NMC
Committed to Nuclear Excellence

Palisades Nuclear Plant
Operated by Nuclear Management Company, LLC

May 14, 2004

10 CFR 50.90

U. S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555-0001

Palisades Nuclear Power Plant
Docket 50-255
License No. DPR-20

Request for Additional Information on License Amendment Request: Palisades' Spent Fuel Pool Crane Upgrade

By letter dated January 29, 2004, Nuclear Management Company, LLC (NMC) submitted a license amendment request on the spent fuel pool crane (L-3 crane) for the Palisades Nuclear Plant. By letter dated April 28, 2004, the Nuclear Regulatory Commission issued a request for additional (RAI) information concerning the license amendment request on the spent fuel pool crane. Enclosure 1 provides the NMC responses to the RAI questions.

Consistent with the January 29, 2004, request, NMC is proposing the following new License Condition 2.C.(6):

(6) NMC is authorized to operate the spent fuel pool crane (L-3 crane) main hoist to the rated capacity of 110 tons and within the single-failure-proof design.

Summary of Commitments

This letter contains no new commitments and no revisions to existing commitments.

I declare under penalty of perjury that the foregoing is true and correct. Executed on May 14, 2004.

Daniel J. Malone
Site Vice President, Palisades Nuclear Plant
Nuclear Management Company, LLC

Enclosure (1)
Attachments (2)

CC Administrator, Region III, USNRC
Project Manager, Palisades, USNRC
Resident Inspector, Palisades, USNRC

A001

ENCLOSURE 1
REQUEST FOR ADDITIONAL INFORMATION ON SPENT FUEL POOL CRANE LICENSE AMENDMENT REQUEST FOR PALISADES NUCLEAR PLANT

Introduction

The Nuclear Regulatory Commission staff is reviewing the proposed license amendment request to Facility Operating License No. DPR-20, relating modifications to the spent fuel pool crane modifications dated January 29, 2004, and has found further clarification is needed as follows:

Nuclear Regulatory Commission (NRC) Requested Information

1. *Provide a more detailed summary of the structural analysis for the upgrade of the spent fuel pool crane.*

Nuclear Management Company, LLC (NMC) Response

1. NMC is providing detailed summaries of the structural analyses which were performed for the upgrade of the spent fuel pool crane as follows:

Seismic Analysis

Engineering Analysis (EA), EA-FC-976-01, "Seismic Input for Fuel Pool Crane Up-Grade," was created to generate the seismic response spectra of the bridge girder with the trolley and lifted loads in various locations and elevations. Three-dimensional analysis models were constructed to represent steel framing and crane, including trolley, bridge girders, end trucks, runway girders, supporting steel columns up to roof trusses, the roof trusses, and vertical and horizontal bracings. The models are consistent with recommendations of American Society of Mechanical Engineers (ASME) NOG-1, "Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder)."

Forty-five different models have been constructed using the same basic model to represent five different locations for the bridge on the rail girders in the north-south direction. For each of these bridge locations, three different locations of the trolley on the bridge are considered. Finally, for each trolley and bridge location, three conditions of the lifted load on the main hoist are analyzed. One of the conditions is with no lifted load on the hoist. The other two conditions are with the rated load of 110 tons on the main hoist at the up and down positions.

The horizontal input response spectrum was obtained from Appendix C of Standard Specification C-175(Q), "Requirements for Seismic Evaluation of Electrical and Mechanical Equipment at Palisades Nuclear Power Plant," for elevation 649'. The vertical input spectrum was from Appendix B of C-175(Q) for ground motion.

Damping values for framing modeled, and for equipment for which spectra are provided, are two-percent critical damping in accordance with C-175(Q) and Final Safety Analysis Report (FSAR) Table 5.7-2.

Maximum responses in the three seismic excitation directions are combined using the square-root-of-sum-of-squares (SRSS) rule consistent with C-175(Q), which states the use of Regulatory Guide (RG) 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis."

The mass of the lifted load of 220 kips was included. The lifted load in the model is supported from the trolley by a beam element, whose element stiffness properties were calculated using pendulum frequency depending on the location of the lifted load (up or down). A confirmatory solution is provided to show that the horizontal component of the amplified lifted load (pendulum) force is smaller than the transferred shear obtained in the small-displacement solution process used.

Design forces and spectra were generated for operating basis earthquake (OBE) only. The safe shutdown earthquake (SSE) forces and spectra are obtained by doubling the corresponding OBE information in accordance with C-175(Q).

The spectrums, in the three directions from the three hoist conditions, were enveloped and 15% widened. These spectrum are for North-South direction, East-West direction and Vertical Direction. The plots of the widened response spectra for the three directions are shown in Attachment 1. These response spectra are used as inputs to other analyses, as described below.

