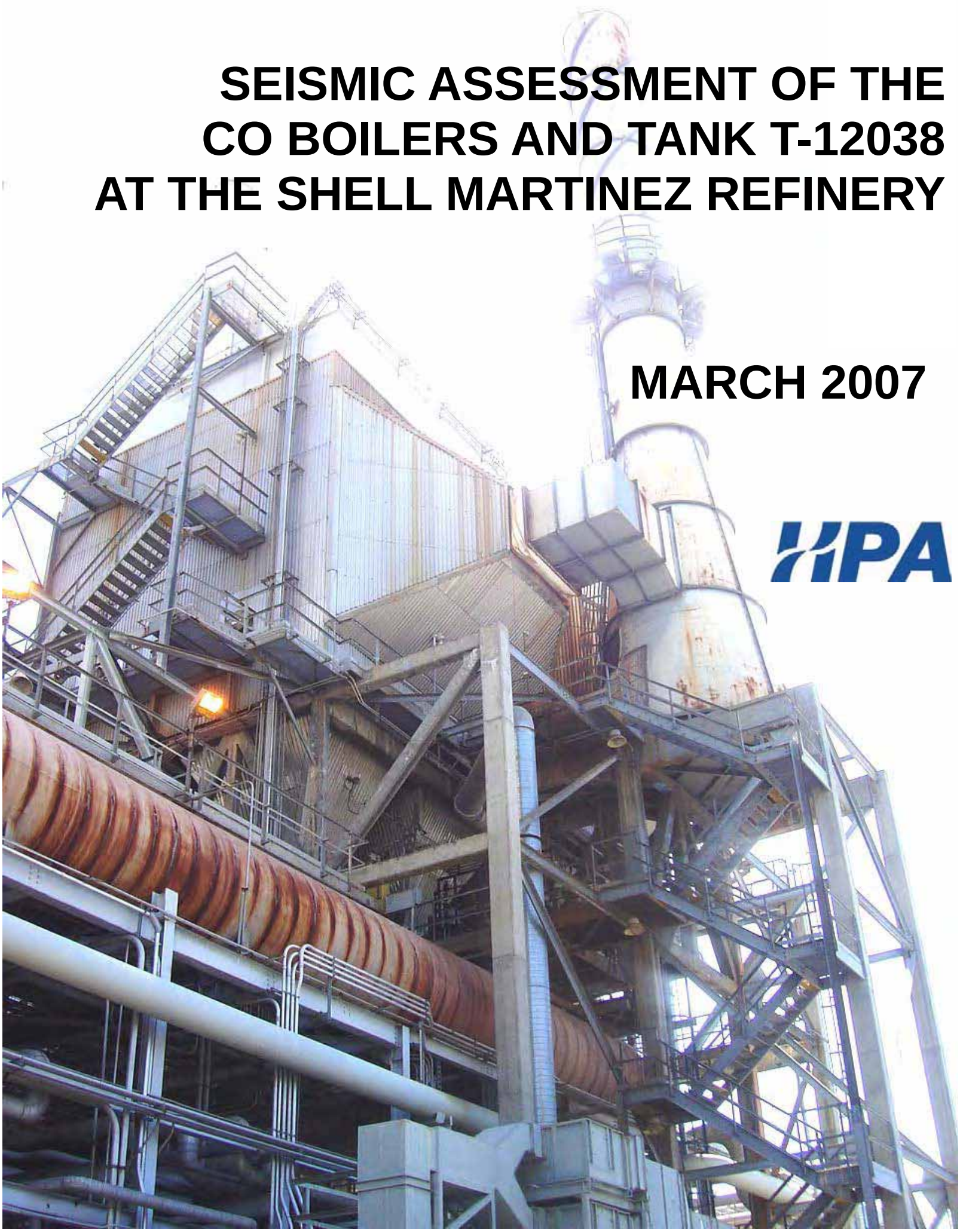
SEISMIC ASSESSMENT OF THE CO BOILERS AND TANK T-12038 AT THE SHELL MARTINEZ REFINERY



MARCH 2007

HPA

SEISMIC ASSESSMENT OF THE CO BOILERS AND TANK T-12038 AT THE SHELL MARTINEZ REFINERY

MARCH 2007

Prepared by:

HPA, Inc.

1939 Harrison Street, Suite 730

Oakland, CA 94612

EXECUTIVE SUMMARY

This report documents a seismic assessment of three CO boilers and a bio-treater tank at the Shell Martinez Refinery in support of their RCRA Part B Hazardous Waste Facility Permit. The assessment was done in accordance with the guidelines for seismic assessments for the California Accidental Release Prevention (CalARP) program, which governs acutely hazardous Regulated Substances in California, and has been widely used for the seismic assessment of other processing units at Shell and other refineries throughout California.

A geotechnical review was performed by Land / Marine Geotechnics (LMG) in support of this seismic assessment and that report was submitted separately to DTSC for review.

In their report, LMG has concluded that the CO boilers are founded on bedrock, and the site is categorically not susceptible to liquefaction. They have provided ground shaking estimates for structural analyses of the CO boilers.

LMG has determined that liquefaction is possible at the tank location, and has provided pile and soil capacities based on liquefied soil conditions for use in the seismic analysis of the tank.

Each CO Boiler consists of two primary structural systems. The boiler and spray tower are supported on a steel braced frame; the electrostatic precipitators and stacks are supported on another steel braced frame.

The boiler and spray tower support frame was analyzed using standard linear analysis techniques, similar to a new design, and evaluated against the CalARP acceptance criteria. The frame was found to have adequate capacity and margin against collapse.

The precipitator and stack structure was evaluated in a similar manner and did not meet the CalARP acceptance criteria for linear analyses, due to overstressed diagonal bracing members, and because the framing configuration lacks diagonal bracing in the lower bay, causing a stiffness discontinuity that is penalized in the CalARP criteria.

That structure was reevaluated using nonlinear pushover analysis techniques, and acceptance criteria from FEMA standards for rehabilitation of existing buildings (FEMA 356). Those criteria define limit states of “Life Safety” and “Collapse Prevention”. Life Safety is a condition where the structure remains stable and has significant reserve capacity, while hazardous nonstructural damage is controlled. Collapse Prevention is where the structure remains standing, and is able to maintain its gravity loads following the earthquake. Any other damage or loss is acceptable. For both limit states, individual structural elements may sustain damage and loss of capacity as long as the overall structure satisfies the limit state.

The FEMA criteria require that the Life Safety objectives be met for the Design Level Earthquake, which is that used for new design. The Collapse Prevention criteria must be met for the Maximum Considered Earthquake, which is 1.5 times the Design Level Earthquake. The structure is considered acceptable if both criteria are met.

The nonlinear analyses have demonstrated that the precipitator and stack support structure may experience some damage in both levels of earthquake in terms of inelastic deformation of individual diagonal bracing members due to compression loads and nonlinear behavior of columns due to bending loads. However, the overall structure remains stable and is able to meet the performance criteria for both Life Safety and Collapse Prevention. Based on these analysis results, no structural retrofit mitigations are recommended for the precipitator and stack support structure.

