

Figure 6-6: Results from linear analysis of bottom bay columns of precipitator / stack support structures, using $Q=2$.



Figure 6-7: Bellow connection between spray tower and precipitator.

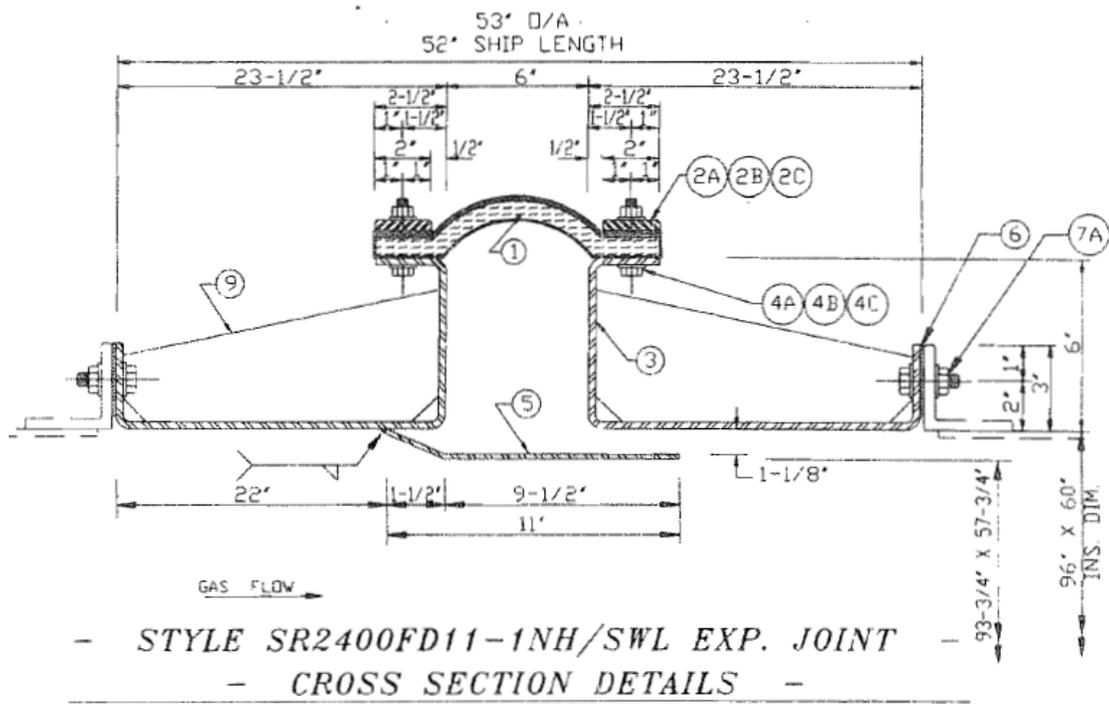


Figure 6-8: Details of Senior Flexonics connection between spray tower (to left of expansion joint) and precipitator (to right of expansion joint).

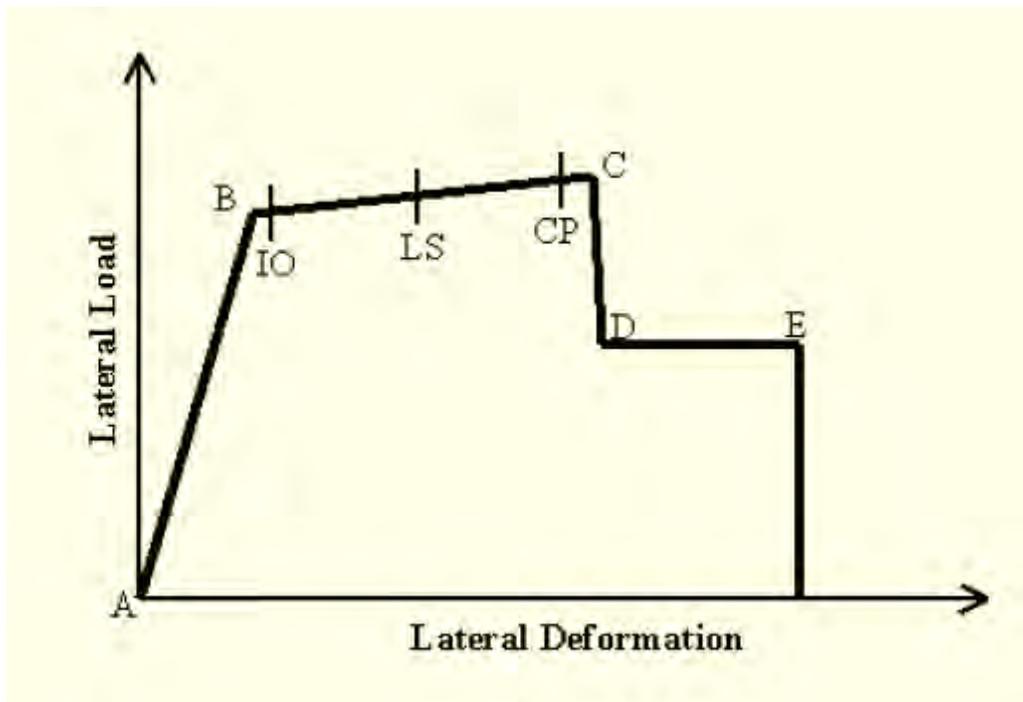


Figure 6-9: Generic force-deformation curve for nonlinear analysis.

NONLINEAR STATIC PUSHOVER ANALYSIS
(Push from North direction & Load Combination 1.2D+E)

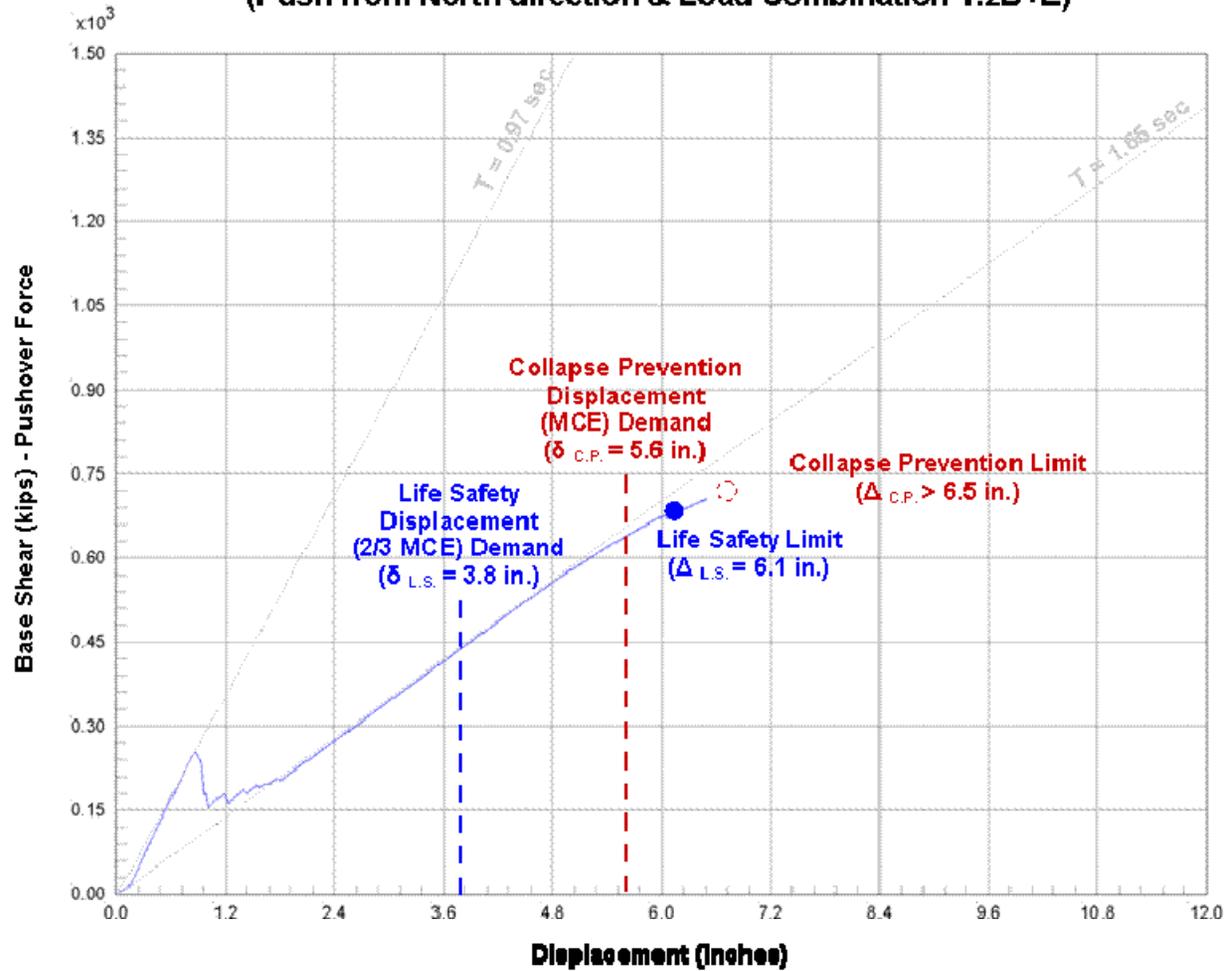


Figure 6-10: Nonlinear pushover analysis results for push from North direction. Vertical load = 1.2D.

