Enclosure 2

BALANCE OF PLANT STRUCTURES
SEISMIC REEVALUATION PROGRAM

SEISMIC REEVALUATION AND UPGRADE OF
TURBINE BUILDING COMPLEX

San Onofre Nuclear Generating Station
Unit 1

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1.0 Introduction

This report provides a description of the various activities associated with the seismic reevaluation and modification of the San Onofre Unit 1 turbine building complex which consists of the turbine pedestal and the four individual structural systems that surround it.

The program scope and the criteria for the seismic reevaluation of the turbine building are described in Reference 1. Activities related to the seismic reevaluation of the turbine building that are both completed and ongoing are discussed in this report. The following specific areas are discussed: (1) the design activities associated with proposed modifications, (2) the seismic reevaluation of the structural components comprising the turbine building complex, and (3) a summary of the preliminary stress analysis results for the various portions of the turbine building complex.

2.0 Description of Structure

The turbine building consists of four individual structural systems which surround the concrete turbine pedestal. These four structural systems are known as the turbine building north and south extensions and the east and west heater platforms.

The turbine building north extension is a one-story structural steel frame building with a mezzanine. It has approximate plan dimensions of 40 feet by 50 feet with an 8-1/2 inch thick prestressed concrete slab at elevation 42 feet, 0 inch, and a steel grating platform at elevation 30 feet, 0 inch. One and one half inch wide expansion joints are provided at the juncture between the extension and the turbine generator pedestal (at elevation 42 feet, 0 inch).

The turbine building south extension is a one-story building employing a steel frame system constructed above ground level. The south extension has approximate plan dimensions of 40 feet by 50 feet, with an 8-1/2 inch thick prestressed concrete slab at elevation 42 feet, 0 inches. One and one-half inch wide expansion joints are provided at the junction of the south extension and the turbine generator pedestal (at elevation 42 feet, 0 inches).

One story steel frame heater platforms are on the east and west sides of the turbine building above ground level. Each platform has approximate plan dimensions of 112 feet by 50 feet and supports an 8-1/2 inch thick prestressed concrete slab at elevation 35 feet, 6 inches.

The concrete turbine pedestal is a reinforced concrete space frame supported by a 5-foot thick mat foundation. It consists of haunched columns at the four corners of the mat foundation and three haunched intermediate walls. The north columns are 8 feet square; the south columns are 8 feet by 8 feet, 2 inches. Wall thickness varies from 4-1/2 feet to 7 feet. The centerline to centerline distances between columns are 34 feet, 0 inches in the east-west direction and 125 feet, 6 inches in the north-south direction. The operating deck consists of an 8-foot thick center section that supports the turbine-generator, which is accommodated through several large openings in the deck. Two cantilevered wings on the east and west sides are 1-foot, 6 inches thick. The top of the deck is at Elevation 42 feet, while the overall height of the structure is approximately 33 feet, 6 inches from the top of the mat foundation.

3.0 Design of Modifications

As discussed in Reference 2, interim modifications have been installed to the north turbine building extension. These modifications were made to ensure that the column to beam connections have the capability to resist the full plastic moment which could be developed by the connected members. Reference 2 also provided an assessment of the withstand capability of the turbine building with these installed modifications.

As discussed in Reference 3, we are proceeding with the installation of seismic bracing for the north, and west turbine building extensions. Concurrently, we are completing the reevaluation of the turbine building complex.

The four structural steel platforms peripheral to the turbine pedestal will be converted from moment resisting frames to moment resisting braced frames by the addition of the seismic bracing. These modifications are expected to reduce the stresses in these structures and preclude potential interactions with adjacent structures by limiting deflections. Foundation modifications are also being designed to accommodate the increased uplift loads due to the new bracing.

The design activities associated with the proposed structural modifications for the north extension and the west heater platform are presently in progress and are on schedule to support the installation of these modifications to the north extension and west heater platform during the outage scheduled to begin about March 1, 1982. As of January 31, 1982, the design of the steel bracing members and their respective connections was 80 percent complete. At present the foundation modification design activities are approximately 60 percent complete.