Auxiliary Building Steel Framing Evaluation

EA-FC-976-07, "Assessment of Auxiliary Building Framing Above Elevation 649'-0" for Increased Crane Loads," qualifies the building steel from below the rails and rail clips all the way to the building column anchors.

This EA used the results of EA-FC-976-03, "Bridge Wheel Loads Calculation," as input bridge wheel reaction loads for the analysis, which includes crane impact loads. For static load cases, both Crane Manufacturers Association of America (CMAA) Specification No. 70, "Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Crane," and ASME NOG-1 load combinations were considered. Impact forces were calculated in accordance with the respective standard or specification, as appropriate. Seismic and impact loads were not combined in any loading direction, which is consistent with ASME NOG-1, Section NOG-4140, "Load Combinations."

Three-dimensional models were constructed to represent the steel framing and runway girders, and include the steel columns, the roof trusses, and vertical and horizontal bracing. The input loads include seven static cases and the response spectrum analysis results from EA-FC-976-01, "Seismic Input Calculation." The input seismic loads for the models are horizontal spectra for elevation 649' of the auxiliary building and vertical ground spectrum. In total, four different models are used. These models cover four different locations of the bridge girder wheels on the runway girders to obtain the maximum design forces/moments at the runway girders and framing structure.

The assessment includes dead loads (including crane), roof live loads, and crane live load of 220 kips in combination with seismic OBE, seismic SSE or wind load. Fourteen load combinations (LC) were used.

Load Combinations

LC1	=	$1.0D + 1.0L + 1.0CN_W$
LC2	=	$1.0D + 1.0L + 1.0CN_E$
LC3	=	$1.0D + 1.0L + 1.0\ CN_W + 1.0W_W$
LC4	=	$1.0D + 1.0L + 1.0\ CN_E + 1.0W_E$
LC5	=	$1.0D + 0.5L + 1.0CS + E$
LC6	=	$1.0D + 0.5L + 1.0CS - E$
LC7	=	$1.0D + 0.5L + 1.0CS + E'$
LC8	=	$1.0D + 0.5L + 1.0CS - E'$
LC9	=	$0.72D + 1.0CN_W + 1.0W_W$
LC10	=	$0.72D + 1.0CN_E + 1.0W_E$
LC11	=	$0.72D + 1.0CS + E$
LC12	=	$0.72D + 1.0CS - E$
LC13	=	$0.9D + 1.0CS + E'$
LC14	=	$0.9D + 1.0CS - E'$

Where,

D	=	Dead loads of structures
L	=	Roof live loads
Ww	=	Wind load from the West
We	=	Wind load from the East
CNw	=	Crane wheel loads from the normal operation including lifted load with impact and with lateral loads in the east direction
Cne	=	Crane Wheel loads from the normal operation including lifted load with impact and with lateral loads in the west direction
CS	=	Crane static wheel loads under seismic condition. This includes lifted load, but does not consider impact.
E	=	OBE seismic loads resulting from a ground surface acceleration of 0.1g
E'	=	SSE seismic loads resulting from a ground surface acceleration of 0.2 g

Allowable Stresses

For normal loads, allowable stresses for steel structural members are based on the Manual of Steel Construction by American Institute of Steel Construction (AISC) allowables.

For load combinations with dead load, OBE seismic load and wind loads, equations 2 and 3 of FSAR Section 5.9.1.1.2 use a load factor of 1.25 for all three loads. It permits allowable stresses up to 0.9Fy. Therefore, the stress increase factor for SSE load combinations is calculated as
0.9/(0.6 x 1.25) = 1.2.

For load combinations with SSE seismic loads, equations 4 and 5 of FSAR Section 5.9.1.1.2 use a 1.0 load factor on dead load and SSE load. It permits allowable stresses up to 0.9Fy. Therefore, the stress increase factor for SSE load combinations is calculated as 0.9/(0.6 x 1.0) = 1.5.

For load combinations involving dead load plus roof loads, plus crane and crane live load with wind loading, the AISC allowable stresses are increased by 1.33. This factor is based on Palisades FSAR Section 5.9.1.1.1.

The coefficient of 0.72 for dead load in LC9 through LC12 is derived from equation 3 of FSAR Section 5.9.1.1.2, which specifies a load factor of 1.25 for dead load when combined with wind load. Note (a) to this equation also specifies using a load factor of 0.9 instead of 1.25 to cover cases when addition of dead load reduces the critical stress level. Since LC3 and LC4 are using unity for dead load, the reduced dead load factor becomes 0.9/1.25 = 0.72 for LC9 and LC10.