NONLINEAR STATIC PUSHOVER ANALYSIS
(Push from North direction & Load Combination 0.9D+E)

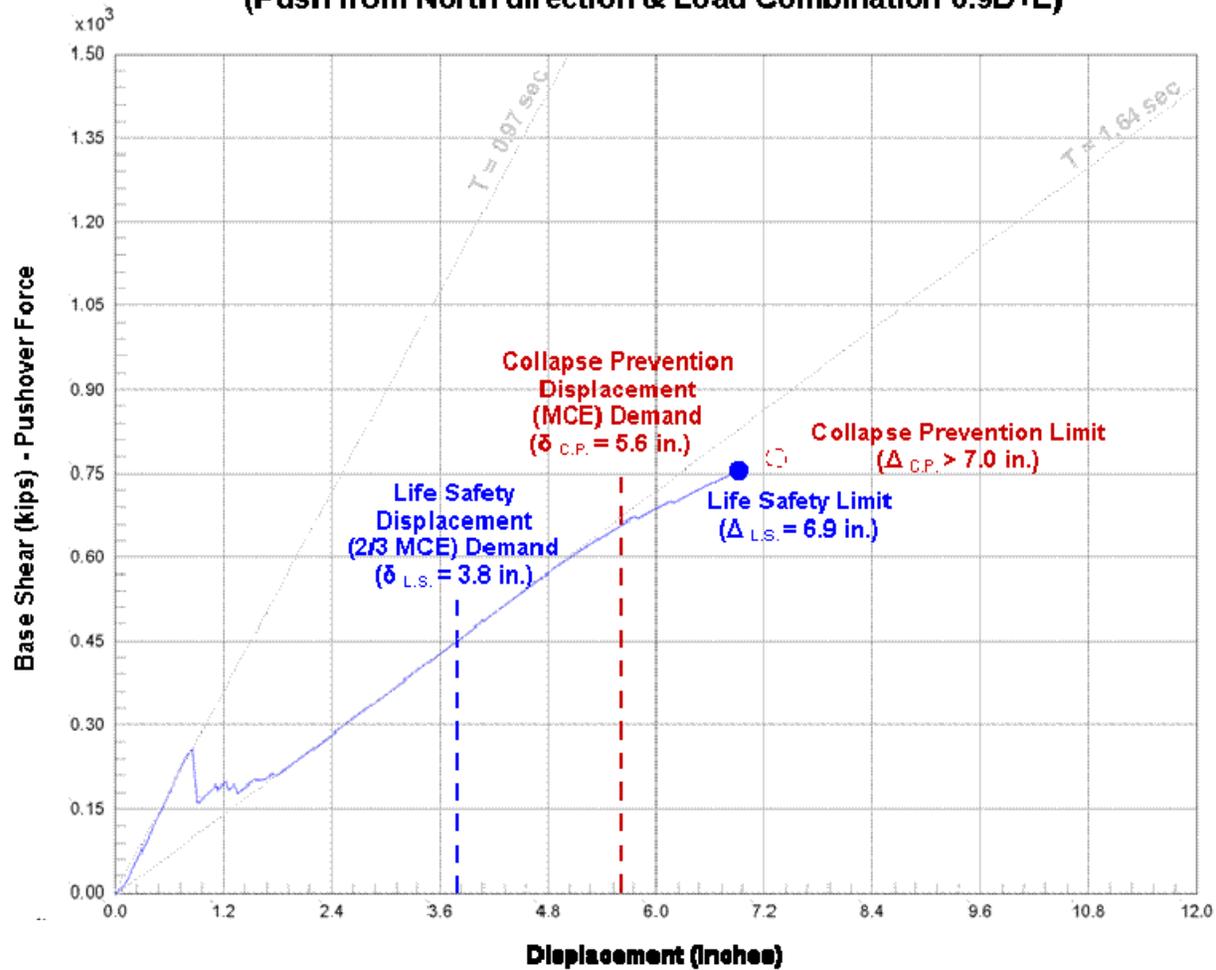


Figure 6-11: Nonlinear pushover analysis results for push from North direction. Vertical load = 0.9D.

NONLINEAR STATIC PUSHOVER ANALYSIS
(Push from South direction & Load Combination 1.2D+E)

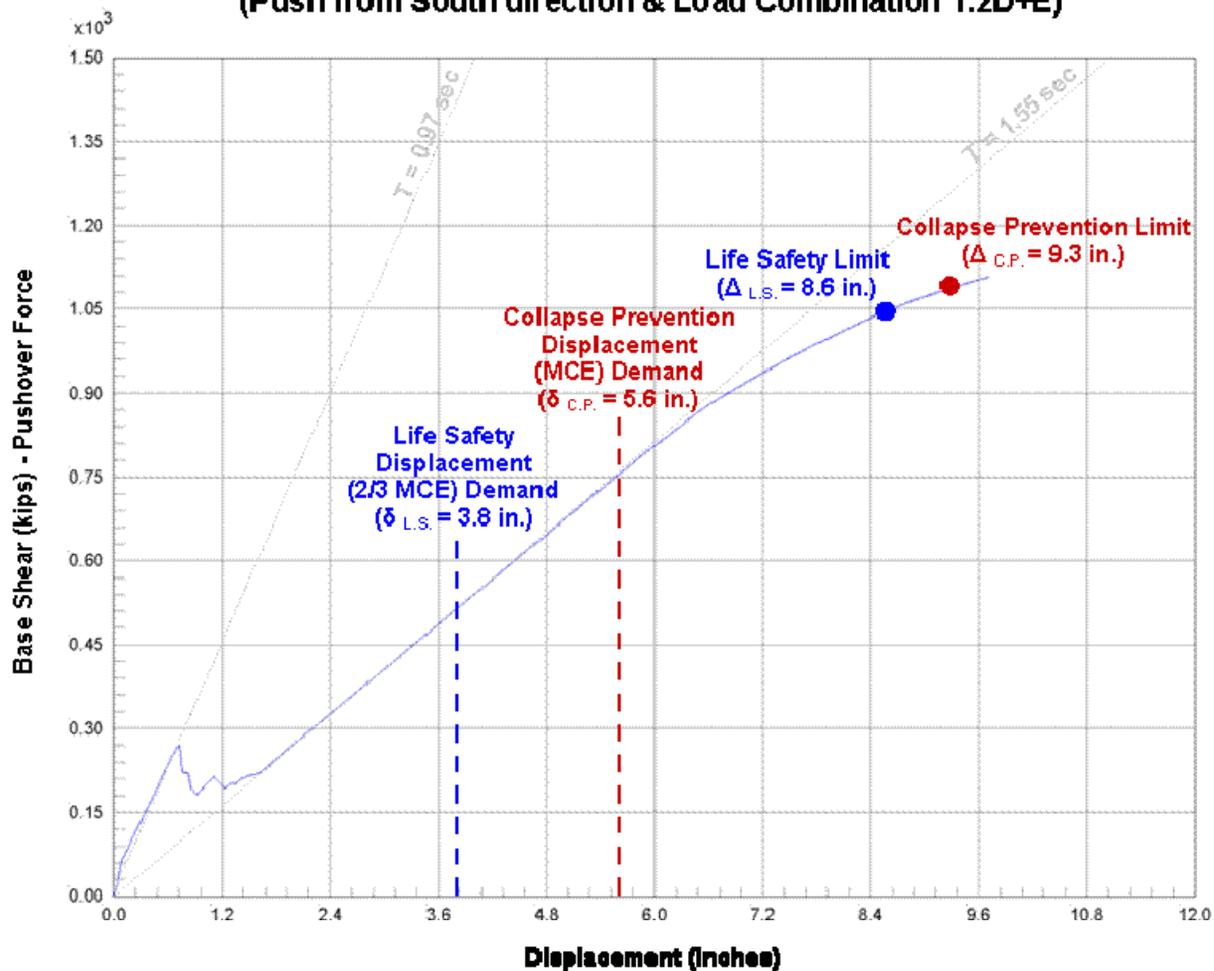


Figure 6-12: Nonlinear pushover analysis results for push from South direction. Vertical load = 1.2D.

