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

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GUIDANCE FOR CalARP PROGRAM SEISMIC ASSESSMENTS

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 (reference # 1) having offsite consequences would not occur as a result of an earthquake. In the past, requests for performing Risk Management and Prevention Programs (RMPPs) have been through the Administering Agencies (AAs) of the program, usually as part of a broader hazard and operability study. Now, the seismic study is part of the mandated state's California Accidental Release (CalARP) program. The purpose of this document is to provide guidance regarding acceptable criteria to be used in such assessments. The guidance provided is applicable to structural systems and components whose failure would 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 (e.g., a 10% probability of being exceeded in 50 years). 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. The owner/operator should consult with the AA prior to beginning the seismic assessment.

1.1 Limitations - Conformance to this document does not guarantee or assure that an 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 RSs.

1.2 Evaluation Scope - The owner/operator, in consultation with the administering agency and seismic consultant, will identify the systems to be evaluated. 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 failure of systems which contain RSs.

3) Utility systems which would be required to operate following an earthquake for emergency reaction or to maintain the facility in a safe condition, (e.g., firewater or emergency power systems).

1.3 Performance Criteria - In order to achieve the overall objective of preventing releases of RSs, 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:

- Maintain structural integrity

- Maintain position

- Maintain containment of material
- Function during the earthquake
- Function after the 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.

It is anticipated that California state law will require that all new facilities permitted after April 1, 1999, be designed in accordance with the 1997 edition of the Uniform Building Code (UBC) (reference # 2). The 1997 UBC provides for increases in earthquake ground motions in the near field zone of major active earthquake faults. It also provides higher amplification factors for soft soil sites. These were two areas of concern regarding previous editions of the UBC which were identified in the RMPP Guidelines and which the Seismic Guidance Committee believes now have been adequately addressed. Therefore it is the consensus of this Committee that RS systems and components designed in accordance with the 1997 UBC / 2000 IBC (or later) provisions provide reasonable assurance of withstanding earthquake effects without either structural failure or a release of RSs having offsite consequences.

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

1.4 Seismic Evaluations Needed - Each owner/operator will have different conditions at their facility, and should work with the Administering Agency to determine which of the following subsections apply to their facility.

1.4.1 Use of 1992 Seismic Guidance Document for Existing Facilities - It is the consensus of the Committee that the contents and intent of this document are essentially the same as the guidance document originally published in 1992 (reference #3). It is therefore expected that evaluations meeting the intent of the 1992 document will also meet the intent of this document. It is recommended that owners/operators assessing the validity of past evaluations, including using the 1992 document, consider conditions that may make a partial or entirely new assessment necessary. Examples of such conditions may include:

- Major changes in the estimated ground motions.
- Significant system modifications, such as changing or addition of equipment or processes.
- The occurrence of an earthquake since the latest assessment.
- The occurrence of other events (e.g. explosion) that have caused structural damage.
- Significant deterioration (e.g. corrosion) in structural members or anchorages.

1.4.2 Facilities Permitted for Construction Prior to April 1, 1999 (1997 UBC Adoption Date) which are subject to CalARP requirements - These facilities include:

- Any facility for which a Risk Management and Prevention Program (RMPP) and accompanying seismic assessment have not been performed.
- Any facility which has RSs that were either not regulated under the RMPP, (eg.: flammable substance handling facilities), or plan to begin using RSs.

Such facilities for which a construction permit or permits has/have been issued prior to April 1, 1999 may generally be deemed to meet the intent of the requirements of Section 4 of this Guidance, provided the following conditions are met:

- Structures that have been previously determined to have met the June 1992 Seismic Guidance (see reference # 3) requirements and
- are not located in the near field zone (i.e., within 10 km of an active fault) and/or are not sited on soft soil and
- for which a visual field inspection reveals well-proportioned and complete lateral force resisting systems.

1.4.3 Facilities Permitted for Construction after April 1, 1999 which are subject to CalARP Program requirements - Design and construction of new facilities containing RSs must satisfy the seismic provisions of the 1997 UBC or the latest IBC adopted by the local jurisdiction. In addition, such facilities must comply with a visual field inspection after construction has been completed, in accordance with Section 3.0 of this document. Areas of weakness may not be apparent in design drawings or calculations.

