STRUCTURAL EVALUATIONS FOR REACTOR BUILDING CRANE UPGRADE

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ABSTRACT

During the process of upgrading the reactor building crane to "single failure proof" status it was discovered that analyzing the existing bridge girder with application of tension-field method could avoid costly modifications. Also, modeling the entire supporting structure and crane girders together, which is more realistic anyway, instead of modeling and analyzing them separately avoided costly modifications at the column bases. The total saving is estimated to be $1.0M and all regulatory requirements and codes and standards were met.

INTRODUCTION

The DOE is not ready for accepting spent nuclear fuels from operating nuclear power plants within the next decade, although originally it was supposed to have done so from 1995. Therefore, all plants planning decommissioning and/or life extension have to remove and store spent fuel in dry storage casks on site, since the spent fuel pools were not built for the capacities required. These casks must be handled per NUREG 0612; i.e., either the cranes must be upgraded to "single-failure-proof" per NUREG 0554 or, a load drop analysis must be performed to show that no safety related equipment is damaged by the failed load. At the James A. FitzPatrick Nuclear Power Plant (JAF), it was decided that upgrading the Reactor Building Crane to "single-failure-proof" i.e., failure of any single component would not result in a total failure of the load handling system, can achieve the objective. Please note that the rated capacity of the crane , 125 Tons will remain the same.

A preliminary evaluation showed that the entire trolley had to be replaced. The replacement trolley was designed by EDERER Inc, Seattle, Washington; with its XSAM (extra safety and monitoring) feature. EDERER submitted a generic Topical Report to the USNRC and received Safety Evaluation Report from the NRC in 1983. The SER concluded that the XSAM trolley meets the NUREG 0554 requirements in general; but site-specific analyses must be performed on remainder of the crane system e.g., the bridge girders and the building column or support structure etc, and must be upgraded, if required.

The original crane was procured in early 1970s and was designed per EOCI-61 and USAS B.30.2-1967. In response to NUREG-0612, it was evaluated to the codes applicable at the time e.g., CMAA-70 and ANSI B30.2, 1976 and was found to meet the requirements of both. However, the procurement specification for the Reactor Building Crane Upgrade project required that " Design, Analysis, Fabrication, Inspection, Testing, Quality Assurance and Handling shall be performed in accordance with requirements of ANSI/ASME NOG-1 (latest revision) " [1], with some minor exceptions. SECTION NOG-4000: Requirements for Structural Components, provides detailed procedure for analysis of the crane. For example:

Loads:

Dead Loads (trolley and bridge load); Live Loads (rated load, critical load, construction load, credible critical load); Impact Load (vertical, transverse horizontal and longitudinal horizontal); Wind Load (operating wind, design wind and tornado wind); Normal Plant Operating Loads (plant operation induced load and static test pressure loads) and Seismic and Abnormal Events Loads (Design Basis or Safe Shutdown Earthquake, Operating Basis Earthquake and Abnormal events like plant equipment failure loads) are the loads to be considered.

Load Combinations:

Please refer to NOG-4140: LOAD COMBINATIONS, for the prescribed load combinations to be considered.

Seismic Analysis:

NOG-1 requires a generalized three dimensional mathematical modeling of the crane with finite elements and response spectrum or time-history analysis for the seismic inputs applied at crane rail from the plant data. Location and number of dynamic degrees of freedom, boundary conditions at trolley and the crane rails, damping values to be used and the decoupling criteria for the crane rails are very well defined for the analysts. Please refer to NOG-4150 thru 4154 for the details.

INITIAL ANALYSIS

The initial structural analysis was performed by NORPAC Engineering for the EDERER Corp [2].

The crane trolley runs on a two-girder bridge at 21'0" apart, with a span of 116'11". The girders are attached by flexible end-ties at the ends. The bridge rails are supported on runway beams (bracket-type beam seat) attached to the top of W36X280 building columns. The crane members were modeled as 3-dimensional beam elements with the hoist rope modeled as a very flexible beam with proper cross sectional area and modulus of elasticity. Modeling and member releases are done in accordance with NOG.

The response spectra at JAF consist of 0.5% damping OBE and 1% damping DBE, with peaks broadened +/- 15% to account for inaccuracies. Since NOG allows use of 4% damping for OBE and 7% for DBE, the response spectra were converted to these damping values using the formula:

$$A_n = A_{0.01} * (0.01/n)^{1/2}$$

Where, A_n is the modified peak response; $A_{0.01}$ is the peak response at 1% damping and n is the appropriate damping value.