NONLINEAR STATIC PUSHOVER ANALYSIS
(Push from South direction & Load Combination 0.9D+E)

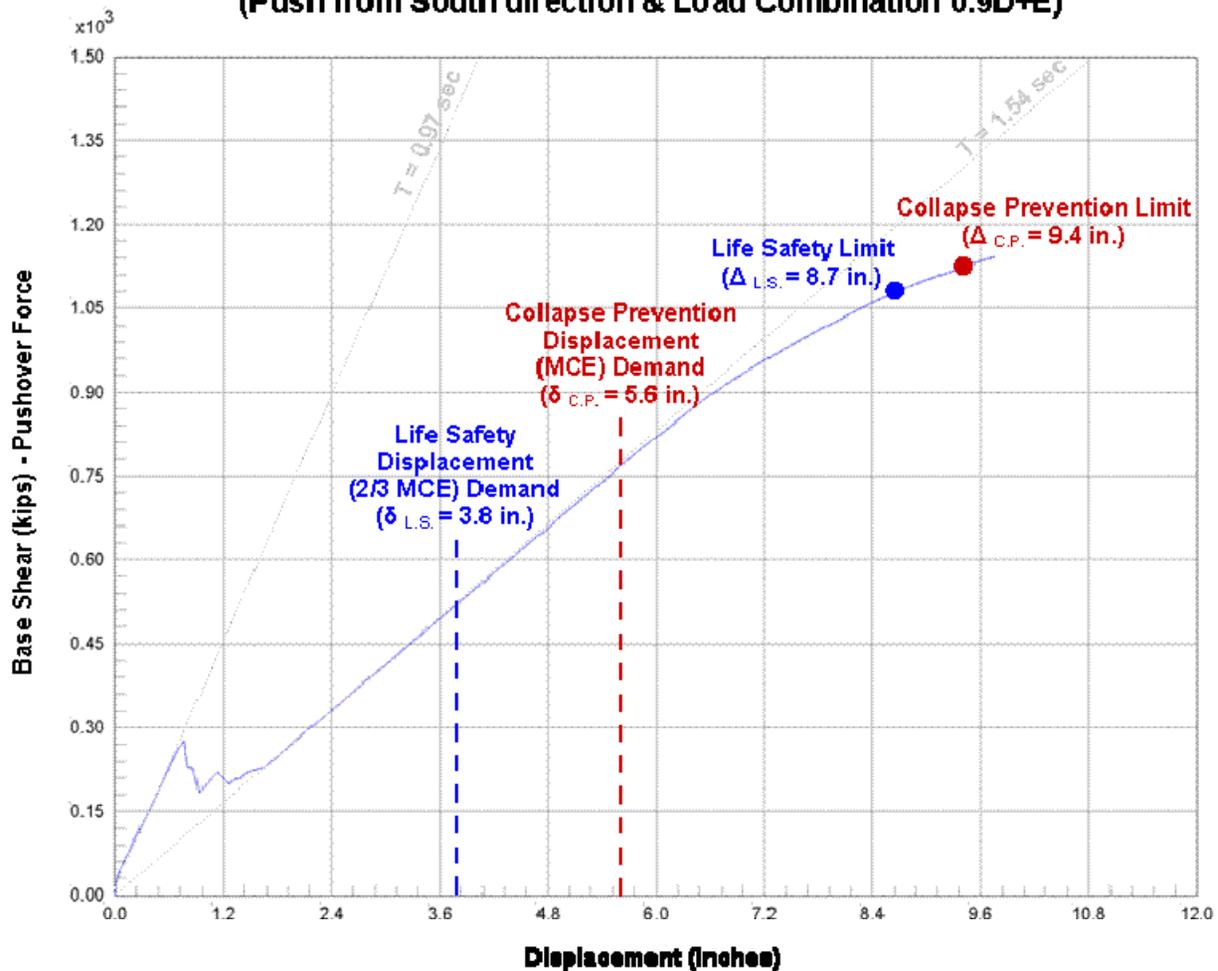


Figure 6-13: Nonlinear pushover analysis results for push from South direction. Vertical load = 0.9D.

**NONLINEAR STATIC PUSHOVER ANALYSIS
(Push from East direction & Load Combination 1.2D+E)**

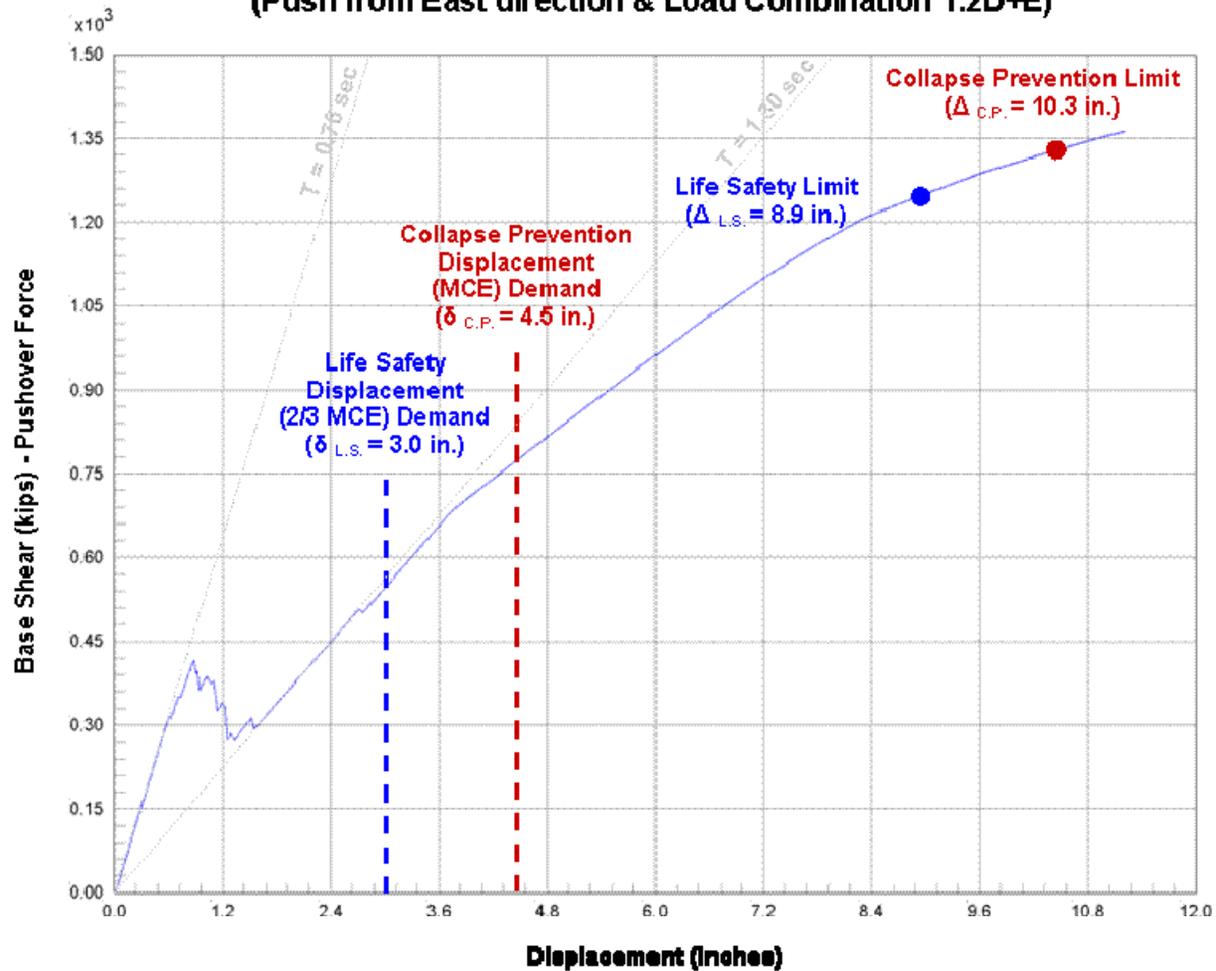


Figure 6-14: Nonlinear pushover analysis results for push from East direction. Vertical load = 1.2D.