2.0 DETERMINATION OF SEISMIC HAZARDS

A site specific seismic hazard assessment should address and, where appropriate, quantify the following earthquake effects:

1) Ground shaking, including local site amplification effects
2) Liquefaction
3) Fault rupture
4) Seismic settlement
5) Landslide
6) Tsunami/seiche

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. For establishing ground motions, any one of the following three procedures may be used.

1. Utilize the 1997 UBC spectral shapes determined in accordance with 1997 UBC requirements considering soil conditions and near source factors using the maps developed for the 1997 UBC.

2. Utilize the Maximum Considered Earthquake (MCE) Spectral Contour Maps developed for the 1997 NEHRP provisions (reference # 4) and the 2000 International Building Code (IBC). The ground motion response spectra used for the evaluation is taken as two-thirds of the MCE values after the MCE values have been adjusted for soil effects.

3. Utilize valid site specific criteria for ground shaking. Such site specific criteria should normally be expressed in terms of elastic response spectra at 5 percent of critical damping. Higher values of damping may be used for specific structures if justifiable by valid test results on a case-by-case basis. These site specific spectra should contain the following features:

   - The spectra should adequately consider current knowledge on source activity levels and attenuation effects through local geology and soils.

   - The spectra should adequately reflect local soil conditions and, as appropriate, account for near-fault effects.

   - The site specific elastic response spectra may be either probabilistic or deterministic spectra, as per the following guidelines. The spectra type (probabilistic or deterministic) producing the lower structural response may be used.

     Probabilistic Spectra - Probabilistic spectra are design response spectra in which each spectral ordinate has one of the following probabilities of being exceeded. Probabilities of exceedance may be either:

     a. a 10% probability in 50 years (a mean return period of 475 years); the full value of the spectral ordinates to be used.

     b. a 2% probability of being exceeded in 50 years (a mean return period of approximately 2475 years); two-thirds of the spectral ordinates to be used.

     Deterministic Spectra - When located within 10 kilometers of a recognized active fault (one capable of producing an earthquake of Magnitude 6.5 or greater), the site specific spectra may be estimated using deterministic methods. The spectral ordinates of a deterministic spectrum shall be based on the maximum magnitude earthquake which can reasonably be expected on the active fault considering appropriate source characteristics and assuming mean attenuation relationships.

It is recognized that less stringent ground motion criteria (with higher probability of exceedance) may be acceptable in certain situations. Examples may include:

- Temporary equipment
- Installations with a short remaining life
- Equipment or components that will contain RSs for very short durations.
- Installation locations where the consequences of a release are significantly lower than for the remainder of the facility.

3.0 WALKTHROUGH CONSIDERATIONS

A critical feature of the evaluation methodology is the physical inspection of the existing facility by a qualified 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 bases for assessment may include proven failure modes from past earthquake experience, basic engineering principles, and engineering judgement. 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 flat bottom tanks is discussed in Section 6. Specific guidance for piping systems is also discussed in Section 7.

The walkdown review will also include a limited review of drawings, as necessary. This may be done, for example, to check adequacy of older reinforced concrete structures, to verify anchorage details, or to identify configurations which can not 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. Note also that a drawing review alone without the physical inspection is not considered sufficient for an existing facility.

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

A detailed description of the walkdown process can be found in ASCE guidelines (see reference #5).

4.0 ANALYTICAL EVALUATION METHODOLOGY

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

4.2 Acceptance for existing structures, systems, and their foundations may be accomplished by one of the following procedures:

4.2.1 Perform an appropriate 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
$D + L + \frac{E}{Q} \leq$	{	1.6 x Working Stress Allowable (without 1/3 increase) or øU or $øR_n$ (using Load Factors of Unity for all loads)

where D = Dead load

L = Live and/or operating load

E = 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 the attached Table 1

R_n = Nominal capacity per AISC Load & Resistance Factor Design (LRFD)

U = Nominal capacity per ACI

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.

   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 structures whose fundamental period are 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 bents, in which seismic loads in individual members may be redistributed without failure, the demand (from the previous equation) on individual members may exceed capacity by up to 50 percent.