Summary of Results

Table 6.1, "Summary of Maximum IC's for Members and Connections and Recommendations," of EA-FC-976-07, provides a summary of the maximum interaction coefficients. This table is provided as Attachment 2. The analysis resulted in the following modifications and limitations for ground level wind speeds as follows:

1. Some of the vertical bracing in the north-south direction were required to be modified.

2. The structural steel framing was able to be qualified for ground level wind speeds up to 90 miles per hour (mph) only. FSAR Section 5.3.1.1 requires Class 1 structures to be designed to withstand the external pressure resulting from a sustained wind velocity of 100 mph.

The vertical bracing in north-south direction of the steel framing has been modified. The heavy load handling procedure FHS-M-23, "Movement of Heavy Loads in the Spent Fuel Pool Area," was revised to specify that the maximum allowable outside wind speed is 90 mph while lifting a heavy load. These modifications and restrictions were necessary because the original configuration for the 110-ton lift did not meet all the allowable stresses. Subsequent analyses confirmed that the allowable stresses that were originally not met would now be met with the modifications and restrictions in place.

Auxiliary Building Concrete Evaluation

EA-FC-976-08, "*Qualification of Concrete Structure Supporting the Increased Crane Loads in the Fuel Handling Area*," uses the column base reactions from EA-FC-976-07 as input to qualify the concrete below elevation 649'. This calculation verifies the adequacy of the concrete structures for the increase crane loading in the fuel handling area, which includes seismic loads.

All of the concrete elements were qualified based on the methodology and acceptance criteria of ACI 318-63, "Building Code Requirements for Reinforced Concrete," in accordance with FSAR Table 5.2-2.

The load factors are applied to dead, live, crane operation, seismic and wind loads in accordance with FSAR Section 5.9.1.1.2.

Load Combinations

$Y_1 = 1/\Phi\ (1.25D + 1.25E)$

$Y_2 = 1/\Phi\ (1.25D + 1.25W)$

$Y_3 = 1/\Phi\ (0.9D + 1.25W)$

$Y_4 = 1/\Phi\ (1.0D + 1.0E')$

Where:

Y = required yield strength of the structures.
D = Dead load of structure and equipment plus any other permanent loads contributing to stress. A portion of live load is added when such load is expected to be present when the plant is operating
E = OBE loads resulting from a ground surface acceleration of 0.1g
E' = SSE loads resulting from a ground surface acceleration of 0.2g

W = Wind loads resulting from a 90 mph wind
(Note: FSAR Section 5.3.1.1 requires Class 1 structures to be designed to withstand the external pressure resulting from a sustained wind velocity of 100 mph. However, EA-FC-976-07 used 90 mph for the design basis wind speed.
Φ = 0.70 for tied reinforced concrete in compression
Φ = 0.90 for reinforced concrete in flexure

The auxiliary building structure supports other loads including floor dead and live loads, fuel pool loads and seismic excitation of the substructure. The concrete elements evaluated were checked for the effects of the local loads applied due to the crane upgrade. The structure as a whole is a robust, deep concrete beam and shear wall structure. The effect of the crane upgrade, which increases the lifted load by 10 tons with corresponding changes in crane hardware dead load, is small in terms of total overall building loads. As such, the adequacy of the existing lateral load resisting system is not significantly challenged by the addition of these loads. Therefore, the evaluation of the effect on the individual columns is appropriate by engineering judgment to verify the adequacy of the structure for the change in loads.

Summary of Results

Columns principally subjected to axial loads,

Maximum Axial Load = 525 kips

Allowable Axial Load = 4953 kips

Maximum Interaction Coefficient (IC) = 525/4953 = 0.11 < 1.0

Columns subjected to axial compression and biaxial bending moments,

Axial Load = 421.1 kips

Moment Load = 1601.4 ft-kips

Allowable Capacity of Cross Section for Axial Load only = 4518.1 kips

Allowable Bending Capacity of Section for Moment only = 3093.3 ft-kips

Spent Fuel Pool Walls

Shear Evaluation

Maximum Concrete Shear Stress = 13.73 psi

Allowable Shear Stress in Unreinforced Concrete = 93.11 psi

Interaction Coefficient for Shear = 13.73/93.11 = 0.15 < 1.0

Axial Evaluation

Maximum Concrete Compressive Stress = 203.8 psi

Allowable Distributed Compressive Stress = 1278.1 psi

Interaction Coefficient for Axial = 203.8/1278.1 = 0.16 < 1.0

NRC Requested Information

2. *Provide the design and licensing basis of the 25-ton auxiliary hook of the spent fuel pool crane. The answer should include the physical as well as the procedural controls over the auxiliary hook.*

NMC Response

2. NMC is providing the design and licensing basis of the 25-ton auxiliary hook of the spent fuel pool (SFP) crane (L-3 Crane), including the physical and procedural controls over the auxiliary hook.