TABLE OF CONTENTS

	Page No.
1. INTRODUCTION	1-1
1.1 BACKGROUND	1-1
1.2 SCOPE OF WORK	1-1
1.3 ORGANIZATION OF REPORT	1-2
2. DESCRIPTION OF SYSTEMS	2-1
2.1 CO BOILERS	2-1
2.2 TANK T-12038	2-1
3. SEISMIC ASSESSMENT CRITERIA	3-1
3.1 HAZARDS ASSESSMENT	3-1
3.2 WALKDOWNS	3-2
3.3 ANALYTICAL ASSESSMENTS	3-3
3.3.1 Linear Static and Dynamic Analysis	3-3
3.3.2 Nonlinear Static and Dynamic Analysis	3-4
4. GEOTECHNICAL AND HAZARDS EVALUATION	4-1
4.1 GEOTECHNICAL	4-1
4.1.1 CO Boilers	4-1
4.1.2 Tank T-12038	4-1
4.2 HAZARDS ASSESSMENT	4-2
4.2.1 Ground Shaking	4-3
4.2.2 Fault Rupture	4-3
4.2.3 Liquefaction	4-3
5. WALKDOWN ASSESSMENT	5-1
6. STRUCTURAL ANALYSIS – CO BOILERS	6-1
6.1 ANALYSIS METHODOLOGY	6-1
6.2 STRUCTURAL ANALYSIS SOFTWARE	6-2
6.3 ANALYTICAL MODEL	6-2

6.3.1 Material Properties 6-2
6.3.2 Structural Framing 6-3
6.3.3 Boundary Conditions 6-4
6.3.4 Frame Releases 6-4
6.3.5 Hinges 6-4
6.3.6 Operating Loads and Masses 6-5
6.3.7 Earthquake Loads – Response Spectrum Analysis 6-6
6.3.8 Earthquake Loads – Pushover Analysis 6-6
6.4 LINEAR ANALYSIS RESULTS 6-7
6.4.1 Performance Criteria 6-7
6.4.2 Q Factors 6-7
6.4.3 CO Boiler Results 6-7
6.4.4 Precipitator Frame Results 6-8
6.4.5 Foundation Results 6-8
6.4.6 Differential Displacements 6-8
6.5 NONLINEAR ANALYSES 6-10
6.5.1 Overall Performance Criteria 6-11
6.5.2 Limit States 6-11
6.5.3 Target Displacements 6-12
6.5.4 Nonlinear Analysis Results 6-12
6.6 CONCLUSIONS AND RECOMMENDATIONS 6-14

7. STRUCTURAL ANALYSIS – TANK T-12038 7-1
7.1 ANALYSIS METHODOLOGY 7-1
7.2 ANALYSIS RESULTS 7-2

8. CONCLUSIONS AND RECOMMENDATIONS 8-1

9. REFERENCES 9-1

ATTACHMENT A - CalARP Seismic Guidance Document A-1

1. INTRODUCTION

HPA, Inc. (HPA) was contracted by Shell Oil Products U.S. (Shell) to perform a seismic assessment of the CO Boilers and the Biotreater Tank (T-12038) at the Shell Refinery in Martinez, California. The seismic assessment is being performed in support of Shell's Hazardous Waste Facility Part B Permit Renewal Application with the State of California Department of Toxic Substances Control (DTSC). The Responsible Engineer for this assessment is Mr. Gayle S. Johnson, P.E.

1.1 BACKGROUND

DTSC has requested Shell to conduct a "Seismic Assessment" of four permitted units at the Shell Martinez Refinery (SMR) as part of the renewal process for the refinery's RCRA Part B Hazardous Waste Facility permit. These four units include the CO Boiler Nos. 1, 2, and 3 and Tank T-12038. Over several meetings, DTSC staff verbally described the general requirements and contents of such an assessment.

Based on these meetings, Shell has proposed to use the seismic guidelines used for conducting seismic assessments to support the California Accidental Release Prevention (CalARP) program. These criteria are currently being used at SMR and other refineries in California for seismic assessments of equipment and structures housing Regulated Substances that are defined by the state as being acutely hazardous. The CalARP seismic guidance criteria are described in more detail in Section 3 and Appendix A of this report.

DTSC staff provided Shell written comments to the proposed work plan in a Technical Notice of Deficiency (NOD) Letter dated August 12, 2005. This report addresses the comments of the NOD letter.

1.2 SCOPE OF WORK

The scope of work consisted of the following primary tasks:

1. Perform a hazards assessment to address all major earthquake hazards that could affect the site, including ground shaking, soil failure (e.g. liquefaction) and fault rupture.

2. Perform a walkdown evaluation of the system. This is a systematic visual review of the entire system for all three boilers and the storage tank by engineers experienced with earthquake performance of industrial and petroleum facilities.

3. Perform a review of structural drawings for potential vulnerabilities.

4. Perform structural analyses of the primary vertical and lateral load resisting systems for one entire boiler system plus the storage tank.

5. Develop recommendations for seismic risk mitigation.

6. Prepare this report to document the findings of the seismic assessment.

1.3 ORGANIZATION OF REPORT

This report is organized in the following manner:

Section 1 provides an introduction and background to the project.

Section 2 describes the structural and lateral load resisting systems of the CO Boilers and Tank T-12038.

Section 3 describes the seismic assessment criteria used for this project.

Section 4 summarizes the geotechnical and hazards assessment, which is documented in detail in a separate report by Land / Marine Geotechnics, previously provided for review to DTSC.

Section 5 describes the walkdown assessment.

Section 6 describes the structural analysis of the CO Boilers.

Section 7 describes the structural analysis of Tank T-12038.

Section 8 provides a list of recommendations.

Section 9 provides a list of references.

2. DESCRIPTION OF SYSTEMS

2.1 CO BOILERS

The CO Boilers are located in the Light Oil Processing (LOP) area of the Shell Martinez Refinery, and were constructed circa 1965. There are three boilers in the LOP area, almost identical in configuration and structural design. Boiler No. 1 is structurally independent, while portions of Boilers No. 2 and 3 share a common line of framing between them.

Each boiler system consists of a large base-supported firebox with a spray tower on top of it, as shown in the photo of Figure 2-1. The lateral load resisting system for the firebox consists of a steel braced frame with steel plate siding of the boiler welded to the structural columns. The furnace is lined with refractory brick.

Above each furnace is an electrostatic precipitator. These units are steel enclosures anchored to steel braced frames.

A steel stack, approximately 165 feet high, is elevated on the same steel braced frame, and adjacent to each electrostatic precipitator. These stacks were originally constructed on top of the boiler systems (where the spray towers are currently situated), but were relocated to their present position in 1972. The systems are connected with two ducts to each precipitator, with flexible bellows isolating them structurally. The stacks are attached to the steel frames with 16 steel bolts.

A stack and precipitator support frame is shown in Figure 2-2.

The foundation for the CO Boilers consists of a combination of spread footings and mat foundations at the base of each primary column. The geotechnical investigation concluded that all of these units are founded on dense to very dense bedrock.

2.2 TANK T-12038

Tank T-12038 is located in the Effluent Treatment Plant area of the refinery, across Marina Vista Way from the LOP area. The tank, shown in Figure 2-3, stores

waste biosolids and Dissolved Nitrogen Flotation (DNF) Float from the effluent treatment plant.

T-12038 is a double walled tank, designed in 1994 in accordance with American Petroleum Institute (API) Standard 650, and includes specific seismic design provisions. The tank is approximately 21 feet high and 20 feet in diameter with a fixed conical roof.

Tank T-12038 was installed on an existing foundation which previously held Tank T-1065, a 20’ high, 20’ diameter steel tank. That tank and foundation were constructed circa 1962. T-12038 was anchored to the existing pad with 16 epoxied anchor bolts, 7/8” diameter.

The foundation consists of twelve 12-inch diameter timber piles supporting the reinforced concrete pile cap / slab. Extensive soil investigations by Bechtel in 1991 indicate that the subsurface consists of sequentially deeper layers of fill, fine grained Bay Sediments, and alluvium. Stiff to hard clays and dense to very dense clayey sands underlie the Bay Sediments in the tank area. As pile tips elevations are estimated to vary from elevation -33 to -43 feet, the piles are estimated to extend about 14 to 22 feet into the stiff and dense soils below the Bay Sediments.

Figure 2-1: CO Boiler and Spray Tower. Electrostatic precipitator is supported on independent structural frame.

Figure 2-2: Stack and Electrostatic Precipitator

Figure 2-3: Tank T-12038

3. SEISMIC ASSESSMENT CRITERIA

The seismic assessment has been performed in accordance with the CalARP Guidance Document of January 2004, which is included as Appendix A to this report. The CalARP document was developed by a committee of regulatory agencies, industry representatives, and seismic consultants to address the seismic assessment portion of the State of California's mandated program, covered by the California Code of Regulations (CCR). It was first published in 1998 and then updated in 2004. These guidelines are widely used by California Administering Agencies (AA's) in both northern and southern California for the seismic evaluation of facilities containing "Regulated Substances" (referred to as "Acutely Hazardous Materials" in prior legislation). They are specifically required by the Contra Costa County Health Services Department (CCCHSD) in their regulation of refineries and chemical plants within Contra Costa County, including the processing units at the Shell Martinez Refinery.

The CalARP guidance document is intended for use in evaluating existing facilities. It contains requirements for hazard assessments, considerations for the performance of walkdown evaluations, and methodology and acceptance criteria for analytical evaluations. It specifically addresses the evaluation of tanks at grade as well as nonstructural elements and equipment.