3) Relative displacements shall be considered and shall include torsional and translational deformations. Structural displacements that are determined from an elastic analysis that was based on seismic loading reduced by Q shall be multiplied by the factor 0.5Q, [where the value of the factor 0.5Q

shall not be taken as less than one (1.0)], to determine displacements to be used in an evaluation.

Generally, the drift (relative horizontal displacement) shall 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 nonstructural elements or the equipment itself.

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 which may be subject to higher vertical acceleration.

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 shall be determined in accordance with the latest International Conference of Building Officials (ICBO) standards. Where the anchorage capacity is greater than 1.25 times the minimum yield strength (but may not be 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, the Q value for determining bolt loads shall 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 re-assessed using a more rigorous approach to determine if structural retrofit is actually required.

8) Note that the importance factor (I), as defined in the UBC base shear equation for design of new facilities, is always set to unity (1.0) for evaluation of existing facilities, unless requested otherwise for "Special" structures by the owner of the facility.

4.2.2 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 non-linear time history analyses would be acceptable. The resulting inelastic deformations must be within appropriate levels to provide reasonable assurance of structural integrity.

4.2.3 ASCE reference #5, 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 ASSESSMENT OF EQUIPMENT AND NON-STRUCTURAL ELEMENTS

Equipment and non-structural elements are considered subsystems if their total weight is less than 25% of the total weight of the supporting structure. Any component or structure founded directly on soil/ground is not considered a subsystem. For subsystems, the anchorage of equipment and non-structural elements supported within or by structures may be evaluated in accordance with Section 1632.2 of the 1997 UBC. If the equipment weight is greater than 25% of the weight of the supporting structure, use Q values equal to the smaller of the values for the equipment or the supporting structure from Table 1. 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. Exception: Equipment and systems that have been previously judged to have met the June 1992 Seismic Guidance requirements and for which a visual field inspection reveals well-proportioned and complete lateral force resisting system, may be deemed to meet the intent of these requirements without further evaluation.

6.0 EVALUATION OF TANKS AT GRADE

Flat-bottom vertical liquid storage tanks have often failed, sometimes with loss of contents during strong ground shaking. The response of such tanks, unanchored tanks in particular, is highly non-linear, 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 that are overconstrained 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:

a. 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 tank shell, normally associated with taller tanks, is "diamond shape" buckling.

b. Weld failure between the bottom plate and the tank shell as a result of high tension forces during uplift.

c. Fluid sloshing, thus potentially causing damage to the tank's roof followed by spillage of fluid.

d. Buckling of support columns for fixed roof tanks.

e. 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.

f. Tearing of tank shell or bottom plate due to overconstrained stairway, ladder, or piping anchored at a foundation and at the tank shell.

g. Tearing of tank shell due to overconstrained walkways connecting two tanks experiencing differential movement.

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

i. Splitting and leakage of tank shells due to high tensile hoop stress.

When evaluating existing tanks at grade for seismic vulnerabilities, the following steps should be followed:

a. Quantification of site specific seismic hazard as outlined in Section 2.

b. Walkthrough inspection to assess piping, staircase and walkway attachments, and other potential hazards.

c. Analytical assessment of tanks to evaluate the potential for overturning and shell buckling.

Engineering judgement 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. If an analytical evaluation is deemed necessary, various industry standards and other methodologies are available in the literature for evaluation of tanks at grade which can be used. These include:

a. API 650 Appendix E 9th Edition, Addendum 4, December 1997 (reference #6, 1997) - This method is standard for design of new tanks for the petrochemical industry. Its provisions are accepted by the UBC and it addresses both anchored and unanchored tanks.

b. AWWA D100 (reference #7, 1985) - 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.

c. Velestos and Yang (reference #8, 1984) - This method is primarily for anchored tanks.

d. Manos (reference #9, 1986) - 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.

e. Housner and Haroun (reference #10, 1980) - This method is primarily for anchored tanks.

Alternatively, the Q factor given in Table 1 for tanks in conjunction with Equation for Demand as outlined 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 a & b listed above, Q factor reductions are inherently included in the determination of seismic forces. In references c, d & e listed above, the Q factors should only be applied to structural modes.