NONLINEAR STATIC PUSHOVER ANALYSIS
(Push from East direction & Load Combination 0.9D+E)

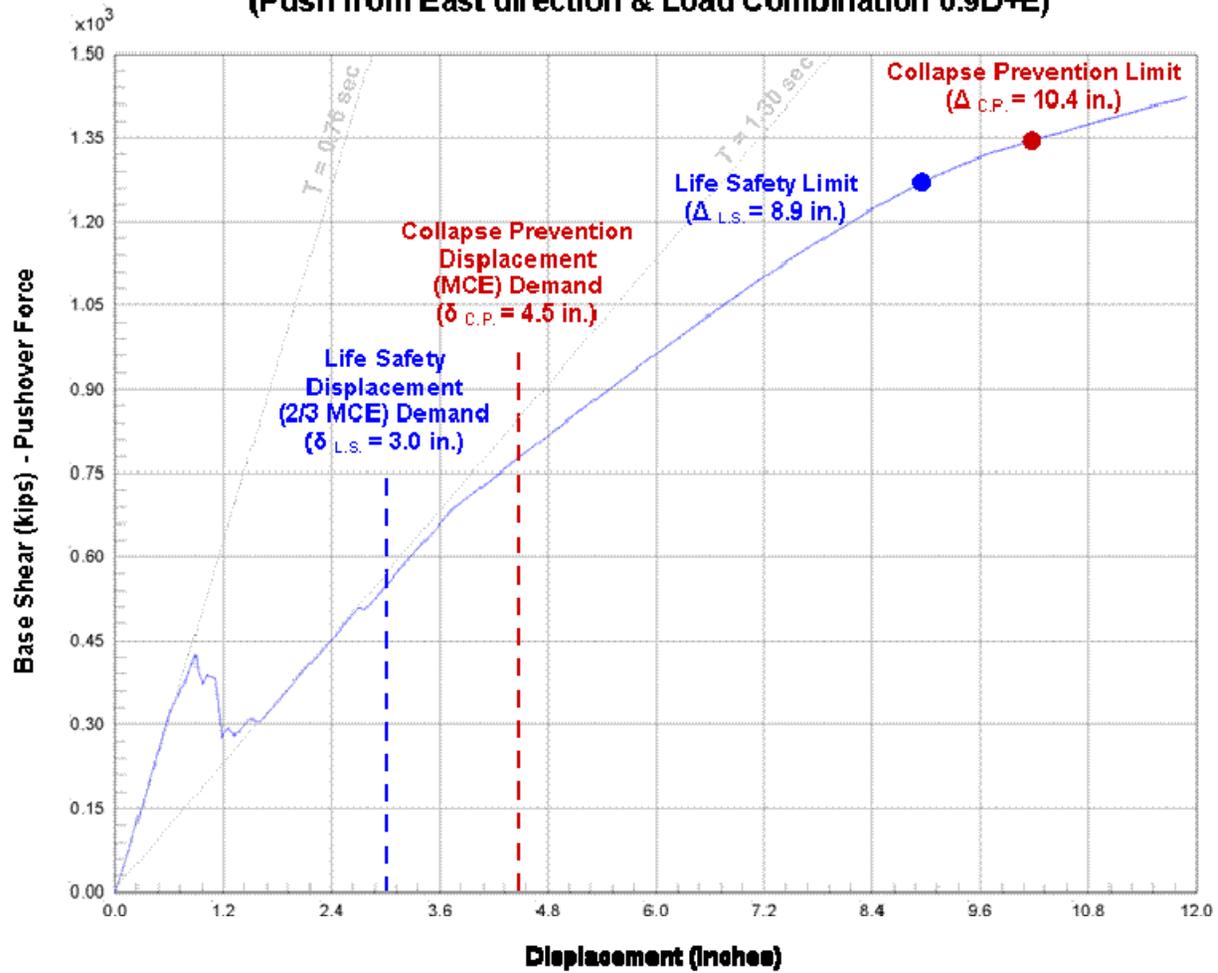


Figure 6-15: Nonlinear pushover analysis results for push from East direction. Vertical load = 0.9D.

NONLINEAR STATIC PUSHOVER ANALYSIS
(Push from West direction & Load Combination 1.2D+E)

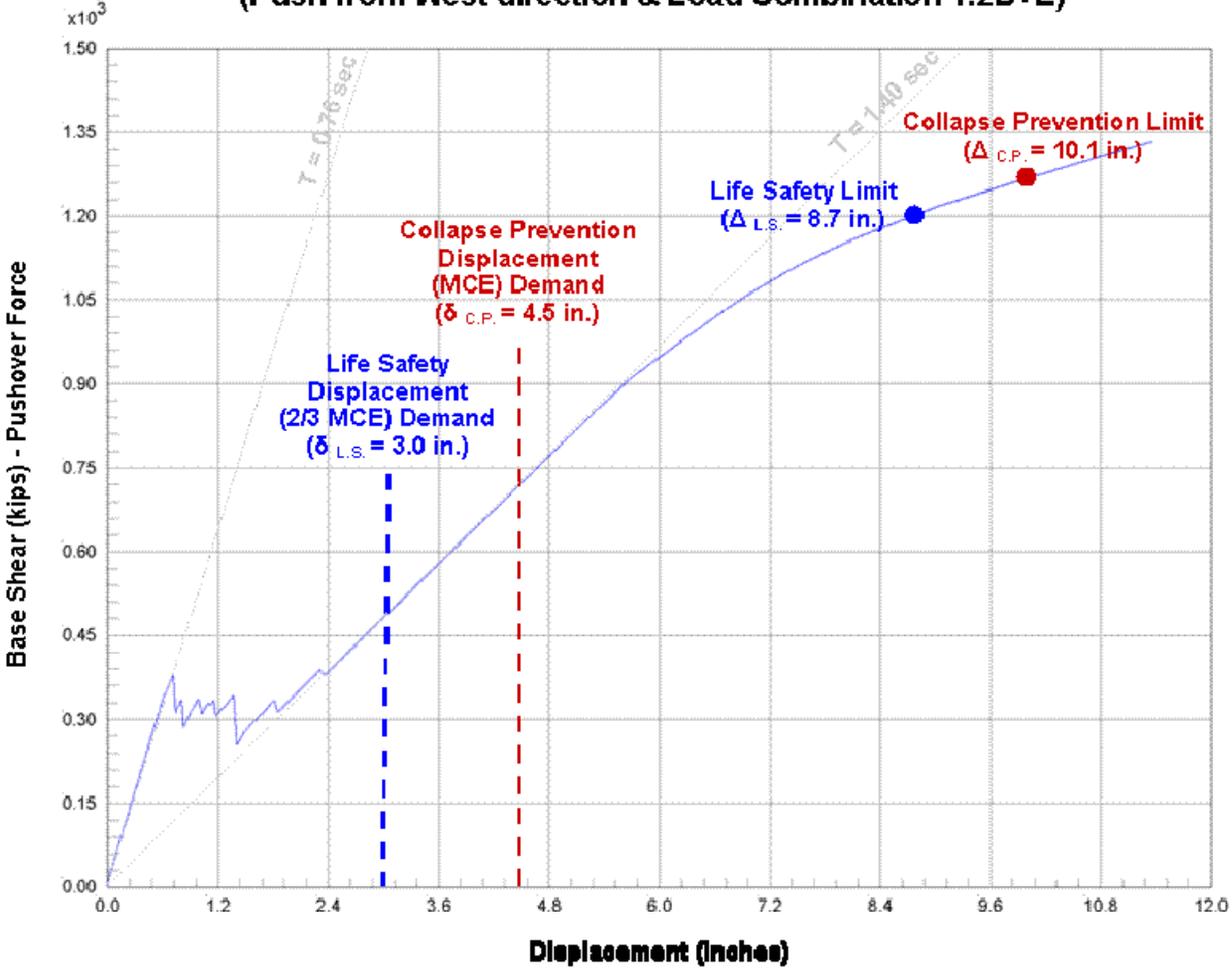


Figure 6-16: Nonlinear pushover analysis results for push from West direction. Vertical load = 1.2D.

NONLINEAR STATIC PUSHOVER ANALYSIS
(Push from West direction & Load Combination 0.9D+E)