3.1 HAZARDS ASSESSMENT

Section 2 of the CalARP guidance document identifies six specific hazards that should be considered:

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

In the NOD letter of August 12, 2005, DTSC has requested that Shell utilize earthquake ground motions specifically computed using probabilistic hazard analysis and the current CGS statewide fault model.

Option 4 of Section 2.1 of the CalARP guidance document is consistent with the DTSC request. The values that can be obtained online at the CGS and USGS websites are the results of probabilistic seismic hazard analyses (PSHAs). These values are interpolated from PSHAs performed for a grid spacing of 0.05 degrees. Interpolated values are typically calculated using the four surrounding corner points. Values are calculated for “firm rock” sites, and must then be adjusted for site specific soil conditions.

3.2 WALKDOWNS

Section 3 of the CalARP guidance document identifies a walkdown of the facility as a critical feature of the evaluation methodology. 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 piping systems is discussed in Section 7 of the CalARP guidance document.

The CalARP guidance document also references ASCE’s “*Guidelines for Seismic Evaluation and Design of Petrochemical Facilities”* for further description of the walkdown process.

3.3 ANALYTICAL ASSESSMENTS

Section 4.2 of the CalARP guidance document specifies that demonstrating acceptability of the structure can be accomplished using either a) linear static or dynamic analysis, or b) nonlinear static or dynamic analysis.

The CO Boilers and T-12038 were assessed first using linear analysis methods, as they are the simplest and most straightforward to apply. As discussed in detail in Section 6, acceptability could not be demonstrated for the CO Boilers using this approach, so the more sophisticated nonlinear analysis methods were applied.

3.3.1 Linear Static and Dynamic Analysis

Section 4.2.1 of the CalARP seismic guidance document describes the methodology for linear static and linear dynamic analysis, which is one of the accepted methods.

The demand equation for this approach is a simplified version of that found in building codes, such as the Uniform Building Code (UBC) or International Building Code (IBC).

DEMAND		CAPACITY BASED ON
$D + L + \frac{E}{Q}$	$\leq$	1.6 x Working Stress Allowable * (without 1/3 increase) or $øR_n$ (using Load Factors of Unity for all loads)

Where,

D	=	Dead load
L	=	Live and/or operating load
E	=	Earthquake load based upon ground motion determined in Section 2.
ø	=	Capacity reduction factor (per ACI) or resistance factor (per AISC)
Q	=	Ductility based reduction factor per Table 1 of the CalARP guidance document
R_n	=	Nominal capacity per ACI or AISC Load & Resistance Factor Design (LRFD)

The "Q" factor is analogous to the "R" factor in Section 1630.2 of the 1997 UBC, except that the values for "Q" in Table 1 of the CalARP guidance document were developed specifically for the types of structures and installations typically found in process facilities, and for the type of conditions that are likely to be found in existing facilities that are not covered in conventional building codes for new design.

3.3.2 Nonlinear Static and Dynamic Analysis

Section 4.2.2 of the CalARP seismic guidance document discusses briefly the acceptance criteria for nonlinear analyses. The document states that, "*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."

For the analysis of the CO Boilers, a nonlinear static pushover approach was used, implementing the acceptance criteria provided in FEMA 356, "*Prestandard and Commentary for the Seismic Rehabilitation of Buildings*." The provisions of FEMA 356 are based on a performance-based design methodology that differs from seismic design procedures used for the design of new buildings currently specified in national building codes and standards. The approach and acceptance criteria used for the final analysis of the CO Boilers are described in detail in Section 6.5.

4. GEOTECHNICAL AND HAZARDS EVALUATION

A geotechnical evaluation and hazards assessment has been prepared by Land / Marine Geotechnics (LMG) for this study. The work was performed under the direction of Mr. R. William Rudolph, G.E., a California Registered Professional Geotechnical Engineer. Their work has been documented in a separate report, which was previously submitted to Shell and DTSC.

4.1 GEOTECHNICAL

4.1.1 CO Boilers

LMG reviewed original soil reports from 1964, prepared for the construction of the LOP area. LMG has determined that the CO Boilers are in an area where grading consisted of cutting up to 60 to 70 feet. The cuts removed all soil deposits and exposed dense to very dense bedrock throughout the CO Boiler site.

LMG has recommended the following criteria for use in the seismic assessment:

- Lateral loads resisted by a combination of passive pressure acting on the vertical faces of the footings and friction along the bases of the footings
- Passive pressure triangular distribution of 350 pcf
- Upper one foot of soil ignored
- Base friction coefficient of 0.40
- Factor of safety of 1.5

4.1.2 Tank T-12038

LMG has reviewed the investigations performed by Bechtel in 1991, as part of a comprehensive geotechnical investigation for the design of the Effluent Treatment Plant. Numerous test borings and cone penetration test (CPT) probes were conducted in the area of T-12038.

Test borings in the area indicate that T-12038 is underlain by sequentially deeper layers of fill, fine grained Bay Sediments and alluvium, which is in turn underlain by

bedrock. Stiff to hard clays and dense to very dense clayey sands underlie the Bay Sediments in the tank area.

Pile tip elevations are estimated to vary from elevation -33 to -43 feet. Piles are anticipated to extend about 14 to 22 feet into the stiff and dense soils below the Bay Sediments.

LMG has computed vertical downward pile capacities which include anticipated downdrag loads due to settlement of the peat and bay mud layer as well as liquefaction related settlement during an earthquake. Vertical pile capacities in compression are shown in Figure 8 of the LMG report and range from about 30 to 130 kips per pile, depending on the pile length.

Lateral pile capacities of the piles were also evaluated by LMG. A single pile is modeled with P-Y springs along its length to model the soil resistance and pile deflection at various depths, and incorporates the structural properties of the pile. Residual strengths associated with liquefaction were used to account for seismic effects on these piles.

The lateral force vs. deflection and lateral force vs. pile moment curves are shown in Figure 9 of the LMG report. It should be noted that these analyses assume that failure is in the soil, and not in the pile itself. The pile was checked as part of the evaluation described in Section 7.

4.2 HAZARDS ASSESSMENT

A hazards assessment was performed by LMG consistent with the CalARP criteria. The following site-specific seismic hazards were considered.

- Ground shaking, including local site amplification effects
- Fault rupture
- Liquefaction and lateral spreading
- Seismic settlement
- Landslides

- Tsunami

A summary of the likelihood of these hazards is given in the following sections.

4.2.1 Ground Shaking

LMG has calculated ground motions at the two sites using values obtained from the USGS National Seismic Hazard Mapping Program. The values are results of probabilistic seismic hazard analyses (PSHAs) performed using the current California Geological Survey statewide fault model. For the CO Boiler site, no adjustment is necessary for the site class. For the T-12038 site, a correction for site class was performed. Values are presented in Table 2 of the LMG report. Response spectra are given in Figures 6 and 7 of the LMG report.

4.2.2 Fault Rupture

The California State Geologist is required to delineate “special studies zones” along known active faults in California. Review of the latest CGS fault maps indicates that neither site is within a special studies zone. The nearest active fault is the Concord Fault, which is more than 4 km from either site.

In May of 1989, Woodward Clyde Consultants prepared a report on the “Evaluation of Holocene Faulting in the Vicinity of the CO Boiler Waste Feed,” which is where T-12038 is currently located. No evidence of Holocene fault displacement near the site of T-12038 or the CO Boilers was found. LMG has concluded that the risk of fault rupture passing through the CO Boiler or T-12038 sites is very low.

4.2.3 Liquefaction

The CO Boilers are founded on bedrock which is categorically not susceptible to liquefaction, and the risk of liquefaction at the CO Boilers is considered to be nil.

The soils under T-12038 were evaluated by LMG using the guidelines of Special Publication 117, and they have concluded that the thin sand layers are susceptible to liquefaction. The practical impact of this liquefaction is settlement estimated on the order of one inch, and downdrag forces acting as additional external loading on the piles. The piles founded in the stiff soils below the liquefiable layers will prevent settlement of the pile cap / slab itself.