If the walkthrough and the evaluation of the tank identifies potential seismic vulnerabilities, mitigation measures are required. These mitigations include reduction of the seismic risk through measures such as addition of flexibility to rigid attachments, reduction of safe operating height or, as a last resort, anchorage of the tank.

Reference #5, Section 7.0 provides a thorough overview of tank failure modes during a seismic event, seismic vulnerabilities to look for during a seismic walkthrough, detailed methodology for analytical evaluation as well as suggested modifications to mitigate seismic hazards.

7.0 EVALUATION OF ABOVE GROUND PIPING SYSTEMS

7.1 Evaluation of piping systems are primarily accomplished by field walkthroughs. Such qualitative evaluations of piping systems are best done by an engineer experienced in this area, visually inspecting the piping system under concern. This is preferred because some piping is field routed and, in some instances, piping and supports have been modified from that shown on design drawings.

This guidance is primarily intended for ductile steel pipe constructed to a national standard such as ASME B31.3 (reference #11). Evaluation of other piping material is discussed in Section 7.6.

7.2 The procedure for evaluating above ground piping systems should be as follows:

1) Identify piping systems to be evaluated.

2) Determine original design code basis and materials of construction, to the extent possible.

3) Assess extent of obvious corrosion/erosion.

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

5) Complete the detailed evaluation of any identified areas and recommend remedial actions.

7.3 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.

7.4 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.

2) Interaction with other elements.

3) Extensive corrosion effects.

4) Non-ductile materials such as cast iron, fiberglass (PVC), glass, etc. combined with high stress or impact conditions.

Seismic anchor movements could result in 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. Interaction is defined as the seismically induced impact of piping systems with adjacent structures, systems, or components, including the effects of the falling hazards. Corrosion could result in a weakened pipe cross section that could fail during an earthquake.

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

1) Large unsupported segment of pipe,

2) Brittle elements,

3) Threaded connections, flange joints, and special fittings, and

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) External corrosion,

9) Inadequate vertical supports and/or insufficient lateral restraints,

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), and

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

7.5 The walkthrough 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 walkthrough should be performed by a licensed engineer familiar with how equipment responds to earthquake loads. Some guidance on how to perform a walkthrough can be found in reference #5. 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 walkthrough is performed and if an analysis is deemed necessary, 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 pipelines with short rod hangers can be considered as effective lateral supports if justified,

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

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 7.5.1 may need to be reduced.

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

7.5.1 Procedures for seismic anchor movement evaluation of piping are as follows:

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

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

$$\frac{iM_{sam}}{Z} \leq 3.05\ S_h$$

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
- 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

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

7.5.3 Procedures for corrosion evaluation of piping are as follows:

1) During walkthrough, identify conditions conducive to external corrosion.

2) Wall thickness should be evaluated for potential reduction due to erosion or corrosion.

3) Extent of internal corrosion/erosion can be evaluated by any of the following methods:

   a. Review of existing corrosion inspection program for RS piping systems,

b. Review of successful operating experience, or

c. Wall thickness measurements.

4) Compare existing corrosion experience and anticipated corrosion to original design corrosion allowance.

7.5.4 Procedures for seismic inertia evaluation of piping are as follows:

1) Large unsupported spans of piping should be checked for inertia loads. The inertial moments should not exceed those allowed in Section 7.5.1. It is not necessary to combine moments from seismic anchor movement and inertial loading. Inertial loading should be based on the appropriate pipe support motions and the characteristics of the piping configuration. Seismic inertial loading usually can be evaluated by simplified calculations.

2) If the intent of ASME B31.3 or other appropriate reference can be demonstrated to have been met for seismic effects, then the piping system is deemed acceptable.

7.6 Procedures for determination of allowable stress levels for piping materials not covered by ASME B31.3 are as follows:

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:

Ductile Materials : $S_h = \frac{1}{3} \sigma_t$ at temperature

Brittle Materials : $S_h = \frac{1}{40} \sigma_t$ 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.

4) If a strength cannot be determined with a reasonable level of confidence, then the piping material should be replaced with one having known properties.