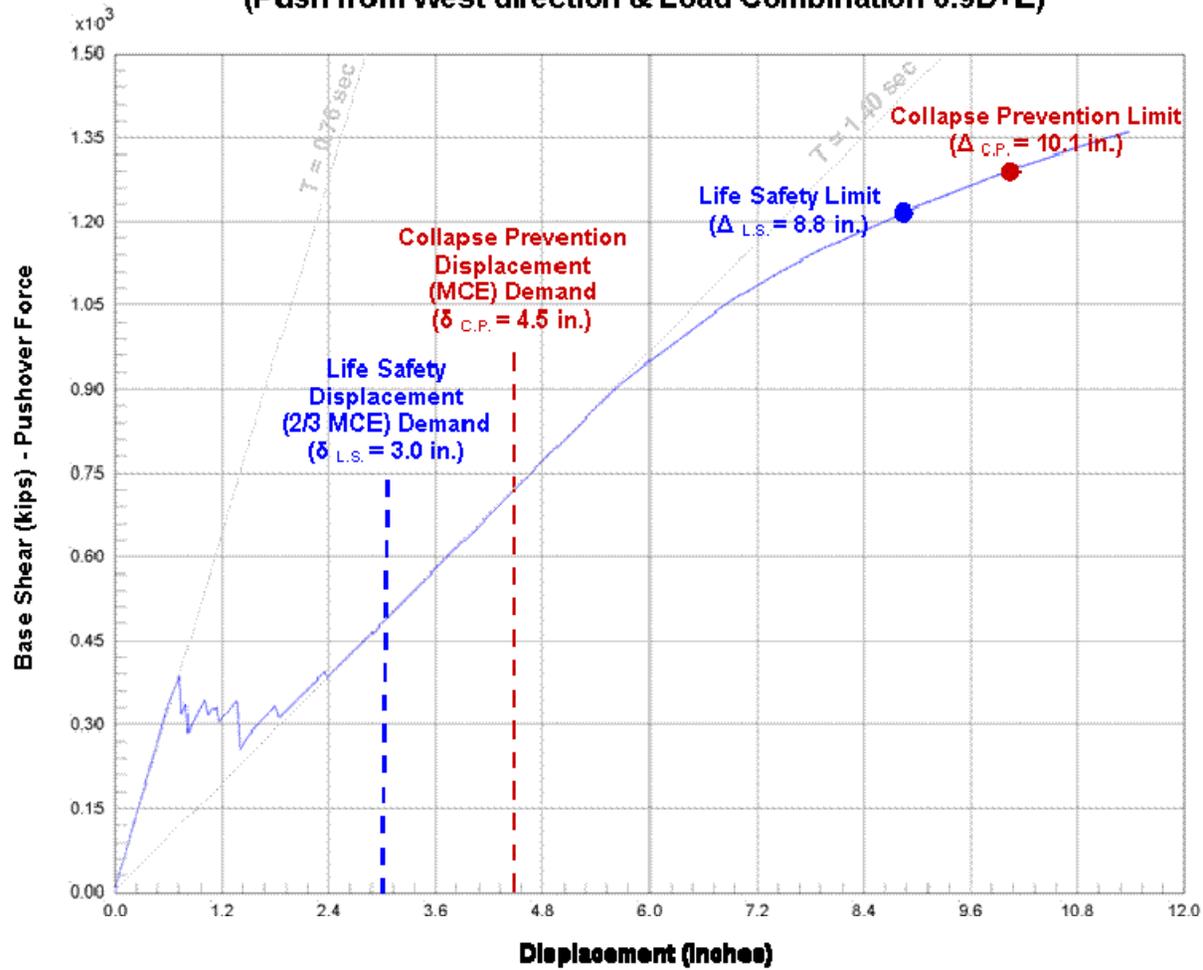


Figure 6-17: Nonlinear pushover analysis results for push from West direction. Vertical load = 0.9D.

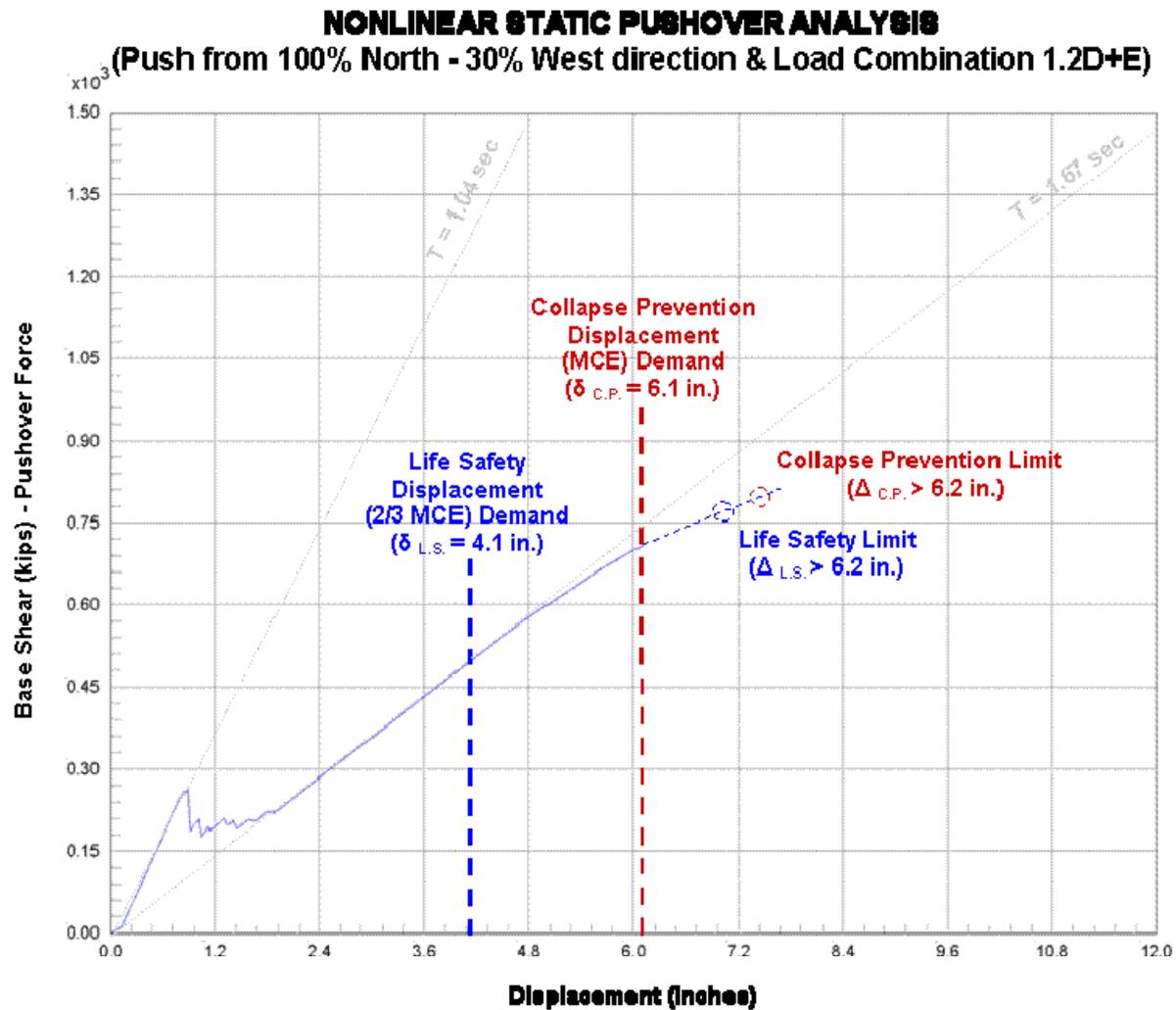


Figure 6-18: Nonlinear pushover analysis results for multidirectional worst-case loads, 100% from North and 30% from West. Vertical load = 1.2D.

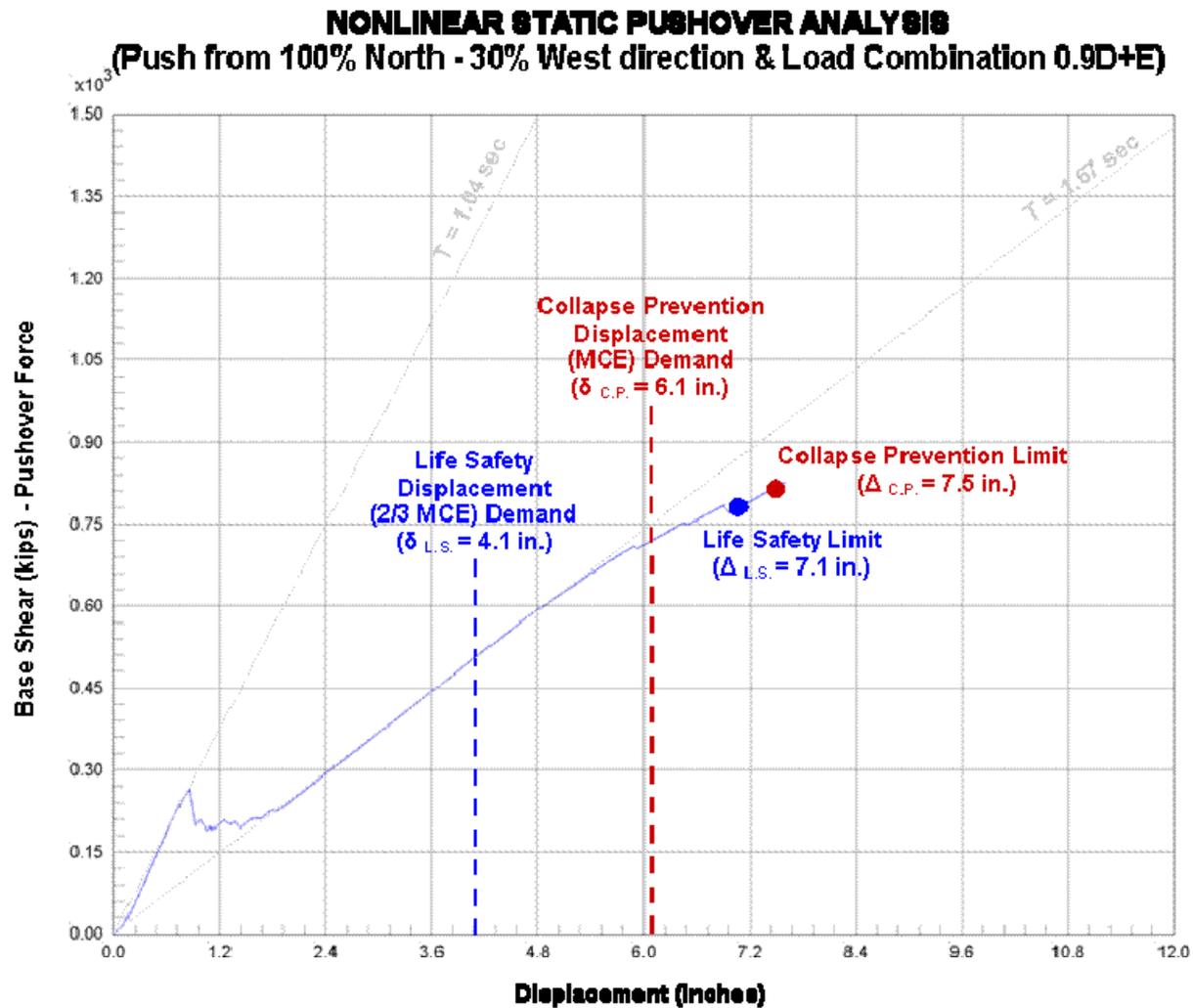


Figure 6-19: Nonlinear pushover analysis results for multidirectional worst-case loads, 100% from North and 30% from West. Vertical load = 0.9D.