Bechtel has concluded that “If liquefaction develops in this area, it is expected to be local in extent, and will not be a design concern for pile-supported structures.” Potential effects would be some local surface settlement. Permanent lateral movements or ground lurching is not anticipated. LMG has concurred with Bechtel’s assessment that the pile foundations are designed to accommodate downdrag forces due to settlement. They also have concluded that the possibility of lateral spreading is very low.

As discussed in Section 4.1.2 above, the effects of liquefaction have been incorporated in the assessment of pile and soil capacities.

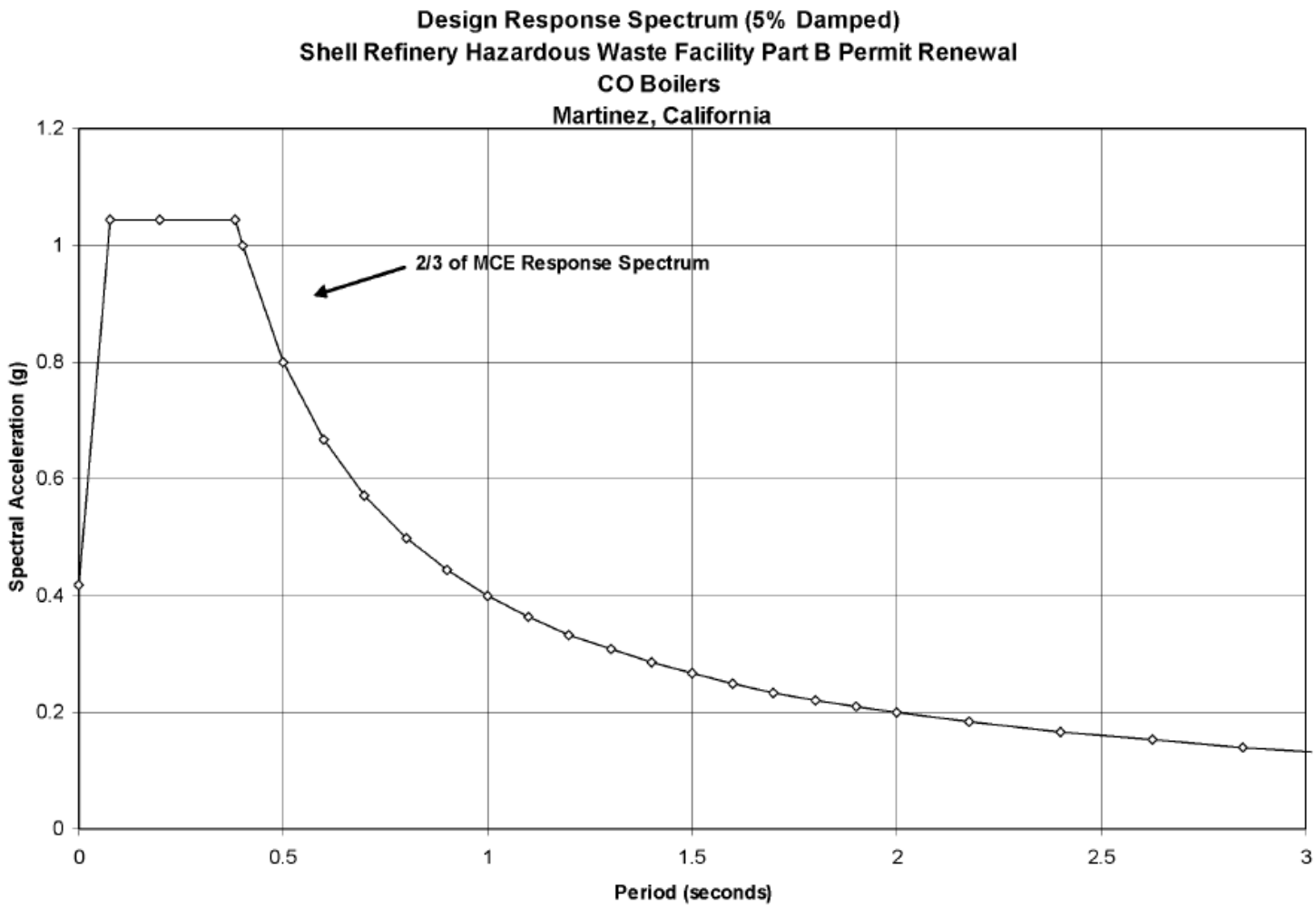


Figure 4-1: 2/3 MCE response spectrum for CO Boilers. Reproduced from LMG report.

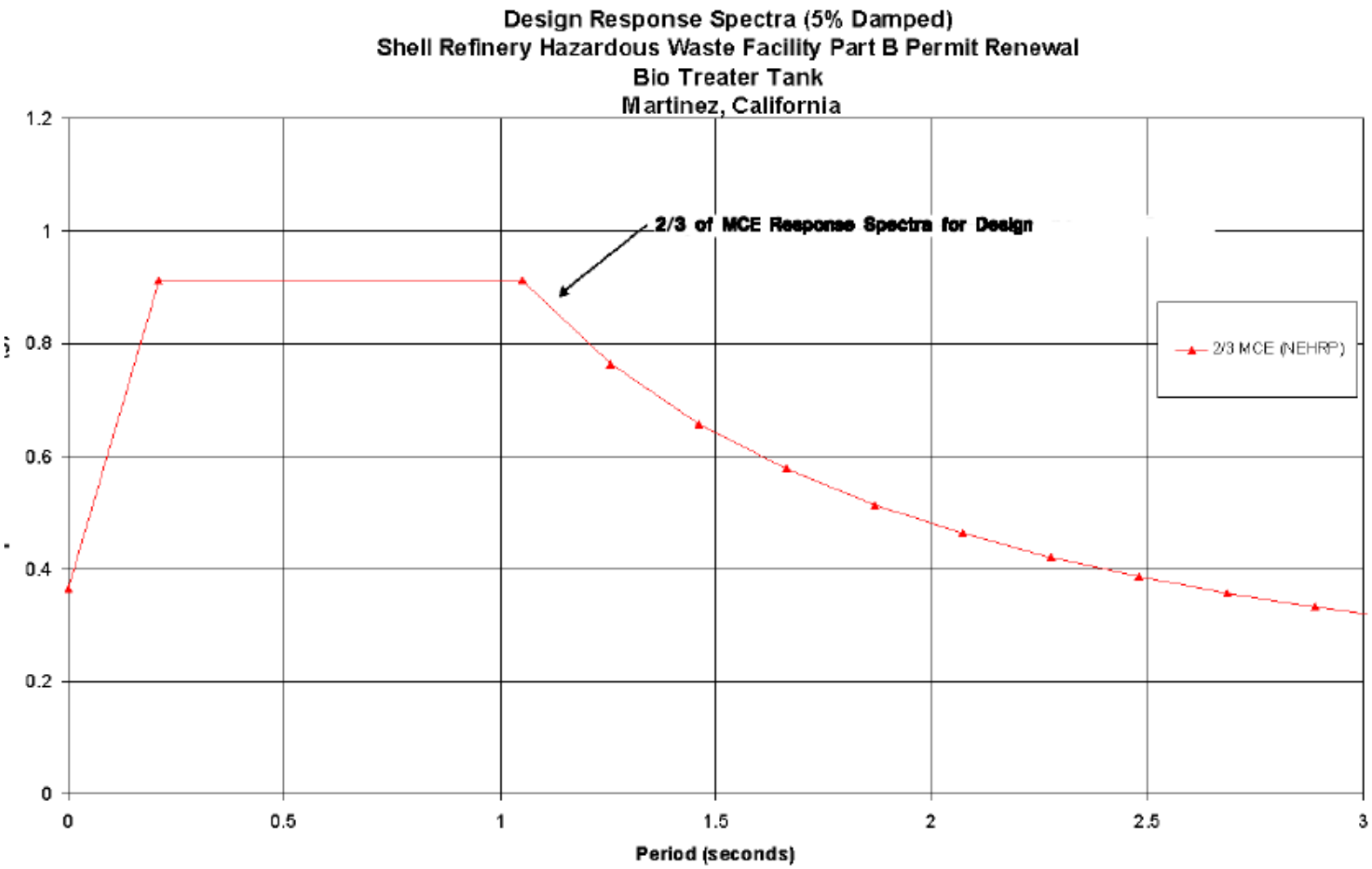


Figure 4-2: 2/3 MCE response spectrum for Tank T-12038. Reproduced from LMG report.