25-Ton Drop – Auxiliary Hook

The L-3 crane auxiliary hook has a rated capacity of 15-tons and is not single-failure-proof. Original load drop structural analyses done by Bechtel were performed using the methodology described in Bechtel Topical Report BC-TOP-9, "Design of Structures for Missile Impact," Revision 1, dated

July 1973, and Revision 2, dated September 1974. 25-ton load drop analyses were performed for four specific locations in the SFP area (Bechtel Report for 25-Ton Cask, "Evaluation of Postulated Cask Drop Accidents," dated August 1974). The four specific locations evaluated were the cask storage area in the SFP, track alley at elevation 625', cask wash down pit (CWP) and to the southwest of the hatch at elevation 649'. The 25-ton drop analyses resulted in no damage to the integrity of the SFP for a drop in the cask storage area or to the integrity of the CWP. The drop in track alley resulted in no significant damage. Based on these analyses, impact limiting pads are not required for drops from the SFP crane auxiliary hoist.

The radiological consequences of a postulated cask drop from the auxiliary hoist are bounded by the postulated drop of a 96-ton transfer cask in the SFP, which is described in FSAR Section 14.11.3.

Palisades' heavy load procedure, FHS-M-23 is used to control all aspects of heavy load handling in and around the SFP. The L-3 crane has two interlock limit switches, which do not allow either the main hook or auxiliary hook to travel over spent fuel in the pool. One of these interlocks is bypassed only during dry fuel storage operations, which allows load handling only in the cask loading area of the SFP. The other interlock limit switch may only be momentarily bypassed in the event that the crane hook drifts into the area over the main SFP to allow movement of the crane back into the cask loading area.

NRC Requested Information

3. *Describe the lift and movement of the new spent fuel cask with the upgraded spent fuel pool crane.*

NMC Response

In the original license amendment request, NMC requested Nuclear Regulatory Commission (NRC) approval to update the FSAR to reflect the L-3 crane main hoist upgrade to the new rated capacity (110 tons) and to reflect the new single-failure-proof design. The response below is being provided as information only and is not intended to limit the scope of the amendment.

In a teleconference with members of the NRC staff, NMC described various stages of the transfer cask loading process. The response below provides clarification to some of the aspects of the process that were discussed.

3. Lifts of the dry shielded canister and transfer cask are made within the existing Palisades heavy loads requirements using the Palisades heavy loads procedure, FHS-M-23. The cask loading sequence, starting with the placement of an empty dry shield canister and transfer cask in the CWP, is described below for the maximum heavy load lifts.

Transport the Transfer Cask/Dry Shield Canister Filled with Water from CWP to SFP

The transfer cask/dry shield canister, filled with water, is raised from the CWP floor (elevation ~634') to the auxiliary building floor (elevation ~649') and moved directly west above the SFP and lowered onto the cask stand (elevation ~608'), located in the northeast corner of the SFP. The lift of approximately 86 tons is made using the transfer cask trunnions and the lifting yoke. Transnuclear, Inc., supplies the transfer cask, lifting yoke and the four cable assemblies used as rigging for the shield plug. The transfer cask/dry shield canister is ungrappled from the lifting yoke while in the SFP.

Spent fuel is transferred from the SFP storage rack to the dry shield canister using the Palisades loading procedure, FHSO-17A, "MSB/DSC Loading Procedure," and the spent fuel handling machine.

Lower Shield Plug onto the Dry Shield Canister

The shield plug is lowered onto the dry shield canister in the SFP following fuel loading. The load of approximately four tons is lowered using the lifting yoke and four cable assemblies.

Transport the Loaded Transfer Cask / Dry Shield Canister from the SFP to CWP

The loaded transfer cask/dry shield canister and the shield plug are raised from the cask stand (elevation ~608') in the SFP to the auxiliary building floor (elevation ~649'), moved directly east to the CWP, and then lowered onto the CWP floor (elevation ~634'). The lift of approximately 108 tons is made using the transfer cask trunnions and the lifting yoke. The transfer cask/dry shield canister is ungrappled from the lifting yoke while in the CWP.

Transport the Transfer Cask with the Loaded and Sealed Dry Shield Canister to Track Alley Hatch

Following loading and sealing operations, the drained and dried transfer cask, with a loaded and sealed dry shield canister, is lifted to the auxiliary building floor (elevation ~649') and moved to the track alley hatch opening. The load of approximately 107 tons is raised and moved using the transfer cask trunnions and the lifting yoke.

