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). Compare the current CalARP seismic loading to the prior evaluations seismic loading and comment.

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

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

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

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

7) 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.

8) 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. Include prior recommendations that were not addressed or which were not addressed adequately since the last evaluation.

9) The revalidation CalARP report should be signed and stamped by the Responsible Engineer (see Section 1.5).
10) The revalidation CalARP report should discuss all deficiencies and recommendations identified during this evaluation regardless of whether or not they were contained in previous evaluation findings. Provide a photograph showing the identified deficiency if possible.
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.

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, *California Accidental Release Prevention (CalARP) Program Detailed Analysis*.
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, September 2010, Approved by Region I LEPC, 1/14/2010.
3. *Uniform Building Code*, 1997 Edition, International Conference of Building Officials, Whittier, California, 1997.
4. ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Reston, Virginia, 2010.
5. California Department of Conservation, *California Geological Survey, Guidelines for Evaluating and Mitigating Seismic Hazards in California*, Special Publication 117A, 2008.
6. *Guidelines for Seismic Evaluation and Design of Petrochemical Facilities*, Second Edition, Task Committee on Seismic Evaluation and Design of Petrochemical Facilities, American Society of Civil Engineers, Reston, Virginia, 2011.
7. Gurbuz, O., A. Lopez, and P. Summers, 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 G. Hau "Seismic Evaluation Acceptance Guideline for Existing Above Ground Piping Systems", ASME PVP Volume 256-1, 1993.
9. API Standard 650, *Welded Tanks for Oil Storage*, 12th Edition, American Petroleum Institute, Washington, D.C., 2013.
10. AWWA D100-11, *Welded Carbon Steel Tanks for Water Storage*, American Water Works Association, Denver, Colorado, 2011.
11. Veletsos, A.S., Contributor, "Guidelines for the Seismic Design of Oil and Gas Pipeline Systems", ASCE, Committee on Gas and Liquid Fuel Lifelines, NY, NY, 1984.
12. Manos, G.W., August 1986, "Earthquake Tank-Wall Stability of Unbraced Tanks," American Society of Civil Engineers, Journal of Structural Engineering, Vol. 112, No. 8, including Erratum in Journal of Structural Engineering, Vol. 113, No.3, March 1987.
13. Housner, G.W., and M.A. Haroun, 1980, "Seismic Design of Liquid Storage Tanks," ASCE Convention and Exposition, Portland, Oregon, April 14-18, 1980.
14. ACI 350.3-06, *Seismic Design of Liquid - Containing Concrete Structures and Commentary*, American Concrete Institute, Farmington Hills, Michigan, 2006.
15. ASME B31.3 - 2012, *Process Piping, ASME Code for Pressure Piping, B31*, American Society of Mechanical Engineers, New York, New York, 2013.
16. ASCE, Los Angeles Section Geotechnical Group, "Recommended Procedures for Implementation of Division of Mines and Geology Special Publication 117, Guidelines