5. WALKDOWN ASSESSMENT

As discussed in Section 3.2, a key feature of the CalARP evaluation methodology is an on-site walkdown review of the existing facilities. This is a primarily visual review that considers the actual conditions of each installation in a systematic, methodical manner. The engineers performing the review investigate for potential seismic vulnerabilities, focusing on proven failure modes from past earthquake experience and engineering judgment. The walkdown review also emphasizes the primary seismic load resisting elements, and the potential areas of weakness due to design, construction, or modification practices. A special emphasis is placed on details that may have been designed without consideration of seismic loads. Additional guidance on the walkdown review can be found in the ASCE publication *Guidelines for Seismic Design and Evaluation of Petrochemical Facilities*.

A walkdown of all three CO Boiler systems was performed. This included the primary structures, supported equipment such as steam drums, control equipment, soot blowers, and ducting and piping.

In general, we found the lateral load resisting systems to be complete. We did note the presence of built up sections on the diagonal bracing of the stack supports. Additional drawing reviews indicated that these sections were created by strengthening the original design T-shape members some time after the original design. The structural model was modified to reflect this change.

Equipment and piping were well supported. Systems crossing between structural systems generally appeared to be adequately capable of withstanding differential movements.

The only concern noted during the walkdown was the flexibility of the bellows between the electrostatic precipitators and the cooling towers. These are supported on separate structural systems and could displace differentially due to out-of-phase earthquake motions. We were concerned that the amount of displacement could exceed the displacement capacity of the bellows. This was identified as a check to perform during the structural analysis described in Section 6. Results of that assessment are discussed in Section 6.4.6.

6. STRUCTURAL ANALYSIS – CO BOILERS

6.1 ANALYSIS METHODOLOGY

The seismic evaluation of CO Boilers No. 1, 2 and 3 was performed using the basic CalARP seismic assessment criteria, which is similar to that used for the design of new structures. Key features of the methodology include the following:

- Linear dynamic response spectrum analysis
- Ground accelerations equal to the Design Level Earthquake as used in current building codes, equal to 2/3 of those from the Maximum Considered Earthquake (MCE)
- Stress levels of each individual member compared to allowable values
- Demonstrating seismic adequacy by passing member stress checks for each individual primary framing member

Where this method could not be used to demonstrate adequacy, additional assessments were performed using nonlinear static pushover techniques. These techniques are commonly used for rehabilitation of existing structures, and provide a performance-based design. Key features of this methodology include the following:

- Structure pushed statically in each direction until set performance limits are achieved (FEMA 356 Life Safety and Collapse Prevention).
- Displacement of structure calculated at 2/3 MCE (Design Level) and at MCE, including inelastic deformation.
- If displacement at set performance limits exceeds displacement caused by designated earthquake levels and individual member inelastic limits are not exceeded for critical structural elements, the structure is presumed to achieve performance goals.

Additional discussion on the two methodologies is provided in Sections 6.4 and 6.5.

Because of the nearly identical design of each boiler, the original intent was to evaluate the primary structural framing for one entire boiler system (boiler, stack, and electrostatic precipitator), and use the results to draw conclusions regarding the other two similarly arranged boilers. During the course of the assessment, the scope of the analysis was expanded to explicitly model all three electrostatic precipitator units and stacks, with their supporting structures and connecting secondary framing.

6.2 STRUCTURAL ANALYSIS SOFTWARE

All analyses of the CO Boilers, including dynamic response spectrum analyses, steel frame design checks and nonlinear static pushover analyses, were performed using the proprietary computer software SAP2000 Advanced Version 10.1.1, developed by Computer and Structures, Inc. of Berkeley, California.

6.3 ANALYTICAL MODEL

As discussed in Section 2.1, each CO Boiler system comprises two independent and structurally isolated steel frames. The first supports the firebox, spray tower, and associated equipment. The second supports the electrostatic precipitator and the stack. Finite element computer models were created of each structure to represent the primary structural framing properties and mass characteristics.

The model of the boiler structure is shown in Figure 6-1. The model shown is Boiler No. 3 (F-68).

The original model of the electrostatic precipitator and stack structure addressed only Boiler No. 3. After running initial analyses, we suspected that the adjacent structures may have greater influence on the response of these flexible structures than for the very stiff boiler structure. As a result, the models of all three precipitators and stacks were coupled into one model. The resultant model used for analyses is shown in Figures 6-2 and 6-3.

6.3.1 Material Properties

The material properties assigned to the structural framing and processing equipment are generally those specified in the original design documentation. In the

absence of detailed general notes for the CO Boiler, consistent material specifications were assumed.

All structural steel shapes, plates, and sheets were originally taken to be ASTM A36 with a yield strength of 36 ksi and a tensile strength of 58 ksi. For the nonlinear pushover analysis, these strengths were increased for the Precipitator frame members to a yield strength of 44 ksi and a tensile strength of 66 ksi to model expected-strength steel properties, in accordance with standard pushover analysis techniques. The stack shell is ASTM A283 Grade C steel with a yield strength of 30 ksi and a tensile strength of 55 ksi. The spray tower shell is ASTM A283 Grade D steel with a yield strength of 33 ksi and a tensile strength of 60 ksi.

Several structural steel members in the precipitator frame are encased in fireproofing. The unit weights for the column and beam fireproofing were taken as 150 pcf and 90 pcf, respectively. The fireproofing material was not considered to increase the stiffness of the member, and was only applied as additional mass.

Varied concrete properties were used for the foundations. For the CO Boiler, the foundation concrete was specified to have a minimum strength of 2500 psi. For the precipitator frames, the foundation concrete was specified to attain a minimum strength of 3000 psi. The reinforcing steel was taken to conform to ASTM A615 Grade 40 with a yield strength of 40 ksi, as specified on the Precipitator drawings.

6.3.2 Structural Framing

Original design drawings of primary structural framing members were available to model the overall geometry and member properties of each frame. The original drawings did not reflect a relocation of each stack from on top of its firebox to the separate elevated precipitator frame. We had also noted during our walkdown reviews that the diagonal bracing on the stack support frames had been changed from a T-shape to an I-shape by welding bottom flanges to several of the beams. That modification, apparently also done at the time of stack relocation, was incorporated into the model.

The CO Boiler was modeled using a combination of frame elements and plate elements. The frame elements were used for the beams and columns, and consisted primarily of W-shapes, WT-shapes, C-shapes, L-shapes, and built-up shapes. Pipe

frame sections were used to model the spray tower. Area sections were used to model the relative stiffness and mass of the heater walls. All area sections were defined as plates with equivalent membrane and bending thicknesses. Mass and weight modification factors were applied to the area sections encasing the fire box to represent existing brick insulation. No other modification factors were applied.

The precipitators and stacks were modeled using frame sections only, including W-shapes, WT-shapes, C-shapes, L-shapes, and built-up shapes. Mass and weight modification factors were applied to several precipitator frame sections to account for concrete fireproofing. The stack was modeled using pipe frame sections in combination with tapered nonprismatic frame sections. General frame sections were used to model the precipitator. These sections were rigid links with no self-weight and significant stiffness, employed to accurately position the precipitators' centers of gravity and distribute their loads and masses to the supporting structure. Similarly, general frame sections were used to position a control node at the center of mass of the entire precipitator from which to monitor overall structural displacements.

6.3.3 Boundary Conditions

The analytical models of the CO Boiler and Precipitator employed translational and rotational restraints at the base of the steel columns that were anchored to the concrete foundation. For the CO Boiler, three different restraints were applied based on the anchorage, base plate detailing, and column load reactions furnished by the manufacturer. The CO Boiler was modeled with 6 fully fixed columns, 2 pinned columns, and 3 pinned in one lateral direction only. For the entire Precipitator, the column bases were assumed to be fully fixed as indicated by their anchorage and base plate detailing. No other points of fixity were applied.

6.3.4 Frame Releases

Diagonal braced members primarily serve as axially loaded sections, carrying lateral loads through the structural system. Major and minor axes moment releases were assigned at both ends of the diagonal braces.

6.3.5 Hinges

Force-displacement and moment-rotation hinges are discrete user-defined elements assigned to structural frame members to model inelastic behavior in the

nonlinear static pushover analyses. These hinges are only active during nonlinear analyses, and behave as rigid members with no effect during any other type of analysis.