8.0 STRENGTHENING CRITERIA

A strengthening and/or management program should be developed to correct deficiencies. If strengthening is required, appropriate strengthening criteria must 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.

When any retrofit construction work 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, this may not be practical. The retrofit design criteria should be consistent with this proposed guidance. It is always advisable to meet code requirements to the extent practical. Therefore, if the intent is to do enough work to satisfy the CalARP Program requirements, but not meet the current code requirements, it behooves the owner, or his/her engineer to discuss the proposed work with the local Building Official to ensure the Building Official is in agreement. If the Building Official does not already have a copy of the CalARP Program guidelines, then he/she should be given one. The Risk Management Plan and General notes on the detail drawings should clearly state whether or not the new construction meets the current Building Code.

9.0 REFERENCES

References may be obtained from:

Engineering Societies Library (Linda Hale 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.

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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

** The notes following the table should be read in conjunction with the tabulated Q-factors.

TABLE 1

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

A. STRUCTURES SUPPORTING EQUIPMENT (Continued)	**Q**
2. Concrete structures	
Ductile moment frame	6 or 8
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.	
Intermediate moment frame	
Ordinary moment frame	4
The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 6):	1.5, 2.5 or 3.5
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.	
Shear wall	1.5, 3 or 5
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.	
Cantilever pier/column	1.5, 2.5 or 3.5
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 $24d_b$.	

TABLE 1

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

B. EQUIPMENT BEHAVING AS STRUCTURES WITH INTEGRAL SUPPORTS	**Q**
1. Vertical vessels/heaters or spheres supported by:	
Steel skirts	2 or 4
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.	
Steel braced legs without top girder or stiffener ring	1.5, 3 or 4
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.	
Steel unbraced legs without top girder or stiffener ring	1.5 or 2.5
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.	
2. Chimneys or stacks	
Steel guyed	4
Steel cantilever	4
Concrete	4

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

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

D. FLAT-BOTTOMED TANKS (see Note 4)	**Q**
1. Anchored (See Note 9) **2. Unanchored**	4 3
E. FOUNDATIONS (See Note 5)	**Q**
1. Piled **2. Spread footings**	6 6
F. ANCHOR BOLTS (see Note 6)	**Q**
1. Anchor bolt yield controls **2. Concrete failure or anchor bolt slippage controls, or there is a non-ductile force transfer mechanism between structure and foundation** (see Note 7)	As for structure 1.5

TABLE 1

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

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, noncompact steel members, high (Kl/r) members and nonductile 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. The cause of these cracks is still under investigation. A number of causes have been postulated, such as poor welding, weld rod and welding procedures, inadequate inspection, inadequately addressed thermal considerations,

TABLE 1

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

and the potential need to move the plastic hinge formation in the beam, away from the face of the column. At the time of writing of this document, the cause(s) and solution(s) are still under investigation. This Committee tentatively 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. Further discussion of this, including some suggested details and retrofit details, is provided in reference #12.

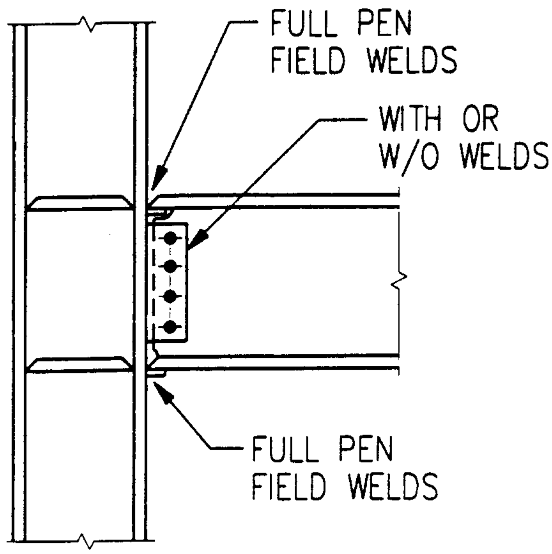

Figure 1: Former standard ductile moment connection detail. As a result of the Northridge Earthquake, this connection was shown to have major problems and is currently prohibited by the Uniform Building Code in seismic zones 3 & 4.

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