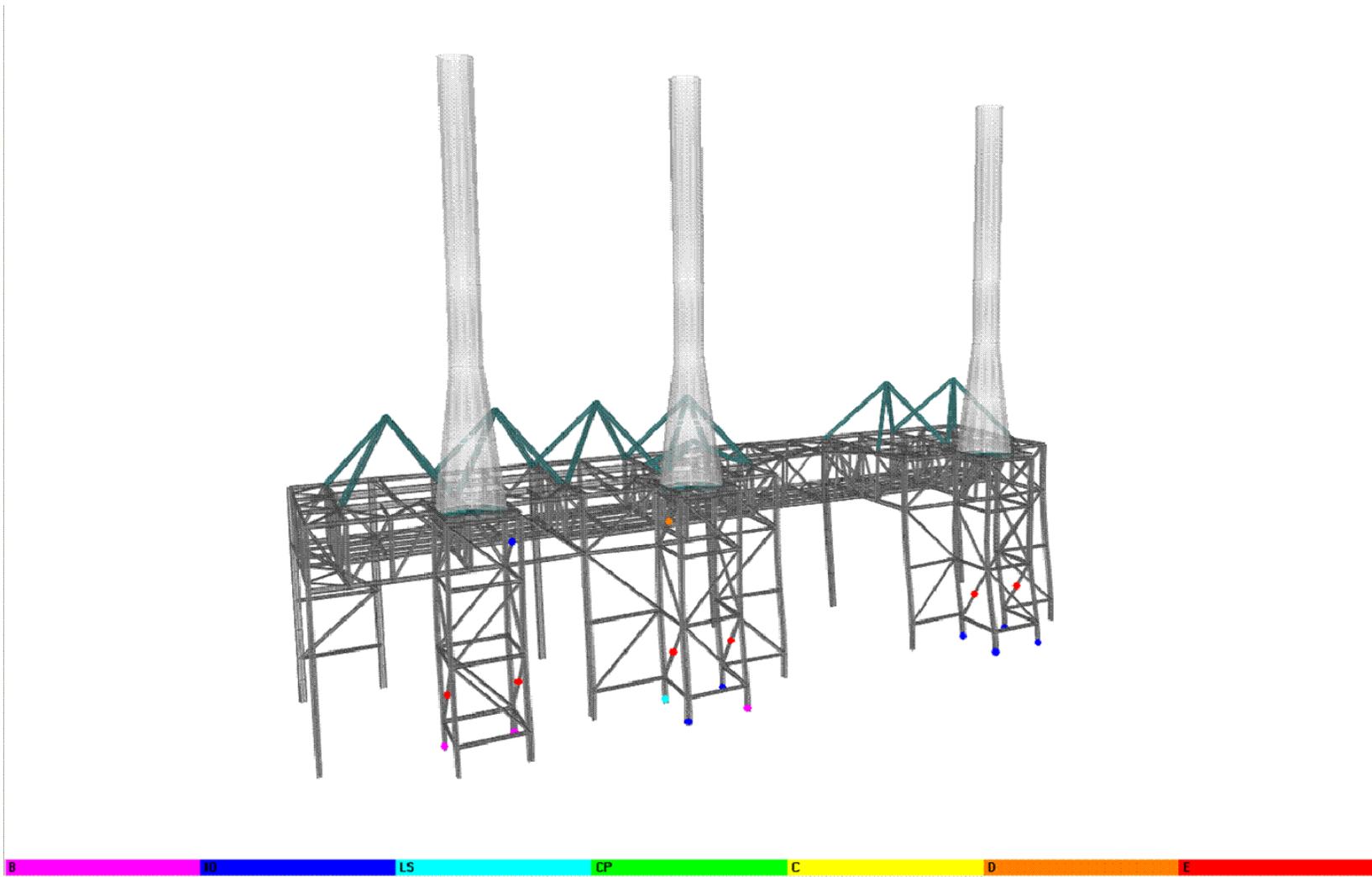


Figure 6-20: Plastic hinges formed at Life-Safety limit state shown in Figure 6-10.

7. STRUCTURAL ANALYSIS – TANK T-12038

7.1 ANALYSIS METHODOLOGY

Tank T-12038 was designed in 1994 to API 650, Appendix E, which is one of the accepted criteria documents in the CalARP guidelines. The tank was checked for seismic capacity using API 650, assuming the inner wall of the tank (primary containment) must resist the entire earthquake loads.

The seismic design procedure in API 650 considers the overturning moment on the tank to be the sum of:

- a) The overturning moment due to the tank shell and roof, together with a portion of the contents which moves in unison with the shell, termed the impulsive component.
- b) The overturning moment due to that portion of the tank contents which moves in the first sloshing mode (i.e., independently of the tank shell), termed the convective component.

Resistance to the overturning moment is provided by the weight of the tank shell and roof and by the weight of a portion of the tank contents adjacent to the shell. The structural adequacy of the tank is determined by a “stability ratio,” which is a measure of the ratio of the overturning moment to the resisting moment. The stability ratio is defined as the quantity $M/[D^2(w_t + w_L)]$ where M is the overturning moment at the bottom of the tank shell (ft - lbs); D is the nominal tank diameter (ft); w_L is the maximum weight of tank contents that may be used to resist shell overturning (lbs per ft of circumference); and w_t is the weight of the tank shell and the portion of the fixed roof supported by the shell (lbs per ft of circumference). The value of the stability ratio must not exceed 1.57 for an unanchored tank. If the stability ratio exceeds 1.57, retrofit or anchorage of the tank or reduction of the liquid height is necessary.

The API methodology is generally considered to be conservative, except where certain conditions are present, such as high yield stress steel and very thin walls. The ASCE Guidelines for Seismic Design of Petrochemical Facilities point out that the

historic performance of anchored tanks is very good and there have been very few problems. They suggest that anchored tanks should be considered as adequate unless anchorage is judged to be capable of tearing the tank shell or bottom plate, causing loss of contents.

7.2 ANALYSIS RESULTS

The stability ratio for the tank was calculated to be 2.54, indicating that anchorage is required. Calculations showed that a minimum of 11 anchor bolts @ $\frac{3}{4}$ " diameter are required. The tank has 12 anchor bolts @ $\frac{3}{4}$ " diameter, so the anchorage is acceptable.

Pile adequacy was checked using the capacities calculated by LMG for the condition in liquefied soils. A maximum base shear of 76 kips results in a lateral load of 6.3 kips per pile. The 12 inch timber pile was calculated to have an allowable bending moment of 38.5 kip-ft. LMG has calculated using their single pile model (Figure 9 of the LMG report) that a lateral load of approximately 8 kips is required to obtain that maximum pile moment. The pile Demand to Capacity Ratio (DCR) of 0.79 is considered to be acceptable.

The piles were also checked for compressive loads and determined to have sufficient capacity with a DCR of 0.93. The largest portion of that demand is due to downdrag force on the pile, due to either liquefaction or settlement. It should be noted that the downdrag force due to liquefaction is unlikely to occur simultaneously with the maximum inertial load due to ground shaking. However, for this calculation, these forces were conservatively considered to act together.

8. CONCLUSIONS AND RECOMMENDATIONS

The analyses presented in this report have demonstrated the seismic adequacy of the CO Boilers and Tank T-12038.

The CO Boilers include two distinct structural systems, one for the spray towers and fireboxes, and one for the precipitators and stacks.

The support structures for the spray towers and fireboxes were shown to be adequate using standard linear analysis techniques. However, these analyses could not be used to demonstrate the adequacy of the precipitator and stack support structures, due to apparent overstress in the braces, as well as in the bottom story of columns, where the lack of bracing causes a stiffness discontinuity.

Nonlinear pushover analyses were used to demonstrate that the structure meets basic safety objectives of being able to survive a design earthquake with minor damage and maintain Life Safety objectives, and that the structure could survive a much larger event without structural collapse.