To monitor the nonlinear behavior of the Precipitator's primary lateral force resisting system, hinges were implemented in critical column and brace locations to capture the formation of yielding mechanisms. Coupled axial and moment hinges (P-M2-M3) were applied to each end of the column members (offset 5% of the member length at each end). One uncoupled axial hinge (P) was inserted at the center of each diagonal brace in the lateral force resisting system. While the locations of hinges are user defined, hinge properties were automatically generated in the model as a function of the frame's steel section and geometry per FEMA 356 guidelines. All hinges were defined as primary components with zero load carrying capacity beyond point "E" on the FEMA 356 member force deformation curve, as discussed in Section 6.5.2 and Figure 6-9. The hinge unloading method was specified as local redistribution. When the capacity of a member hinge begins to drop (corresponding to a negative stress-strain slope), this method decreases the load on the hinged member and redistributes this load to neighboring members, effectively simulating the stabilization of local inertia forces due to rapid unloading of the structural member.

6.3.6 Operating Loads and Masses

SAP analyses utilize loads and masses in two distinct ways: (1) loads are applied in units of weight and contribute to static analyses only, and (2) masses are applied in units of weight per g (32.2 ft/s^2) and contribute to dynamic analyses only. Therefore, equivalent loads and masses were placed on the models to capture both the static and dynamic performance of the structures.

Self-weight loads and masses for both the CO Boiler and Precipitator framing were assessed by SAP as a function of the structural steel framing and geometry. These self-weight values address the spray tower, stacks, and concrete fireproofing loads and masses. The unit weight for the steel used in these analyses was 490 pcf.

For the CO Boiler, additional operating loads and masses applied included the steam drum, silencer, flue, steel grating, stairs, platforms, ducts, and miscellaneous components. These loads and masses were introduced either at discrete points or along member lengths.

For the Precipitator, additional operating loads and masses were applied to model the precipitators, conveyors, steel grating, stairs, platforms, ducts, and miscellaneous components. Of these items, the precipitators SP-75, SP-76, and SP-77 are the most significant, with a dead load of 638 kips and a live load of 8 kips each (i.e. a total operating load of 646 kips). The dead load values were taken directly from the vendor's design drawings. The live load values were based on the maximum fill height and measured density of dust in the hoppers, taken from shipping data and operations limits.

6.3.7 Earthquake Loads – Response Spectrum Analysis

For the linear analyses, horizontal earthquake loads are calculated using the 2/3 MCE evaluation response spectrum recommended by LMG, as described in Section 4.1. The vertical response spectrum is taken as 2/3 the horizontal spectrum.

The worst case loads for each member are calculated using a "100-30-30" combination, where the loads in each direction are combined with 30% of the loads from the two orthogonal directions simultaneously. Note that this is more conservative than requirements for new design, which consider loads in only one horizontal direction, with a factor to account for some equivalent vertical load that is much smaller than used here.

6.3.8 Earthquake Loads – Pushover Analysis

Earthquake loads for the nonlinear pushover analyses are simulated by incrementally displacing the center of gravity of the structure laterally in the direction of interest. Displacements, rotations, and member strains are monitored at each step, and the structure is displaced until those strains go beyond specified criteria. As individual members are pushed beyond linear limits, the changes in structure stiffness and period are taken into account in the analysis.

The structure was pushed in each of the four primary horizontal directions to determine the worst case loading. As a final case, the structure was pushed with 100% in the worst case direction, and 30% in the orthogonal horizontal direction.

For the pushover analyses, vertical loads are not incremented in the same manner as the horizontal loads. Rather, vertical loads are applied as two load cases proportional to the dead load D. The first case is 1.2D; the second is 0.9D.

6.4 LINEAR ANALYSIS RESULTS

6.4.1 Performance Criteria

The performance of each structure is determined using standard steel design techniques similar to those found in building codes for new design. The loads due to accelerations are divided by a reduction factor, “Q”, as discussed in Section 3.3.1. After combining earthquake and dead loads, the demand on each individual member is checked against its capacity. If the Demand to Capacity Ratio (DCR) is less than 1.0, the member is presumed to be adequate. If all primary structural framing members are adequate, the entire structure is presumed to be adequate.

Both the CO Boiler and Precipitator were evaluated using the SAP steel frame design tool which implements the 1999 American Institute of Steel Construction Load & Resistance Factor Design (AISC-LRFD99) code. The program calculates and displays the member DCRs both numerically and by color.

6.4.2 Q Factors

Table 1 of the CalARP guidelines in Appendix A provides the Q factors for various structural systems. For these analyses, Q factors ranging from 2 to 6 were used, as shown in Table 6-1.

The value of Q=5 is that normally applied to steel braced frames. This value was assigned to the entire CO Boiler frame because the rigid firebox provides structural stiffness similar to that of a braced frame. The Q=5 value was also assigned to the braced portions of the Precipitator frame, located above the base column sections.

The value of Q=2 was used for the unbraced column sections at the base of the Precipitator frame, where there is a stiffness discontinuity in the global structural system. The reduced Q value results in a severe penalty for using this design detail.

6.4.3 CO Boiler Results

All DCRs for the CO Boiler structure were less than 1.0. The details of the highest loaded column and brace are shown in Table 6-2. Figure 6-4 shows the location of those members, and indicates by color coding the DCRs of other members throughout the entire structure. As shown, the majority of the structure is blue, indicating DCRs of less than 0.5.

Based on this, the structural framing for the CO Boiler structure is considered to be adequate.

6.4.4 Precipitator Frame Results

Figure 6-5 shows graphically the locations and DCRs for those braces and columns with DCRs greater than 1.0 for the steel braced frame with Q=5.

Figure 6-6 shows the same information only for those column members in the unbraced bay at the base of the structure, taken from the analysis with Q=2.

Table 6-3 provides the details for the highest loaded brace and column members.

Given the overstressed members in this analysis, and the significant amount of overstress, this linear analysis cannot be used to demonstrate the adequacy of the structure for earthquake loads. Options for mitigation include:

a) Seismic retrofit

b) Reassessment using a more rigorous approach to determine whether retrofit is actually required (as per paragraph (7) in Section 4.2.1 of the CalARP guidelines).

Section 6.5 describes the additional assessments performed on the Precipitator structure only to determine whether retrofits are actually necessary.

6.4.5 Foundation Results

The foundations were checked for maximum bearing capacity on the columns using the ductility reduction factor of Q=6, and the recommended bearing capacities provided by LMG. A DCR of 0.7 was calculated, which is considered acceptable.

6.4.6 Differential Displacements

Each precipitator is attached to its CO Boiler spray tower on the north side and its stack on the south side via two expansion bellows each side. The displacements were evaluated to determine the possible deformation that might be taken up by the bellows due to earthquake loads, should the supporting structures move out-of-phase.

The attachment between the precipitator and stack is considered to not be an issue for two reasons:

a) They are supported on a common structure, with the bellows very near the stack base. Therefore, we do not expect that the displacements will be out-of-phase, and the relative displacements will likely be very small.

b) We understand from Shell staff that the content of the gas on this side of the precipitator is the same as coming out of the stack, so is not a significant concern, should a leak occur at this location.

Table 6-4 shows the maximum absolute displacements at the precipitator and spray tower. Response spectrum analyses can only provide the absolute maximum displacement; however, the earthquake can hit in any direction, and the maximum displacements could be either positive or negative, and they may not occur at the same time. For this reason, the absolute maximums are combined using SRSS to provide the maximum total relative movement. Table 6-5 indicates a maximum relative displacement at the Precipitator / Spray Tower bellows of 3” transverse combined with 0.8” longitudinal, or 2” longitudinal with 1.3” transverse.

Figure 6-7 shows one of the bellow connections between a precipitator and spray tower. Measurements of the existing offsets of all 6 connections indicate separations of approximately 4” longitudinally, and very small offsets of less than an inch transverse and vertical. Existing folds of the material appear to indicate a capability of absorbing a few inches of additional displacement.

We also reviewed the details of the expansion joints that are currently in place from vendor drawings and design submittals. Two designs are currently being used, with one from Senior Flexonics, Inc. shown in Figure 6-8, and a nearly identical design from Pathway, Inc.

