failure occurs before flexural failure in a significant number of the beams.	1.5, 2.5 or 3.5
Shear wall The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 6): 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 is not continuous to the foundation. The following structural characteristics are usually indicative of a Q=3 value (also see Note 6): 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 135^o or more.	1.5, 3 or 5
Cantilever pier/column The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 6): 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. The following structural characteristics are usually indicative of a Q=2.5 value (also see Note 6): c. The ties are not anchored into the member cores with hooks of 135^o 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 $8d_b$ or less. f. Development length for longitudinal bars is less than $24d_b$.	1.5, 2.5 or 3.5

TABLE 1
(Continued)

B. EQUIPMENT BEHAVING AS STRUCTURES WITH INTEGRAL SUPPORTS	Q
1. Vertical vessels/heaters or spheres supported by:	
Steel skirts The following structural characteristics are usually indicative of a Q=2 value (also see Note 6): 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.	2 or 4
Steel braced legs without top girder or stiffener ring The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 6): a. The bracing system includes "K" braced bays. b. Brace connections are not able to develop the capacity of the diagonals. c. Column splice details cannot develop the column capacity. The following structural characteristics are usually indicative of a Q=3 value (also see Note 6): d. Diagonal elements designed to carry compression have (kl/r) greater than 120. e. The bracing system includes chevron ("V" or inverted "V") bracing that was designed to carry gravity load. f. Tension rod bracing with connections which develop rod strength.	1.5, 3 or 4
Steel unbraced legs without top girder or stiffener ring The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 6): a. Column splice details cannot develop the column capacity. b. Axial load demand represents more than 20% of the axial load capacity.	1.5 or 2.5
2. Chimneys or stacks	
Steel guyed	4
Steel cantilever	4
Concrete	4

TABLE 1
(Continued)

C. PIPEWAYS	**Q**
Note: This includes pipeways supporting equipment that does not weigh more than 25% of the other dead loads. For pipeways supporting equipment that weighs more than 25% of the other dead loads, see Section A, STRUCTURES SUPPORTING EQUIPMENT.	
1. Steel	
Ductile moment frame (see Note 8)	8
Ordinary moment frame (see Note 8)	6
Braced frame	6
Cantilever column	4
2. Concrete	
Ductile moment frame	8
Ordinary moment frame	5
Cantilever column	3.5

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. ANCHOR BOLTS (see Note 6)	**Q**
1. Anchor bolt yield controls	As for structure
2. Concrete failure or anchor bolt slippage controls, or there is a non-ductile force transfer mechanism between structure and foundation (see Note 7)	1.5
G. PIPING	**Q**
1. Piping in accordance with ASME B31, including in-line components with joints made by welding or brazing.	12
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.	6
3. Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing.	9
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.	4.5
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.

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. If bolt yielding controls the evaluation of the anchor bolts (as opposed to concrete failure or anchor bolt slippage), and there is a ductile force transfer mechanism between the structure and foundation (such as the use of properly proportioned anchor bolt chairs between skirts or tank shells and the foundation), then the Q-factor to be used for both the evaluation of the anchor bolts and the rest of the structural system corresponds to that for the structural system itself.

   If concrete failure or anchor bolt slippage controls the evaluation of anchor bolts (as opposed to bolt yielding), or there is a non-ductile force transfer mechanism between the structure and foundation, then a Q-factor of 1.5 should be used for the evaluation of the anchor bolts and the rest of the structural system. Also see Note 7.

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.

TABLE 1
(Continued)

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. This Committee suggests that for determining the connection forces using a Q-value equal to one half (1/2) of Q for the structure system, but not less than 2, where this type of connection is present, unless justified otherwise.

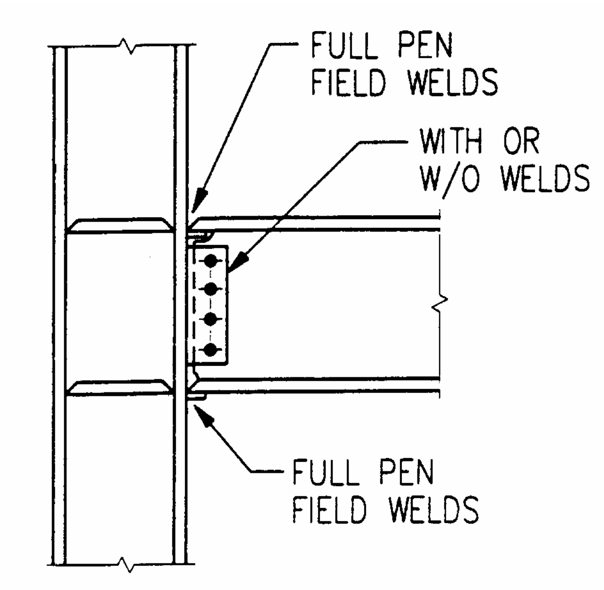

Figure 1: Former Standard Ductile Moment Connection Detail. (As a result of the Northridge Earthquake, this connection was shown to have major problems.)

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.

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 geotechnical report for the facility should be made available to the engineer performing the CalARP seismic review.

If the soil profile is known to be uniform over the entire area, a geotechnical report developed for an adjacent facility may be adequate. 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.

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

1) The owner may contract with a licensed geotechnical engineer to provide a report that will be adequate for the CalARP seismic review.

2) The engineer may engage a licensed geotechnical engineer as a sub-consultant to provide a geotechnical report.

3) 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 have a geotech report performed).

A standard geotechnical investigation report should include the information in the following list. The listed items are divided into two “tiers” or types of information. The first tier lists the basic minimum contents of a geotechnical (soils) report. The second tier lists information which the engineer performing the CalARP seismic review will eventually require, and it will be convenient and beneficial if the geotechnical report provides a professional presentation of this information.

Tier 1 -- Minimum Contents of Geotechnical Report for CalARP Review

1) Plot Plans drawn to scale depicting the locations of exploratory borings.

2) Boring logs (to depth of at least 50 feet) indicating ground surface elevation, blow counts (penetration), graphic log of material encountered, depth to groundwater (if encountered), soil classification and description (per ASTM standards), moisture content and dry density.

3) Geologic setting, subsurface soil conditions soil types, and regional groundwater information.

4) Recommendations for appropriate foundation schemes and design parameters including soil bearing capacity, estimated total/differential settlements, and lateral resistance.

5) Recommendations for the design of retaining walls including active and passive earth pressures.

Tier 2 -- Desirable Additional Contents of Geotechnical Report

1) Recommendations pertaining to seismic design parameters based on ASCE/SEI 7-05 or the latest California Building Code adopted by the local jurisdiction. Parameters such as Site Class; Site MCE 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.

2) Results of geologic and seismic hazard analysis (based on guidelines in SP117) including poor soil conditions, locations of active and potentially active faults, fault rupture potential, liquefaction, seismically-induced settlement/differential settlements, and seismically-induced flooding.

ATTACHMENT B

WALKDOWN FORMS FOR EQUIPMENT AND PIPING

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

Comments:

SCREENING EVALUATION: LOAD PATH

Noted Load Path Concerns:

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

Comments:

Seismic Evaluation
<u>CalARP Walkdown Review Sheet</u>
Piping

Line Number:		Date:	
Drawing Number:		By:	

Evaluation Summary (Circle one)

Adequate   Not Adequate   Further Evaluation Required

Inspection Attributes				
	Yes	No	Inac	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				

Page 1 of 2

Line Number:		Date:	

Notes and Sketches