The associated design activities include the rerouting of various piping and instrumentation lines, instruments, control panels, electrical junction boxes, conduits and raceways in order to install the bracing members. The design of this relocation work is approximately 80-90 percent complete, and the physical reloaction of some of these items is in progress.

4.0 Methods of Analysis

The turbine building complex was analyzed utilizing a three-dimensional finite element model. This model includes all four extensions of the turbine building and the turbine pedestal, gantry crane and a lumped mass representation of the fuel pool. The model also incorporates the previously mentioned modifications to the four platform structures peripheral to the turbine pedestal. These modifications consist primarily of a lateral bracing scheme for each of the four steel structures (east and west heater platforms and the north and south extensions), as well as the modifications to several of the existing column footings.

The finite element model utilized various element types to represent the different parts of the building. The structural steel framing was represented as an assemblage of beam elements and the concrete deck was modeled with plate elements connected to the steel framing. The masonry walls at the periphery of the extensions, although not load bearing walls, are bolted at the deck level to the steel framing with ties. These masonry walls were represented by a grillage of beams having equivalent linear elastic properties, based on an independent nonlinear analysis of the walls. This representation assured that the out of plane reaction of the wall due to inertia forces was properly transmitted to the steel framing.

Soil-structure interaction effects were considered in the analysis, by representing the soil medium by equivalent springs. Horizontal translational, vertical translational and rotational springs were attached to the individual column bases (nodal points) at the foundation. The stiffness properties of the springs were determined based on the elastic half space theory using strain dependent soil properties. Because of its thickness, the basemat of the turbine pedestal was assumed rigid and a single set of springs was attached to the center of the mat. The connections of the basemat to the columns and walls of the pedestal and to the turbine building columns founded on the basemat were represented by rigid links.

The time history analysis method was used to calculate in-structure response spectra which were used in subsystem evaluations. The response spectrum method was used to calculate forces, moments and stress resultants in the dynamic structural model. Both analysis techniques utilized eigenvalues and eigenvectors that were calculated by the subspace iteration method. For the response spectrum analysis modal responses were combined in accordance with the provisions of USNRC Regulatory Guide 1.92. In the time history analysis modal responses are combined directly by adding the computed responses at each time step. These analysis techniques are described in detail in Reference 4.

The eigenvalue analysis of the dynamic finite element model has been completed. The analysis computed the first 140 modes. The principal mode shapes were plotted and checked to assure the model's dynamic response was consistent with expected behavior for the structural system. A response spectrum analysis has been performed for each of the three principal directions and the resulting stresses were combined utilizing the square root of the sum of the squares (SRSS) technique. A time-history analysis was also performed to compute the in-structure response spectra at the various points of interest. All of the above analyses were performed separately for the two governing conditions: (1) the gantry crane located on the south extension and (2) the gantry crane located on the north extension.

5.0 Structural Evaluation

The stress evaluation of the various structural members comprising the turbine building complex is presently in progress. The governing stress conditions for all of the modified components within the north extension and the west heater platform have been identified in order to finalize the design of modification to these structures. The structural evaluation of the remaining in-situ components is in progress. The following paragraphs describe the preliminary results of the stress analysis of the various elements of the turbine building complex.

5.1 Concrete Pedestal

The turbine pedestal is a massive reinforced concrete structure that was originally designated Seismic Category A. Preliminary calculations indicate that this structure is capable of withstanding loadings due to a 2/3g Housner response spectrum. The detailed stress analysis is currently in progress and it is anticipated that these calculations will be completed in mid-March.