Both designs indicate an ambient condition separation of 6” prior to compression of the expansion joint. The “liner plate” (shown as circled number “5” in Figure 6-8) overlaps the spray tower side by more than 1” to more than 3” for the two designs in the ambient condition preventing material from reaching the fabric belt of the expansion joint (gas flow is from left to right in the figure). In the operating condition, this overlap

will be from 3” to 5”, much larger than the potential maximum extension of 2” due to relative movement in an earthquake.

The liner plate is separated from the spray tower side duct by 1-1/8” in both designs. The liner plate will act as an effective restraint and limit the potential differential transverse movement at the expansion joint. Again, the overlap of the liner plate will prevent gas from flowing back and contacting the expansion bellows.

In summary, the design of the expansion joint appears to be capable of withstanding the maximum seismic movements without a release of materials.

One other related differential movement item was noted in the walkdown review for this area for the thermowells on the west side bellows for CO Boiler No. 3. These penetrate through an insulating thermal shield that protects personnel on the platform with no clearance. Shell operators have identified that these thermowells are used to monitor inlet gas temperature and do not have an active control function. As such, damage to the thermowells in an earthquake would not result in any release scenarios. However, Shell has modified these penetrations to allow a minimum of 2” clearance in the longitudinal direction of the duct.

6.5 NONLINEAR ANALYSES

As discussed in Section 3.3.2, the nonlinear analyses used to reassess the Precipitator support structure were performed using static pushover techniques, with acceptance criteria consistent with FEMA 356. That document is specifically referenced in Section 8 of the CalARP Guidelines for “building-like” nonbuilding structures such as this frame.

6.5.1 Overall Performance Criteria

The primary structural members of the frame were identified as the main columns and beams. These are the members that provide capacity of the structure to resist collapse under seismic forces in any direction.

For these members, damage and some degradation of stiffness is allowed to occur. However, the overall function of these elements in resisting collapse cannot be compromised.

Other members, such as braces, are classified as secondary members, in that they affect the lateral stiffness and distribution of forces, but are not essential to collapse prevention.

For these members, substantial degradation of stiffness and strength is permissible. The only requirement is that sufficient strength be preserved so that the structure can support gravity loads under the maximum induced deformations.

6.5.2 Limit States

Figure 6-9 shows the general force-deformation behavior used for primary members in the structure, i.e. columns. The slope from point A to B is typically a percentage of the linear slope (e.g. 10%) and is meant to account for strain hardening. Point C represents the ultimate strength of the component. Any additional deformation results in a strength degradation, as from point C to D. Beyond point D, the component behaves with significantly reduced strength, and has zero strength beyond point E.

For an individual member, three primary limit states are defined in FEMA 356, with applicable deformations as indicated in Figure 6-9.

- Immediate Occupancy (IO): The structure remains safe to use. Any repairs are minor.

- Life Safety (LS): Structure remains stable and has significant reserve capacity. Hazardous nonstructural damage is controlled.

- Collapse Prevention (CP): The structure remains standing, but only barely. Any other damage or loss is acceptable.

As the structure is incrementally pushed over in the pushover analysis, the response is monitored for the life-safety and collapse prevention limit states. The structure is assumed to reach its overall "Life-Safety" capacity when the first column hinge reaches the "Life-Safety" limit state. The structure is assumed to reach its overall "Collapse Prevention" limit state when the first hinge reaches its "Collapse Prevention" limit states.

In these analyses, the braces are allowed to completely fail, as long as the structure in its overall collapse prevention limit state can be demonstrated to have the capability to preserve its ability to support gravity loads.

6.5.3 Target Displacements

In the pushover analyses, target displacements are calculated for the Maximum Considered Earthquake (MCE) and the design level earthquake (2/3 MCE). The target displacement represents the maximum displacement likely to be experienced during each level event. The calculation of target displacement is an iterative process that incorporates the stiffness degradation, increase in natural period, increase in damping, and change in demand that occurs as the amount of nonlinear behavior increases within the structural framing elements and connections.

6.5.4 Nonlinear Analysis Results

Figures 6-10 to 6-17 and Tables 6-6 to 6-9 show the pushover analysis results for loads from each of the four primary lateral directions, and with vertical accelerations down (1.2D) or up (0.9D).

Each of the results show similar characteristics, that the initial deformation is linear. As the compression braces begin to buckle, and load is redistributed, the

structure capacity shows the “saw tooth” changes, with a slight drop in capacity. As the structure then begins to act as a moment frame, the behavior becomes linear again, but with a reduced slope, indicating the reduced stiffness and higher period. The slope continues to be reduced as additional yielding takes place and the period shifts again.

For all directions and vertical load combinations, the structure was shown to have adequate deformation capacity for the 2/3 MCE and MCE demand levels. Each figure shows the deformation at the life-safety and collapse prevention limit states, as well as the calculated demands associated with each earthquake level.

Note that for the pushover analyses from the North, the capacities were demonstrated to exceed the demands. However, the collapse prevention limit state was not reached during the analysis due to numerical limitations in the SAP analyses, and the inability to reach a stable solution at those displacements. The limits shown graphically are implied.

The critical direction was determined to be the pushover from the North, with a downwards vertical force. As a final check, that direction was run with an incremental load of 100% from the north, coincidental with 30% from the west. Both directions were incremented, while the vertical load was kept constant.

Figures 6-18 and 6-19 and Table 6-10 show the results from that analysis. The structure had not reached its “Life-Safety” or “Collapse Prevention” limit states at the deformations of 4.1 inches and 6.1 inches for the 2/3 MCE and MCE, respectively. The maximum deformation obtained by SAP in the analysis was at 6.2 inches for the 1.2D vertical load case (Figure 6-18). The dashed line shows graphically the inferred deformation capacity. Examination of the results at the final deformation in the analysis showed that the diagonal compression braces in the orthogonal direction were beginning to buckle, initiating numerical instabilities and rapid stiffness changes that the computer program SAP was unable to accommodate. We believe that this is the reason for the lack of numerical convergence of the analysis and inability to push the structure further, and does not reflect any structural instability.

Figure 6-19 shows the same analysis for the upwards vertical load (0.9D). For this analysis, convergence was achieved for the entire analysis, beyond the collapse prevention limit state deformation of 7.5 inches.

Figure 6-20 indicates the hinge formation for the “Life Safety” limit state deformation of 6.1 inches for the loading from the North with downward vertical (1.2D), as shown in Figure 6-10. The color coding at the bottom of the figure indicates the location on the force-deformation curve of Figure 6-9, with the light blue color at the base of one column under the Precipitator stack from for Boiler No. 2 showing the location of the first hinge to reach the Life Safety limit. At this deformation shown, the structure deformation has already exceeded the target demand for life-safety, which was calculated to be 3.8 inches due to the design level earthquake (2/3 MCE), and the target demand for collapse prevention, which was calculated to be 5.6 inches at the MCE.

6.6 CONCLUSIONS AND RECOMMENDATIONS

The CO Boiler and spray tower support structures were evaluated using linear analysis methods and the CalARP seismic criteria. Based on the results of that analysis, little or no structural damage is expected in the design level event, and no actions for structural mitigation are recommended.

The electrostatic precipitator and stack support frame was evaluated using both linear and nonlinear analysis methods. Based on the results of those analyses, we expect that the structure might experience nonlinear behavior in the design level event, including buckling of braces and minor nonlinear response of columns. However, the structure maintains significant reserve capacity and is not at risk of collapse. In the Maximum Considered Earthquake (roughly 2,500 year return period), the structure may have extensive damage, but is able to support gravity loads following the event without collapse.

We believe that these analyses reasonably demonstrate that the structures achieve the Basic Safety Objectives of FEMA 356 and the intent of the CalARP Seismic Guidance document, and no retrofits to the structural framing are required.