Also, response spectra were available for column base el. of 369'0" and top of the columns at el. 425'0", whereas the crane rails are supported at el. 402'0". A linear interpolation was used to determine the responses at el. 402'0". It was observed that for both horizontal and vertical directions, the 4% damped OBE spectra values exceeded the 7% damped DBE values and therefore the OBE was used for analysis.

Crane runway was included in the crane analysis model, restrained for all six degrees of freedom by springs located at each of the building columns. NOG provides coupling/decoupling criteria for crane analysis.

The building structure which supports the crane was modeled including columns, end walls, side walls, roof truss and other roof framing members for the walls and the roof. Runway beams were not included in the model since they were included in the crane model.

Mathematical model was developed in conformance with NOG. In Figure-1 below just the Trolley Model is shown for illustration.

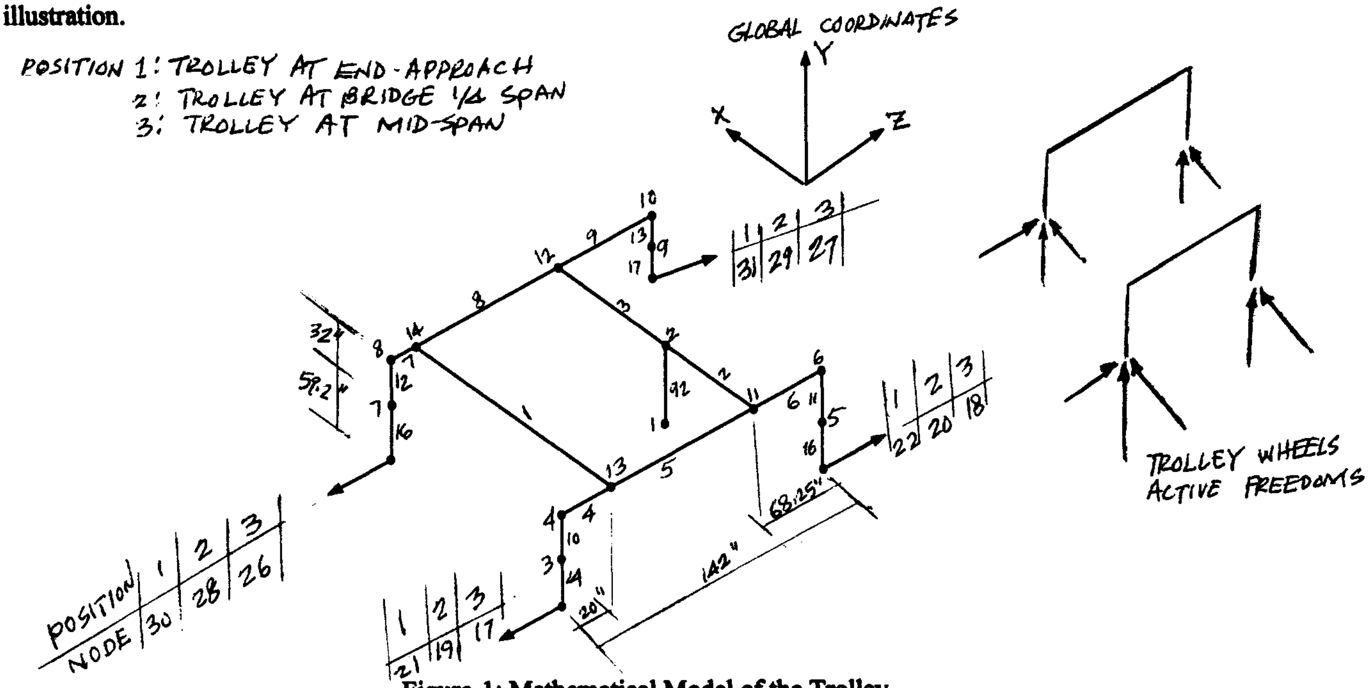

Figure-1: Mathematical Model of the Trolley

Loading Conditions:

Four bridge positions were used for analysis; e.g.,

- bridge at end approach
- bridge with a set of wheel trucks mid-span between the two end columns
- bridge on top of intermediate columns
- bridge on center span of the building

These locations were chosen for maximizing the runway and building column loading and also for maximum crane response.

Also, three trolley positions were considered for each of the bridge locations described above; e.g.,

- trolley at end approach
- trolley with the midpoint between the heavily loaded wheels and the trolley load girt centered at girder ¼ point
- trolley with the midpoint between the heavily loaded wheels and the trolley load girt centered at girder mid-span

High hook and low hook positions were considered for each loading condition. All requirements are in accordance with NOG. The pendulum effect was determined to be negligible. The computer program used was IMAGES-3D Version 2.0 by Robert L. Cloud and Associates.