Tank T-12038 was demonstrated to have adequate capacity for the design event. The foundations were given special attention because of the likelihood of liquefaction in a large earthquake. Based on our evaluation, the tank foundation has sufficient strength due to piles that extend into competent soils below the bay mud.

9. REFERENCES

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2. "Guidelines for Seismic Evaluation and Design of Petrochemical Facilities," Task Committee on Seismic Evaluation and Design of Petrochemical Facilities, American Society of Civil Engineers (ASCE), 1997.
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4. "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," FEMA 356. Prepared by the American Society of Civil Engineers, for the Federal Emergency Management Agency. November 2000.
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APPENDIX A

CaIARP

Seismic Guidance Document

**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

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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 September of 1998 (Ref. 2). 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. When a probabilistic risk assessment approach is planned, the owner/operator should consult with the AA to describe why this approach is being planned and explain differences between this approach and the deterministic method.

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 AA 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 AA 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 failure of systems that contain 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.

Since mid-1999, all new facilities permitted in California have been designed in accordance with the 1997 Uniform Building Code (UBC) (Ref. 3). Beginning in 2005, new facilities in California are currently anticipated to be designed in accordance with the National Fire Protection Association (NFPA) 5000 Building Code which references American Society of Civil Engineers (ASCE) Standard SEI/ASCE 7-02 (Ref. 4) for its seismic provisions. It is the consensus of this Committee that RS systems and components designed and properly constructed in accordance with the 1997 UBC or NFPA-5000 (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 have been permitted in California since mid-1990 (i.e., which 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 having offsite consequences (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.

State and national policies have consistently 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

regular inspection and repair of systems containing RS 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 AA to determine which of the following subsections apply to their facility.

1.4.1 Occurrence of Conditions That Would Trigger the Need for a New Assessment – 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) Significant system modifications, 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 structural members, foundations or anchorages.

1.4.2 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. The analytical portion of the revalidation (Section 4) may rely on a previous CalARP assessment report, provided the report satisfies the reporting Requirements of Section 9 of this guidance document. However, any revalidation should include the performance of a walkdown in accordance with Section 3 of this document.

1.4.3 Existing Facilities Constructed Since Mid-1990 – 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:

- 1) The near field requirements of 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., within 15 km of an active fault).

- 2) The soft soil site conditions of the 1997 UBC were considered in the design of the facility or the facility is not located on a soft soil site.
- 3) A walkdown in accordance with Section 3 reveals adequate lateral force resisting systems.

1.4.4 New Facilities Permitted for Construction After October 2003 That Are Subject to CalARP Program Requirements – Design and construction of new facilities containing RS must satisfy the seismic provisions of the 1997 UBC or the current California Building Code (expected to be NFPA 5000). 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.

1.5 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. For establishing ground motions, any one of the following 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 and other provisions of the 1997 UBC.
- 2) Utilize the Maximum Considered Earthquake (MCE) Spectral Contour Maps developed for the 2000 NEHRP provisions (Ref. 5) and SEI/ASCE 7-02 (Ref. 4). The MCE ground motions should be obtained from the MCE CD ROM which is provided with the 2000 NEHRP provisions. Longitude and latitude of the facility, instead of the ZIP code, should always be used for obtaining the ground motion. 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 values obtained online from the United States Geological Survey (USGS) National Seismic Hazard Mapping Project, which can be obtained at the web site <http://geohazards.cr.usgs.gov/eq/html/canvmap.html>. Longitude and latitude of the facility, instead of the ZIP code, should always be used for obtaining the ground motion.
- 4) Utilize Peak Ground Acceleration (PGA) and spectral acceleration values from the California Geological Survey (CGS), which can be obtained at the web site <http://www.consrv.ca.gov/cgs/rghm/psha/index.htm>. Longitude and latitude of

the facility, instead of the ZIP code, should always be used for obtaining the ground motion.

- 5) 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:
 - a. The spectra should adequately consider current knowledge on source activity levels and attenuation effects through local geology and soils.
 - b. The spectra should adequately reflect local soil conditions and, as appropriate, account for near-fault effects.
 - c. 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:

- i. A 10% probability in 50 years (a mean return period of 475 years); the full value of the spectral ordinates to be used.
- ii. 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 15 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 should be based on the maximum magnitude earthquake that 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:

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

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 "special studies zones" along known active faults in California. Fault maps are described and can be found online at the CGS web site at <http://www.consrv.ca.gov/cgs/rghm/ap/index.htm>.

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. 6). 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.
- 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 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 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 – Tsunamis, or tidal waves, are generated by distant earthquakes and undersea fault movement. Traveling through the deep ocean, a tsunami is a broad and shallow, but fast moving, wave that poses little danger to most vessels. When it reaches the coastline however, the waveform pushes upward from the ocean bottom to make a swell of water that breaks and washes inland with great force.

A seiche occurs when resonant wave oscillations form in an enclosed or semi-enclosed body of water such as a lake or bay. Seiches may be triggered by moderate or larger local submarine earthquakes and sometimes by large distant earthquakes.

A tsunami or seiche may result in flooding of low-lying coastal areas. The greatest hazard results from the inflow and outflow of water, where strong currents and forces can erode foundations and sweep away structures and equipment. The rupture of storage tanks from debris impact and foundation erosion can result in fires and explosions.

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.

3.0 WALKDOWN CONSIDERATIONS

A critical feature of the evaluation methodology is the onsite review 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 basis for assessment may include proven 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 flat bottom 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. 7). Examples of walkdown evaluation sheets are provided in Figure 6.1 of Reference 7 for equipment and References 8 and 9 for piping. Items of concern identified in the walkdown should be addressed in the seismic report.

4.0 ANALYTICAL EVALUATION METHODOLOGY

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

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

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
$D + L + \frac{E}{Q} \leq$	$\left\{ \begin{array}{l} \\ \\ \\ \end{array} \right.$	<p>1.6 x Working Stress Allowable * (without 1/3 increase)</p> <p style="text-align: center;">or</p> <p>ϕR_n (using Load Factors of Unity for all loads)</p>

* For steel beam shear, the capacity should be limited to 1.44 x Working Shear Stress Allowable.

Where,

- D = Dead load
- L = Live and/or operating load
- E = Earthquake load based upon ground motion determined in Section 2.
- ϕ = 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 ACI or AISC Load & Resistance Factor Design (LRFD)

- 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 (i.e. UBC-97, figure 16-3), 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 $0.67 \times T_{\text{peak}}$, the maximum spectral acceleration in the range of $0.5 \times T$ to $1.5 \times T$ 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 or multiple bracing systems), in which seismic loads can be redistributed without failure, the demand (from the previous equation) on an individual bent 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 nonstructural elements 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) standards. 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 UBC base shear equation for design of new facilities, should always 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.

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.

4.2.3 Recommended Guidelines for Seismic Evaluation and Design of Petrochemical Facilities – ASCE (Ref. 7), 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 ANALYTICAL ASSESSMENT OF EQUIPMENT AND NONSTRUCTURAL ELEMENTS

Permanent equipment and nonstructural elements 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 nonbuilding structure.

The supported permanent equipment and nonstructural elements 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 Section 1632.2 of the 1997 UBC or Section 9.6 of SEI/ASCE 7-02. The equipment or the nonstructural element itself should be checked for the acceleration levels based on the above referenced sections. Alternatively, a structure-and-subsystem dynamic analysis using site-specific criteria, as defined in Section 2 of this document, may be performed if the provisions of Section 1632 or Section 9.6 result in excessive demand.

If the permanent equipment or nonstructural element 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 nonbuilding structure, such a standard may be used, provided the ground motion used for analysis is in conformance with the provisions of Section 2. However, equipment and systems that have been previously judged to have met the September 1998 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.

6.0 EVALUATION OF TANKS AT GRADE

6.1 Scope – Vertical liquid storage tanks with supported bottoms should be addressed when they meet one of the criteria in Section 1.2, Evaluation Scope. 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 7 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.

- 7) Tearing of tank shell due to over-constrained walkways connecting two tanks experiencing differential movement.
- 8) Non-ductile anchorage connection details (anchored tanks) leading to tearing of the tank shell or failure of the anchorage.
- 9) 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 tanks at grade 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. If an analytical evaluation is deemed necessary, various industry standards and other methodologies are available in the literature for evaluation of tanks at grade that can be used. These include:

- API 650 Appendix E (Ref. 10) - This method is a standard for the design of new tanks for the petrochemical industry. Its provisions are accepted by the UBC and SEI/ASCE 7 and it addresses both anchored and unanchored tanks.
- AWWA D100 (Ref. 11) - 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.
- Veletsos and Yang (Ref. 12) - This method is primarily for anchored tanks.
- Manos (Ref. 13) - 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.
- Housner and Haroun (Ref. 14) - This method is intended primarily for analysis of anchored tanks but is often used for both anchored and unanchored tanks.
- ACI 350.3-01, (Ref. 15) - Applies to Concrete Tanks (both round and rectangular).