for Analyzing and Mitigating Landslide Hazards in California", Published by Southern California Earthquake Center (SCEC), June 2002.
17. ASCE, Los Angeles Section Geotechnical Group, "Recommended Procedures for Implementation of Division of Mines and Geology Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California", Published by Southern California Earthquake Center (SCEC), March 1999.
18. FEMA 350 *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, Federal Emergency Management Agency, Washington, D.C., June 2000.
19. 2012 *International Building Code*, International Code Council, Inc., Country Club Hills, Illinois, June 2011.
20. FEMA P646 *Guidelines for Design of Structures for Vertical Evacuation from Tsunamis*, Federal Emergency Management Agency, Washington, D.C., June 2008.
21. ASME B31Ea – 2010, *Addenda to ASME B31E – 2008 Standard for the Seismic Design and Retrofit of Above – Ground Piping Systems*, American Society of Mechanical Engineers, New York, New York, 2010.
22. *2013 California Building Code Title 24, Part 2, Volumes 1 and 2*, California Building Standards Commission, Sacramento, California, 2013.
23. National Science and Technology Council, *Tsunami Risk Reduction for the United States: A Framework for Action*, December 2005.
24. Bernard, B, L Dengler, and S Yim (editors), 2007, *National Tsunami Research Plan: Report of a Workshop Sponsored by NSF/NOAA*.
25. ASCE/SEI 41-13, *Seismic Rehabilitation of Existing Buildings*, American Society of Civil Engineers, Reston, Virginia, 2013.
26. API Standard 620, *Design and Construction of Large, Welded, Low-pressure Storage Tanks*, Eleventh Edition, American Petroleum Institute, Washington, D.C., 2008.
27. US Department of Energy, *Nuclear Reactors and Earthquakes*, Report No. TID-7024, January 1961.
28. *Guidelines for the Design of Buried Steel Pipe*, American Lifelines Alliance, July 2001.
29. M.J.N Priestley (Editor & Chairman) et. al, "Seismic Design of Storage Tanks," Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering, December 1986.
30. Eurocode 8, (1998), "Design provisions for earthquake resistance of structures, Part 1-General rules and Part 4 - Silos, tanks and pipelines," European committee for Standardization, Brussels.
31. *CSU Seismic Requirements*, The California State University, Office of the Chancellor, December 21, 2011.

Table 1. Ductility-Based Reduction Factors (Q) for Existing Structures and Systems

A. STRUCTURES SUPPORTING EQUIPMENT	**Q**
This covers structures whose primary purpose is to support equipment, such as air coolers, spheres, horizontal vessels, exchangers, heaters, vertical vessels and reactors, etc.	
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 7): 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 7): 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

(Continued)

Table 1. Ductility-Based Reduction Factors (Q) for Existing Structures and Systems (Continued)

Braced frame	2, 4 or 5
The following structural characteristics are usually indicative of a Q=2 value (also see Note 7):	
a. There is a significant strength discontinuity in any of the vertical lateral force resisting elements, i.e., a weak story (see ASCE7-10 Table 12.3-2).	
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 7):	
e. Diagonal elements designed to carry compression have (kl/r) greater than 120.	
f. The bracing system includes chevron ("V" or inverted "V") bracing that was designed to carry gravity load and/or beams not designed to resist unbalanced load effects due to compression buckling and brace yielding.	
g. Tension rod bracing with connections which develop rod strength.	
Cantilever column	2 or 3.5
The following structural characteristics are usually indicative of a Q=2.0 value (also see Note 7):	
a. Column splice details cannot develop the column capacity.	
b. Axial load demand represents more than 20% of the axial load capacity.	

(Continued)

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 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 1.5, 2.5 or 3.5
Ordinary moment frame The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 7): 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 7): 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° 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 35 d_b long. Any column bar lap splice is not enclosed by ties spaced 8 d_b or less. i. Development length for longitudinal bars is less than 24 d_b. j. Shear failure occurs before flexural failure in a significant number of the beams.	

(Continued)

Table 1. Ductility-Based Reduction Factors (Q) for Existing Structures and Systems (Continued)

Shear wall The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 7): 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 7): 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 8 d_b. f. For coupled shear wall buildings, stirrups in any coupling beam are spaced at greater than 8 d_b or are not anchored into the core with hooks of 135° or more.	1.5, 3 or 5

(Continued)

Table 1. Ductility-Based Reduction Factors (Q) for Existing Structures and Systems (Continued)

Cantilever pier/column The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 7): 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 7): c. The ties are not anchored into the member cores with hooks of 135° 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 35 d_b long. Any pier/column bar lap splice is not enclosed by ties spaced 8 d_b or less. f. Development length for longitudinal bars is less than 24 d_b.	1.5, 2.5 or 3.5

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 7): 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

(Continued)

Table 1. Ductility-Based Reduction Factors (Q) for Existing Structures and Systems (Continued)

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 7): 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 7): 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 and/or beams not designed to resist unbalanced load effects due to compression buckling and brace yielding. 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 7): 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

(Continued)