Table 6-1: Q Values for Linear Analysis

Item	Classification in Table 1 of CalARP Guidelines	Ductility-Based Reduction Factor (Q)
Primary Structural Steel Framing – Boiler and Electrostatic Precipitator Structure	Steel Braced Frame	5
Lowest Bay of Columns – Electrostatic Precipitator Structure	Steel Braced Frame with Soft Story	2
Foundations	Spread Footings	6

Table 6-2: Member Performance Summary for Linear Analysis – Boiler and Spray Tower Support Structure

Structural Element	Member Type	Axial DCR	Major Bending DCR	Minor Bending DCR	Total DCR
Worst Case Column	BUILT-UP W12 W/ C15 & 1"PL	0.320	0.536	0.092	**0.948** **Adequate**
Worst Case Brace	W8X17	0.814	0.017	0.107	**0.939** **Adequate**

Table 6-3: Member Performance Summary for Linear Analysis – Precipitator and Stack Support Structure

Structural Element	Member Type	Axial DCR	Major Bending DCR	Minor Bending DCR	Total DCR
Worst Case Column (Bottom Bay)	W14X74-FP	0.651	0.803	0.297	**1.751** **Not Adequate**
Worst Case Brace	WT6X22.5	1.342	1.224	0.000	**2.565** **Not Adequate**

Table 6-4: Maximum Absolute Displacements at Bellow Locations

Location	Earthquake Direction	N-S Transverse Displacement (inches)	E-W Longitudinal Displacement (inches)
CO Boiler	100%X 30%Y 30%Z	1.02	0.47
CO Boiler	30%X 100%Y 30%Z	0.37	1.34
Precipitator	100%X 30%Y 30%Z	2.81	0.67
Precipitator	30%X 100%Y 30%Z	1.25	1.41

Table 6-5: Maximum Relative Displacements at Bellow Locations

Location	Earthquake Direction	N-S Transverse Displacement (inches)	E-W Longitudinal Displacement (inches)
CO Boiler / Precipitator	100%X 30%Y 30%Z	2.99	0.82
CO Boiler / Precipitator	30%X 100%Y 30%Z	1.30	1.95

Table 6-6: Global Performance Summary – Pushover Analysis
Precipitator and Stack Support Structure – N Direction

FEMA 356 Limit State	Probabilistic Ground Motion	Initial/ Elastic Period (seconds)	Target Displacement (inches)	Limit Displacement (inches)	Base Shear (kips)	Meets Performance Criteria ?
Life Safety	⅔ MCE	1.01	3.8	6.1	438	**Yes**
Collapse Prevention	MCE	1.01	5.6	> 6.5	638	**Yes**

Values presented are for governing load combination of (E + 1.2D)

Table 6-7: Global Performance Summary – Pushover Analysis
Precipitator and Stack Support Structure – S Direction

FEMA 356 Limit State	Probabilistic Ground Motion	Initial/ Elastic Period (seconds)	Target Displacement (inches)	Limit Displacement (inches)	Base Shear (kips)	Meets Performance Criteria ?
Life Safety	⅔ MCE	1.01	3.8	8.6	514	**Yes**
Collapse Prevention	MCE	1.01	5.6	9.3	755	**Yes**

Values presented are for governing load combination of (E + 1.2D)

Table 6-8: Global Performance Summary – Pushover Analysis
Precipitator and Stack Support Structure – E Direction

FEMA 356 Limit State	Probabilistic Ground Motion	Initial/ Elastic Period (seconds)	Target Displacement (inches)	Limit Displacement (inches)	Base Shear (kips)	Meets Performance Criteria ?
Life Safety	⅔ MCE	0.76	3.0	8.9	545	**Yes**
Collapse Prevention	MCE	0.76	4.5	10.3	778	**Yes**

Values presented are for governing load combination of (E + 1.2D)

Table 6-9: Global Performance Summary – Pushover Analysis
Precipitator and Stack Support Structure – W Direction

FEMA 356 Limit State	Probabilistic Ground Motion	Initial/ Elastic Period (seconds)	Target Displacement (inches)	Limit Displacement (inches)	Base Shear (kips)	Meets Performance Criteria ?
Life Safety	⅔ MCE	0.76	3.0	8.7	482	**Yes**
Collapse Prevention	MCE	0.76	4.5	10.1	723	**Yes**

Values presented are for governing load combination of (E + 1.2D)

Table 6-10: Global Performance Summary – Pushover Analysis
Precipitator and Stack Support Structure – 100%N - 30%W Direction

FEMA 356 Limit State	Probabilistic Ground Motion	Initial/ Elastic Period (seconds)	Target Displacement (inches)	Limit Displacement (inches)	Base Shear (kips)	Meets Performance Criteria ?
Life Safety	⅔ MCE	1.04	4.1	> 6.2	494	**Yes**
Collapse Prevention	MCE	1.04	6.1	> 6.2	708	**Yes**

Values presented are for governing load combination of (E + 1.2D)

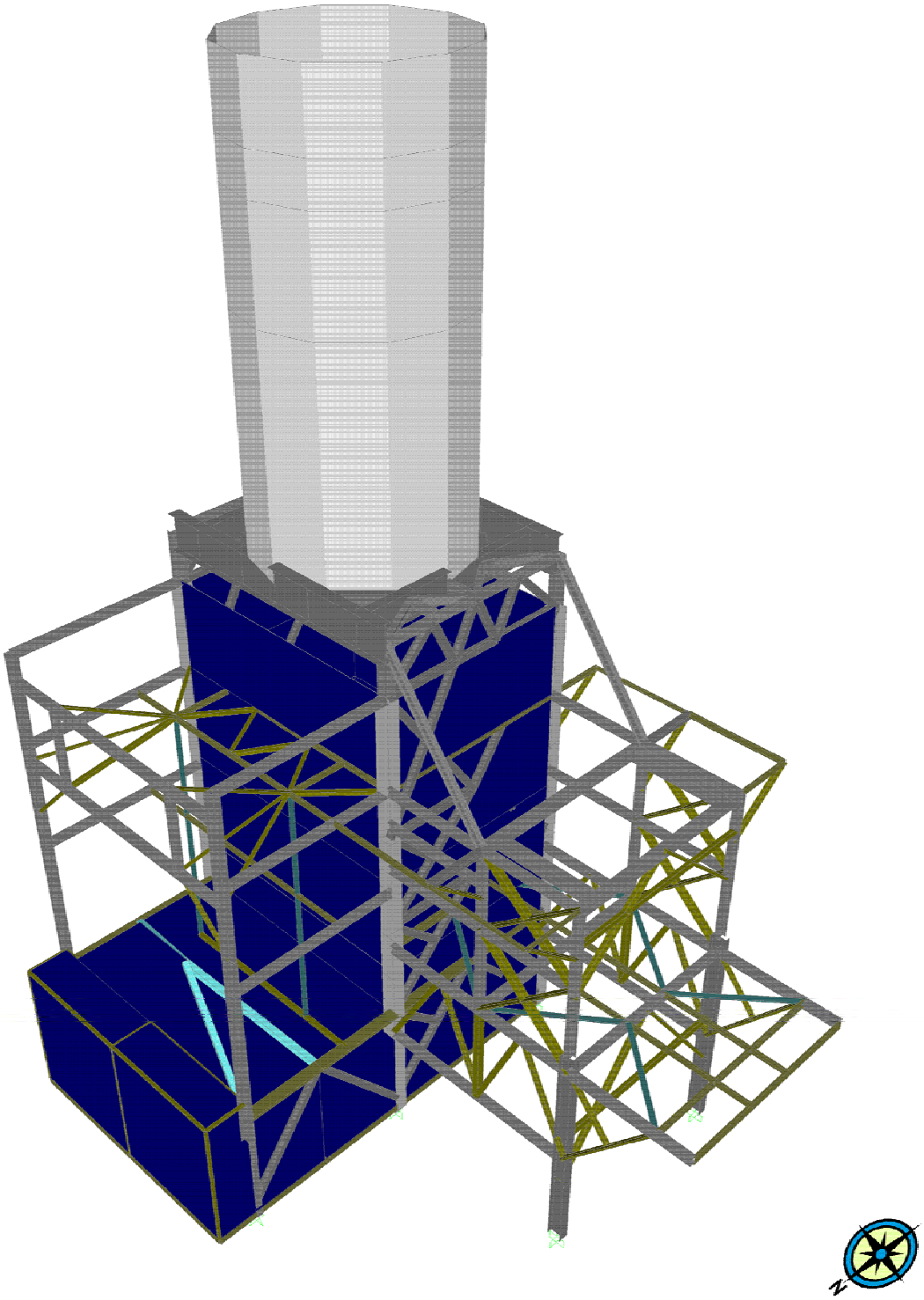


Figure 6-1: SAP Model of CO Boiler and Spray Tower