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 10 and 11 listed above, Q factor reductions are inherently included in the determination of seismic forces. In References 12 to 15 listed above, the Q factors should only be applied to impulsive or structural modes (not sloshing modes).

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:

$$d_s = 0.5 D_i S_a$$

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.

This guidance is primarily intended for ductile steel pipe constructed to a national standard such as the American Society of Mechanical Engineers (ASME) B31.3 (Ref. 16). Evaluation of other piping material is discussed in Section 7.1.8.

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

- 1) Identify piping systems to be evaluated.
- 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.

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.

7.1.1 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 (PVC), 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 7.

Additional aspects of piping systems which should also be reviewed during the walkdown 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) Obvious 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 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 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, and
- 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

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 recommended procedures for seismic inertia evaluation of piping are as follows:

- 1) Large unsupported spans of piping may be candidates for significant inertia loads. The inertial moments should not exceed those allowed in Section 7.1.4. 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.

7.1.7 Allowable Stress – The recommended 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:
 - i. Ductile Materials: $S_h = \sigma_t / 3$ at temperature
 - ii. 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" nonbuilding structures (those with framing systems that are specifically listed in the building codes), the procedures and analysis methods outlined in documents such as FEMA 356 (Ref. 17) 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. 6). However, Section 12 of Reference 18, developed by the Los Angeles Section of ASCE, gives additional guidelines for mitigating landslide hazards. Section 8 of Reference 19, 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.

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. The report contents may be broken into two tiers of information. These are recommended minimum contents and recommended supplementary documentation.

9.1 Tier 1: Recommended Minimum Report Contents – CalARP seismic reports should at least contain the following information:

- 1) The reason for performing the seismic evaluation. For example, is it an evaluation of a facility not previously reviewed? Or is it an evaluation that is basically a revalidation of a previous study? If a revalidation, it may be necessary to reference the previous report, and indicate how that previous report was used and the extent to which it was relied upon.
- 2) 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 which therefore were not reviewed should also be noted.
- 3) A characterization of the soil profile at the site, and the basis for that characterization.
- 4) 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, 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. Depending on the circumstances, the assessment may also be based on previous seismic evaluation reports, drawing reviews and/or structural/seismic calculations.
- 6) When obvious, 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) 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) Assessment of existing detection and mitigative systems and, where appropriate, recommendations for new mitigative 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.

9.2 Tier 2: Recommended Supplementary Documentation – It may be appropriate to include the following supplementary documentation in a CalARP seismic report:

- 1) Reference to 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.
- 2) 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.
- 3) Supplementary documentation of the observations made and the assessments performed. These may include photographs (where permissible) and copies of walkdown sheets or field notes.

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.

Cited References:

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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
<p>1. Steel structures</p> <p>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.</p> <p>Ordinary moment frame (see Note 8) The following structural characteristics are usually indicative of a Q=2 value (also see Note 6):</p> <ul style="list-style-type: none"> 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. <p>The following structural characteristics are usually indicative of a Q=4 value (also see Note 6):</p> <ul style="list-style-type: none"> 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. <p>Braced frame The following structural characteristics are usually indicative of a Q=2 value (also see Note 6):</p> <ul style="list-style-type: none"> 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. <p>The following structural characteristics are usually indicative of a Q=4 value (also see Note 6):</p> <ul style="list-style-type: none"> 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. <p>Cantilever column The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 6):</p> <ul style="list-style-type: none"> a. Column splice details cannot develop the column capacity. b. Axial load demand represents more than 20% of the axial load capacity. 	<p>6 or 8</p> <p>2, 4, or 5</p> <p>2, 4, or 5</p> <p>1.5 or 2.5</p>

** 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
<p>2. Concrete structures</p> <p>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.</p> <p>Intermediate moment frame</p> <p>Ordinary moment frame The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 6):</p> <ul style="list-style-type: none"> 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. <p>The following structural characteristics are usually indicative of a Q=2.5 value (also see Note 6):</p> <ul style="list-style-type: none"> 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 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. <p>Shear wall The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 6):</p> <ul style="list-style-type: none"> 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. <p>The following structural characteristics are usually indicative of a Q=3 value (also see Note 6):</p> <ul style="list-style-type: none"> 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° or more. <p>Cantilever pier/column The following structural characteristics are usually indicative of a Q=1.5 value (also see Note 6):</p> <ul style="list-style-type: none"> 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. <p>The following structural characteristics are usually indicative of a Q=2.5 value (also see Note 6):</p> <ul style="list-style-type: none"> 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 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. 	<p>6 or 8</p> <p>4 1.5, 2.5 or 3.5</p> <p>1.5, 3 or 5</p> <p>1.5, 2.5 or 3.5</p>

TABLE 1

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

B. EQUIPMENT BEHAVING AS STRUCTURES WITH INTEGRAL SUPPORTS	Q
<p>1. Vertical vessels/heaters or spheres supported by:</p> <p>Steel skirts The following structural characteristics are usually indicative of a Q=2 value (also see Note 6):</p> <ul style="list-style-type: none"> a. The diameter (D) divided by the thickness (t) of the skirt is greater than $0.441 \cdot E/F_y$, where E and F_y are the Young's modulus and yield stress of the skirt, respectively. <p>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):</p> <ul style="list-style-type: none"> 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. <p>The following structural characteristics are usually indicative of a Q=3 value (also see Note 6):</p> <ul style="list-style-type: none"> 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. <p>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):</p> <ul style="list-style-type: none"> a. Column splice details cannot develop the column capacity. b. Axial load demand represents more than 20% of the axial load capacity. <p>2. Chimneys or stacks</p> <p>Steel guyed</p> <p>Steel cantilever</p> <p>Concrete</p>	<p>2 or 4</p> <p>1.5, 3 or 4</p> <p>1.5 or 2.5</p> <p>4</p> <p>4</p> <p>4</p>

TABLE 1

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

C. PIPEWAYS	Q
<p>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.</p>	
<p>1. Steel Ductile moment frame (see Note 8) Ordinary moment frame (see Note 8) Braced frame Cantilever column</p>	<p>8 6 6 4</p>
<p>2. Concrete Ductile moment frame Ordinary moment frame Cantilever column</p>	<p>8 5 3.5</p>

TABLE 1

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

D. FLAT-BOTTOMED TANKS (see Note 4)	Q
<ol style="list-style-type: none"> 1. Anchored (See Note 9) 2. Unanchored 	<p>4 3</p>
E. FOUNDATIONS (See Note 5)	Q
<ol style="list-style-type: none"> 1. Piled 2. Spread footings 	<p>6 6</p>
F. ANCHOR BOLTS (see Note 6)	Q
<ol style="list-style-type: none"> 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) 	<p>As for structure 1.5</p>

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

TABLE 1

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

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, 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 the 1998 guidance document, the cause(s) and solution(s) were 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 20.

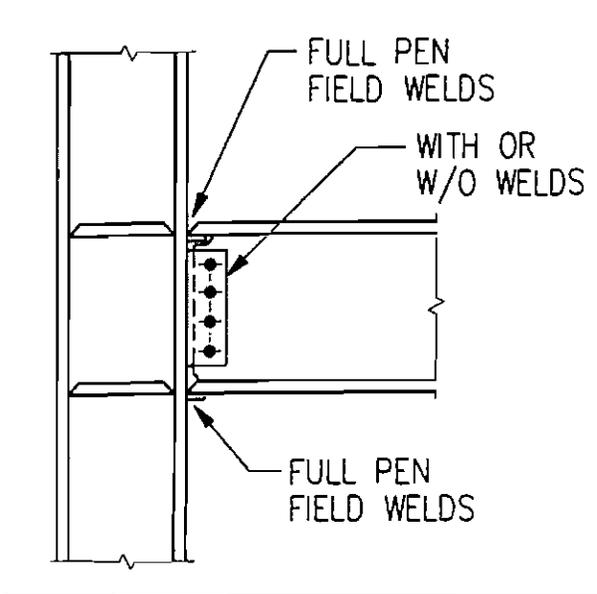


Figure 1: Former standard ductile moment connection detail. As a result of the Northridge Earthquake, this connection was shown to have major problems and is currently prohibited by the UBC in seismic zones 3 & 4.

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

10. The committee is aware that the latest building codes greatly limit or prohibit the use of ordinary steel moment frames and braced frames for new building construction in areas of high seismicity and that these limits and restrictions are being applied to non building structures in some jurisdictions. Based on the performance of these systems in industrial facilities in past earthquakes, the current practice provides adequate seismic

TABLE 1

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

safety for these structures. Therefore, the committee believes the recent restrictions on the ordinary braced and moment frame should not be applied to typical industrial facilities.

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 subconsultant 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 1997 UBC or the latest California Building Code adopted by the local jurisdiction. Parameters such as Soil Profile Type, S; Seismic Zone Factor, Z; Seismic Coefficients, C_a and C_v ; Seismic Source, Type A, B or C; and Near Source Factors, N_a and N_v .
- 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.