Lower the Transfer Cask with the Loaded and Sealed Dry Shield Canister onto Transfer Trailer

The transfer cask, with a loaded and sealed dry shield canister, is lowered from the track alley hatch opening (elevation ~649') onto the transfer trailer located in track alley (elevation ~625'). The lift of approximately 107 tons is lowered and downloaded from the vertical position to the horizontal position using the transfer cask trunnions and the lifting yoke.

Heavy Load Movements and Rigging

The heavy load movements are made using the L-3 single failure proof crane main hook and the approved load paths described in procedure FHS-M-23.

FHS-M-23 specifies that the rigging required for single failure proof lifts comply with NUREG-0612, "Control of Heavy Loads at Nuclear Power Plants." The transfer cask is designed and fabricated to meet NUREG-0612 and ANSI N14.6, "Special Lifting Devices for Shipping Containers Weighing 10,000 Pounds (4500 kg) or More," requirements. The lifting yoke complies with the design and licensing requirements in NUREG-0612 and ANSI N 14.6. The four cable assemblies used with the lifting yoke to lower the shield plug comply with the requirements in NUREG-0612. Impact limiting pads are not used for lifts made with the single failure proof crane.

Evaluation for Cask Overturning and Suspended Loads

The potential for overturning an ungrappled, loaded transfer cask during a seismic event while in the SFP building was evaluated in preparation for use of the NUHOMS® system at the Palisades Nuclear Plant. The evaluation included scenarios for the placement of the transfer cask on a transfer cask stand in the SFP, on a transfer cask stand in the CWP, and directly on the CWP floor. The evaluation concluded that the transfer cask will not overturn in any of these placement scenarios.

While the transfer cask is suspended on the L-3 main hook, vertical seismic inertial loads are not high enough to overcome gravity. The lateral seismic loads generated are accounted for in the design of the crane mechanical, structural, and rail systems. Therefore, transfer cask seismic restraints are not required while the transfer cask is suspended from the crane.

In conclusion, a transfer cask seismic restraint is not required for the scenarios evaluated inside the auxiliary building in the SFP area.

ATTACHMENT 1

EA-FC-976-01, CHARTS, "WIDENED RESPONSE SPECTRA FROM FARSI ANALYSIS"