Mathematical Analysis:

NOG allows mathematical analysis using either response spectrum or time history methodology. The response spectrum analysis for a lumped mass, multiple-degrees-of-freedom system was carried out. The basic equation of motion used was:

$$([K] - (w^2/g)[W]) \{X\} = \{0\}$$

Where:

$[K]$ = stiffness matrix; g= acceleration due to gravity; w^2= eigenvalue ; $[W]$= weight matrix; $\{X\}$=eigenvector (mode shape) and $\{0\}$= null vector

Weights were from available data of the crane and new trolley. Natural frequencies, mode shapes and participation factors were calculated from modal analysis. The NRC 10% rule for closely spaced modes was applied. Once the frequencies and mode shapes were determined, seismic analysis was performed. Three-dimensional input response spectra applied and generalized displacements and accelerations were calculated. From the generalized results, the individual and combined modal displacements and accelerations were calculated. The three components of earthquake motion were combined by square-root-of-the sum-of squares (SRSS) method.

Results of Analysis:

The analysis showed that the maximum response was with the hook at high position, the bridge at the end approach and the trolley at mid-span. Similar results also found parallel to the runway. Member stresses were obtained by combing the static load case with the seismic stresses for accelerations in all three directions. The following recommendations were made from the results of the initial analysis:

- The end ties joining the bridge girders needed to be redesigned due to severe overstress. It required a pinned connection in the horizontal direction for the girders to deflect relatively independently but must be able to carry the moment about the vertical axis. This design change was performed since the top-of-the-rails elevations may vary somewhat.
- The box bridge girders (108" X 42" and span of 116'-11") were overstressed and required stiffeners on flange and the web. Flange plates (2 plates 1"thickX6" wide) had to be extended 25'0" on either side from the center span to reach an allowable stress limit per AISC (see Figure-2). This recommendation was not accepted, since "tension-field-action" was not considered.
- Although the stresses in the columns were acceptable, the column anchor bolts stresses exceeded the AISC allowable limits when the crane was located over the column. This recommendation needed reevaluation, since the required modification was cost prohibitive and cumbersome.
- The bolted joints between the runway and the columns and also between the bridge truck and the girder were overloaded and the existing A-325 bolts needed to be replaced with A-490 bolts. This was done.

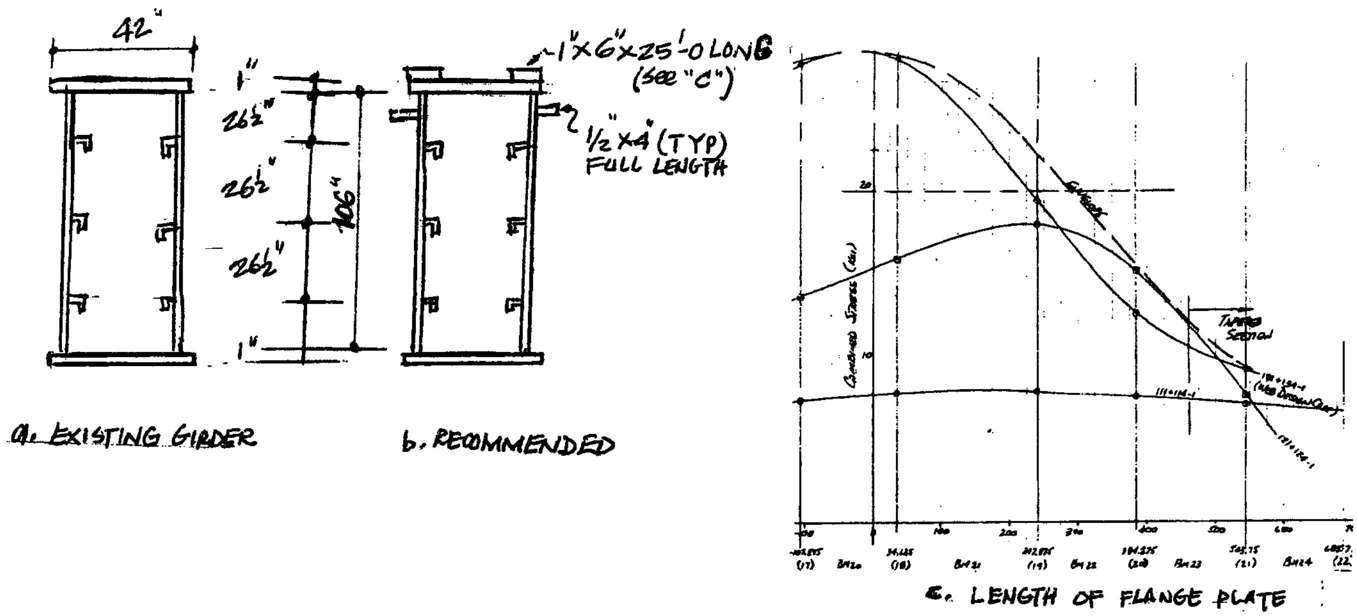

Figure 2: Recommended Bridge Girder Modification

REANALYSIS:

1.Reanalysis of the Bridge girders

As noted above, the bridge girders were reanalyzed applying tension field action. From AISC Ninth Edition, [3], " Unlike columns, which actually are on the verge of collapse as their buckling stage is approached, the panels of the plate girder web, bounded on all sides by the girder flanges or transverse stiffeners, are capable of carrying loads far in excess of their "web buckling" load." The stiffeners must be able to act as compression struts. This is analogous to a panel of a Pratt truss; i.e., a diagonal strip of the web acts as a tension member while the transverse stiffeners act as compression members. The maximum bending stress calculated was 27.3 ksi (=1.33X F_b'; the factor 1.33 is applicable since seismic loading is considered) and the maximum web shear stress was 13.1 ksi.

This analysis avoided the girder stiffening described under Initial Analysis and saved approximately $400,000.00.

2. Reanalysis of the Building Structure:

The apparent cause of the overstress in anchor bolt was the modeling consideration that columns are fixed at the base. Due to the unavailability of original design calculations this was the initial assumption, which resulted in anchor bolt overstress. However the installation of the anchor bolts on extended base plate was even further difficult task.

The steel columns at the refuel floor are on the edge of the concrete floor and to install more anchor bolts would require base plate extension in one direction only – into the inner side of the floor. Thus base plates would be eccentric and column loads must be transferred to the base plate by using brackets. The other problem was locating the anchor bolts to miss rebars. The cost estimate ran to approximately $500,000.00. Thus further analysis was necessary. ANATECH Consulting Engineers performed the reanalysis [4]. The program used was ABAQUS.

- Unlike the initial analysis, crane and building were coupled in a single mathematical model
- Beam elements were used for the three dimensional finite element model for response spectrum analysis
- Crane and trolley positions considered are the same as previous analysis
- Column base plates with two bolts were considered pinned in both lateral directions. Base plates with four bolts were modeled with rotational stiffness based on base plate analysis performed earlier.
- Runway girders were connected to the building column with appropriate offset and such that all three translations are transmitted trough the joint.
- The final model had 5349 degrees of freedom and used 853 modes, which span the frequency range from 0.8 to 45.38. The mass participation was 90% or greater of the total mass in all three directions.
- Dynamic results use SRSS for combining the results from three directions of earthquake loading.
- Dynamic values are combined with static values using addition of absolute values.