5.2 Structural Steel Framing

The modifications to the north and west portions of the turbine building are currently being designed and will be installed during the outage beginning March 1, 1982. These modifications will limit the deflection of the structures sufficiently to prevent interaction with the adjacent fuel storage building and control and administration building. The detailed calculations for these structures were 80% complete as of January 31, 1982. The anticipated completion date for these calculations is March 1, 1982. The evaluation of the south and east platforms has begun and will be completed in mid-March. Preliminary calculations indicated that the proposed modifications to the turbine building structural steel framing will ensure that the calculated stresses in the structural steel framing are within the limits of the BOPSSR criteria.

5.3 Masonry Walls

Reinforced masonry walls are located on portions of the outer perimeter of both the north and south extensions and both the west and east heater platforms. In order to evaluate these walls an analytical model was developed that takes into account nonlinear inelastic behavior. The model was developed by Computech Engineering Services, Inc. and is based upon current test data for similar walls. A complete account of the model's development and analysis results can be found in References 5, 6 and 7. The analysis of this model directly addresses the walls integrity under the out-of-plane loading that would be encountered during the design basis earthquake. In addition, information useful in evaluating the integrity of the wall under multi-component seismic loading and the integrity of connections between the wall and the structural steel framing is provided by the nonlinear analysis.

The results of the nonlinear out-of-plane analysis described in Reference 7 show that all the walls respond within the criteria presented in Reference 5. The maximum out-of-plane deflection was calculated to be approximately 10". The actual deflection of this wall will probably be substantially less due to a number of conservatisms in the analysis. Among these conservatisms is the assumed weight of attached equipment. In addition, the scaled real time histories utilized envelope the Housner spectra by as much as 75%. Therefore, it is expected that the maximum deflections actually experienced during a Design Basis Earthquake would be on the order of 4" to 6".

Since these walls are non-bearing and non-shear walls, the applied bearing and shear loads are due only to the walls self weight and the weight of attached equipment. Based on the results of preliminary analyses, all these walls meet the criteria for in-plane and combined component loading as specified in Reference 5.

The connection of the masonry walls to the structural steel framing is such that only those loads normal to the wall are resisted. Therefore, in-plane loads are not transferred to or from the structural steel framing. The nonlinear out-of-plane analysis provided reaction loads at the top support in order to evaluate the double pin connection between the masonry wall and the structural steel frame. The supports were evaluated against working stress allowables in order to ensure the ability of the walls to respond inelastically. As determined by preliminary analysis, the most critical single connection is stressed 22% over working stress allowables. Modifications to ensure these support stresses remain within working stress limits for the west heater platform will be designed and installed during the March 1, 1982 plant outage.

6.0 Conclusions

The seismic reevaluation of the turbine building complex will be completed by March 31, 1982. Modifications to the north turbine extension, the west heater platform, and the associated masonry walls will be completed during the outage scheduled to begin March 1, 1982. Preliminary calculations indicate that the turbine pedestal will satisfy the requirements of the BOPSSR criteria. Implementation of modifications to the east heater platform and the south turbine extension will be evaluated following completion of the analysis of these structures.

7.0 References

1. Letter from K. P. Baskin to D. M. Crutchfield dated February 23, 1981.

2. Letter from K. P. Baskin to D. M. Crutchfield dated September 28, 1981.

3. Letter from K. P. Baskin to D. M. Crutchfield dated November 3, 1981.

4. Hadjian, A. H., et. al., BC TOP 4A, Revision 3, Seismic Analyses of Structures and Equipment for Nuclear Power Plants, Bechtel Power Corporation, 1974.

5. Seismic Evaluation of Reinforced Concrete Masonry Walls, Volume 1, Criteria, Computech Engineering Services, Inc., Berkeley, California.

6. Seismic Evaluation of Reinforced Concrete Masonry Walls, Volume 2, Inelastic Analysis Methodology, Computech Engineering Services, Inc., Berkeley, California.

7. Seismic Evaluation of Reinforced Concrete Masonry Walls, Volume 3, Masonry Wall Evaluation, Computed Engineering Services, Inc., Berkeley, California.