3 Pages Follow

CALC NO. EA-FC-976-01

Page 115

SAFETY RELATED

SARGENT & LUNDY ENGINEERS

PROJECT 11000-022 REV 0

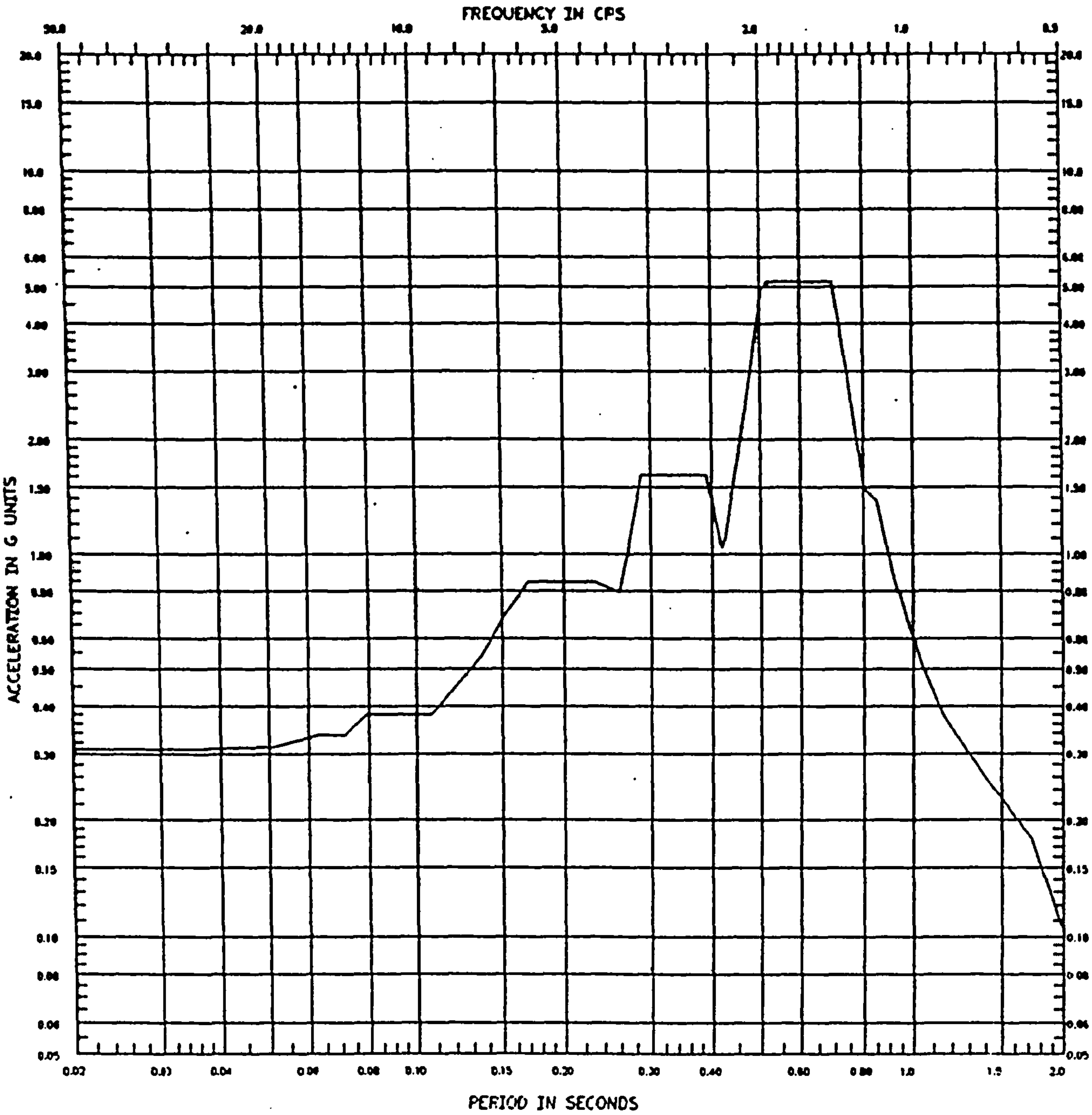
09/26/00

PEAKS WIDENED ON EACH SIDE BY 15 %

WIDENED RESPONSE SPECTRUM FROM FARSI ANALYSIS

DAMPING 0.02

15% WIDENED OBE RS FOR TROLLEY



SPECTRA NO. OBE

NODE

ELEVATION

1 DIRECTION NS ANGLE

LOCATION TROLLEY

CALC NO. EA-FC-976-01 Page 116

SAFETY RELATED

SARGENT & LUNDY ENGINEERS

PROJECT 11000-022 REV 0

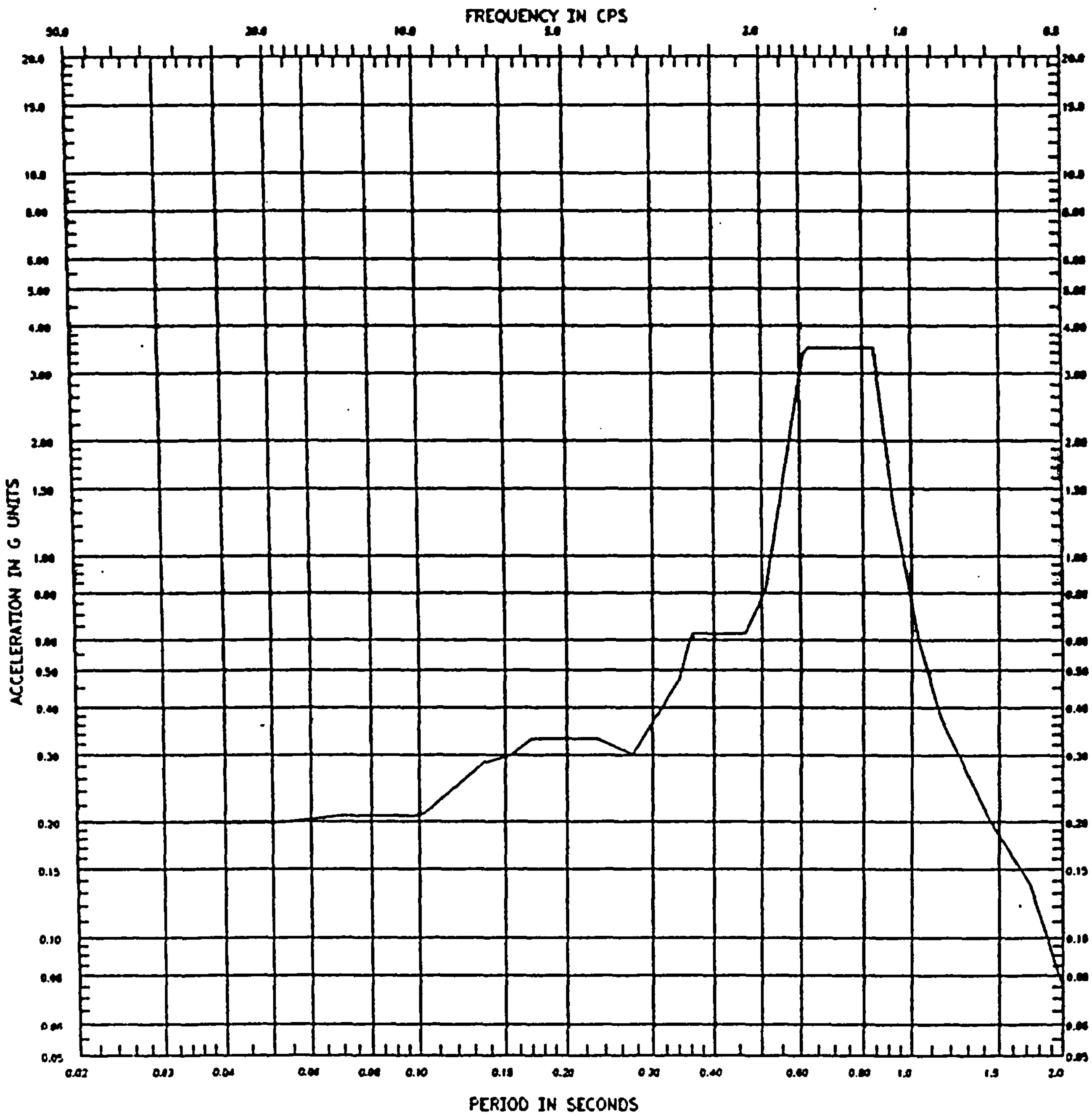
09/26/00

PEAKS WIDENED ON EACH SIDE BY 15 %

WIDENED RESPONSE SPECTRUM FROM FARSI ANALYSIS

DAMPING 0.02

15% WIDENED OBE RS FOR TROLLEY



SPECTRA NO. OBE

NODE

ELEVATION

2 DIRECTION EW ANGLE

LOCATION TROLLEY

CALC NO. EA-FC-976-01 page 117

SAFETY RELATED

SARGENT & LUNDY ENGINEERS

PROJECT 11000-022 REV 0

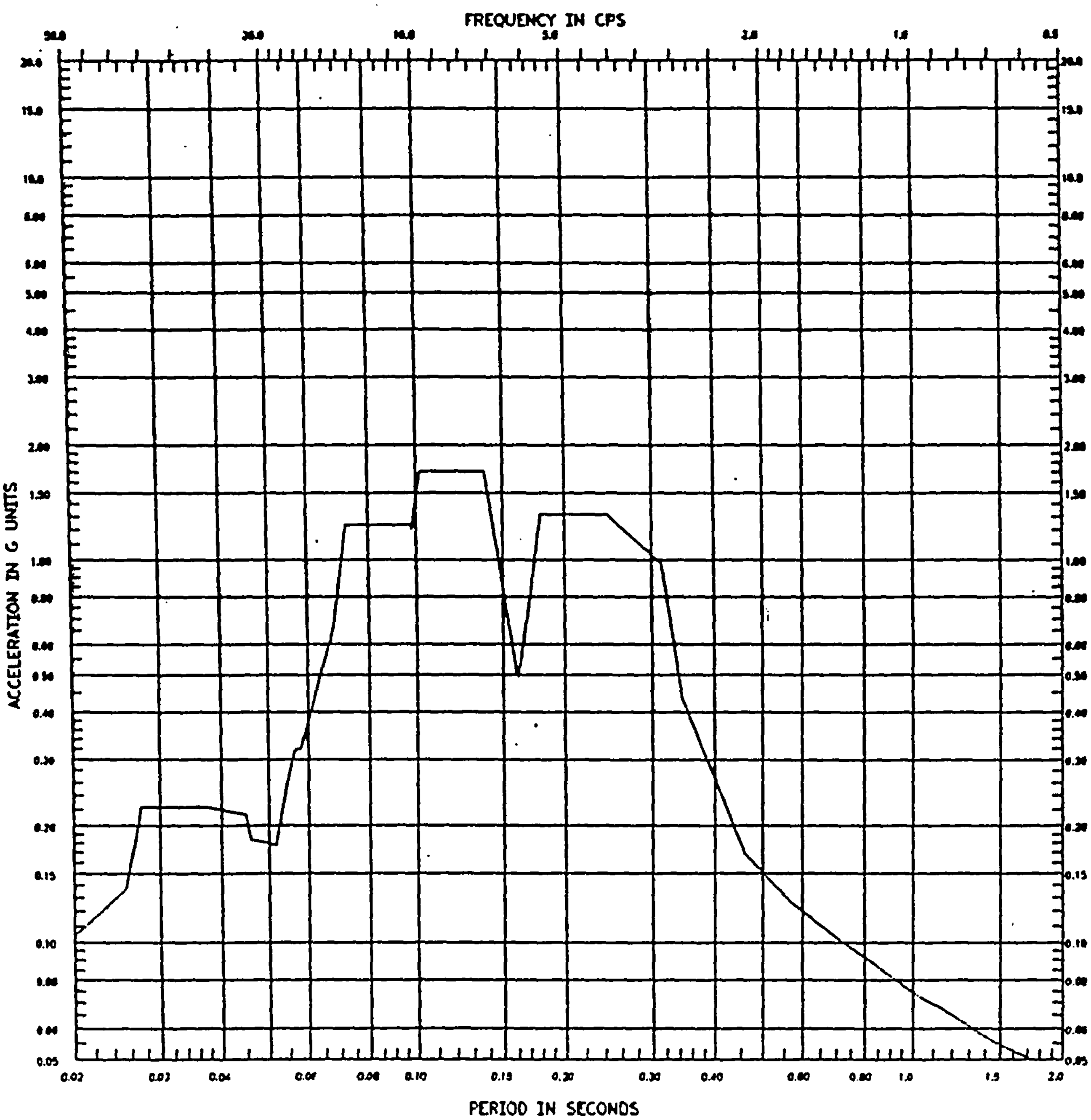
09/26/00

WIDENED RESPONSE SPECTRUM FROM FARSI ANALYSIS

15% WIDENED OBE RS FOR TROLLEY

PEAKS WIDENED ON EACH SIDE BY 15 %

DAMPING 0.02



SPECTRA NO. OBE

NODE

ELEVATION

3 DIRECTION VT ANGLE LOCATION TROLLEY

ATTACHMENT 2