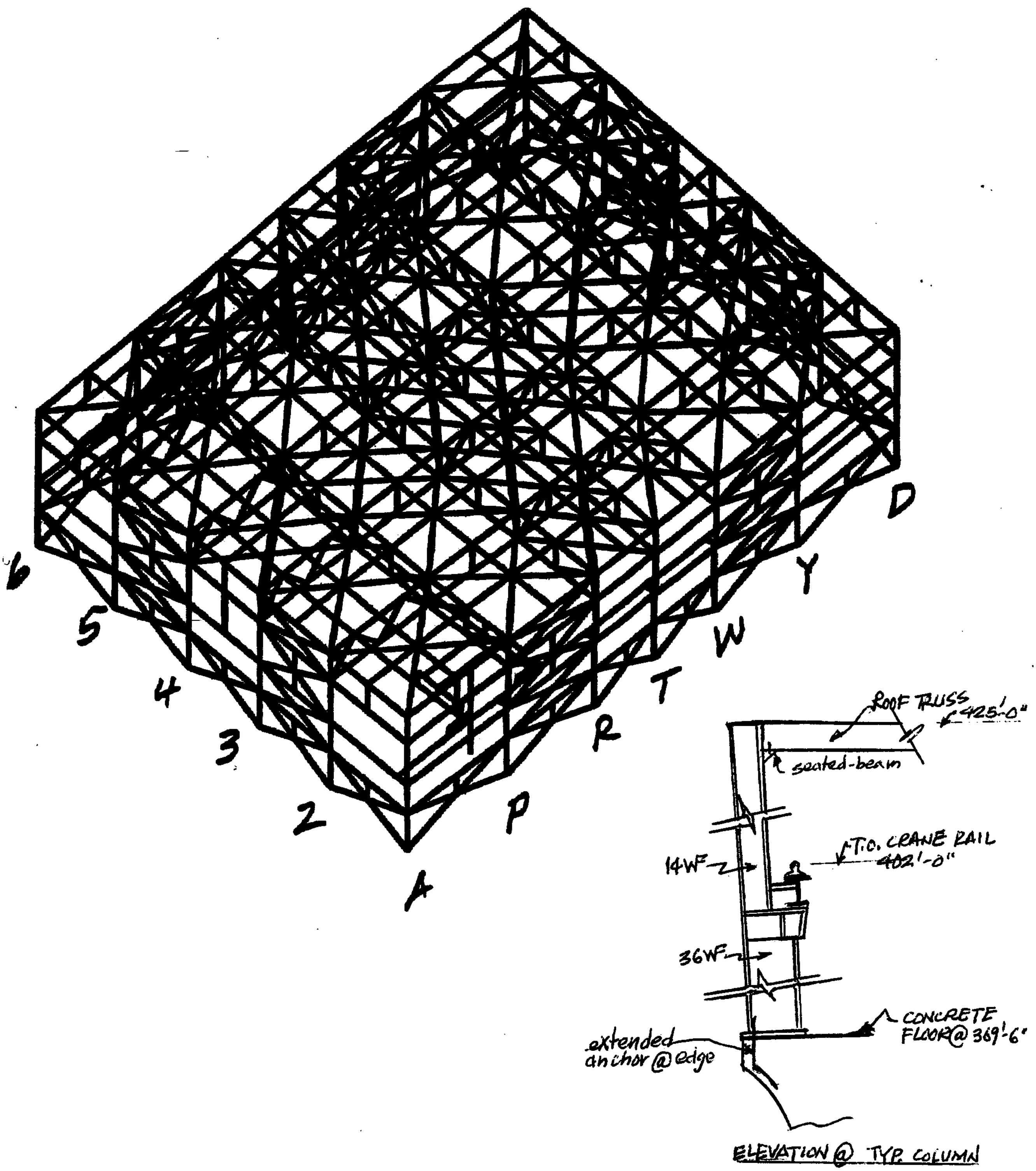

Figure 3: Mathematical Model of the Coupled Building and Crane
(Trolley at P; 258K Load; Beam elements only)

Results of the Reanalysis:

Results of the response spectrum analysis for member loads and joint forces were used for AISC design checks. The following are the results of this reanalysis effort:

- Column base plates and anchor bolts are adequate for the imposed loading. The stresses were well within allowables per AISC codes.
- All steel members including the Reactor Building roof truss and the upgraded crane are qualified
- The bolted connection of the lower chord of the roof truss to the column is overstressed at the column line where the crane is positioned when the seismic event occurs. This is clearly a localized condition, which could result in a localized plastic deformation without affecting the structural integrity of the building.

CONCLUSIONS:

- Code compliance and meeting the regulatory requirements are very important at an operating nuclear power plant; however application current analytical methodology within the bounds of these must be carefully evaluated prior to recommending any modifications. The bridge girder stiffening was not required.
- Safety and dose exposure to installation crew also must be considered for modification at operating plants.
- Engineers employed at operating nuclear power plants must keep up-to-date with the profession so that they can adequately evaluate the works done for them by outside A/Es and consultants. Those not attached to an operating plant are completely unaware of the modification requirements and do sometimes recommend the design change that appears best in the analysis; e.g., the crane company recommended replacement of 5/8" ASTM -A325 bolts with 7/8" since that worked. However this would have required lead paint removal, drilling bigger holes and replacing them, all work requiring high scaffolds and working in the reactor building . So we asked for a reevaluation with A-490 bolts and that worked.
- Practical decision making is very essential under some circumstances e.g., coincidence of cask handling (a two week period in two years time), concurrent occurrence of an earthquake equivalent to the plant design basis earthquake (with a probability of 1 in 50 to hundred years) and the crane located directly over a column during that period (most earthquakes are of 15 second or less duration) is practically of extremely low probability. Calculated approximate probability $<10^{-6}$ and so not credible per NRC. However, there will be an open item under the cask handling project to check the roof truss bottom chord bolts if there is an earthquake during the process of lifting or lowering the casks.

REFERENCES:

[1] ASME NOG-1-1995: RULES FOR Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder

[2] Connacher, N., NORPAC Engineering Inc, Seattle, Washington; New York Power Authority, James A. FitzPatrick NPP, Reactor Building Crane Seismic Analysis, January 2000.

[3] AISC (American Institute of Steel Construction): Manual of Steel Construction, Ninth Edition, October 1994

[4] ANATECH Consulting Engineers, San Diego, CA: Seismic Qualification of Reactor Building for Overhead Crane Trolley Upgrade, March 2000