EA-FC-976-07, TABLE 6.1, "SUMMARY OF MAXIMUM [INTERACTION COEFFICIENT'S] IC'S FOR MEMBERS AND CONNECTIONS AND RECOMMENDATIONS"

1 Page Follows

Table 6-1 Summary of maximum ICs for Members and Connections and Recommendations
(Member ICs are for axial plus bending stress)

Item	Member/ Conne	Description	ICmax	Section in EA	Observation/ Recommendation
1	Member	Crane Columns WF 33x130 at Column Line	0.913	6.3.2.5	O.K.
2	Member	Roof Columns WF 12x99 at Column Lines 2	0.772	6.3.2.5	O.K.
3	Member	Crane Columns WF 33x220 at Column Lines 27.5, 26, 22.5, and 19.5			
		(a) Seismic load combination	0.955	6.4.2	O.K.
		(b) Wind load combination at 100 mph grou	1.311	6.3.2.5	N.G. See Item 3 (c)
		(c) Wind load combination at 90 mph ground	0.965	6.7.1	O.K.
4	Member	Roof Columns WF 12x133 at Column Lines	0.852	6.3.2.5	O.K.
5	Member	Crane Girders: WF 36x194 + WF 18x96	0.986	6.4.1	O.K.
6	Member	Vertical bracing in north-south direction at Column Rows F0.1 and F0.9			
		(a) Between elevations 652-9" to 675'-7.5"	2.111	6.3.2.5	Needs modification; see Item 6(b)
		(b) Same as (a) with modification	0.641	6.5.2	Assumes modification per Note (1)
		(c) Below elevation 652'-9"	0.741	6.5.2	O.K.
		(d) Above elevation 675'-7.5"	0.977	6.5.2	O.K.
7	Member	Roof truss members	0.98	6.3.2.5	O.K.
8	Connection	Base plate and embedment of WF33x220 at Column Lines 27.5, 26, 22.5, and 19.5			
		(a) Seismic load combination	0.677	6.6	O.K.
		(b) Wind load combination at 90 mph ground	0.933	6.7.2	O.K.
9	Connection	Lateral connection of runway girders to roof	0.911	6.5.3	O.K.
10	Connection	Connections of vertical bracing in north-south direction at Column Rows F0.1 and F0.9			
		(a) Between elevations 652-9" to 675'-7.5"	1.423	6.5.2	Needs modification; see Item 6(b)
		(b) Same as (a) with modification	0.545	6.5.2	Assumes modification per Note (1)
		(c) Below elevation 652'-9"	0.858	6.5.2	O.K.
		(d) Above elevation 675'-7.5"	0.584	6.5.2	O.K.
13	Connection	Connections of roof truss			
		(a) Vertical members to chords	0.783	6.5.1	O.K.
		(b) Diagonal members to chords	0.372	6.5.1	O.K.
		(c) Chords to roof columns	0.371	6.5.1	O.K.

Notes for Table 6-1:

(1) The existing 6x4x1/4 angles shall be doubled (or add L6x4x5/16 if 1/4" not available) in the indicated elevation zone. This will double of the four-bolt connections, and it will adequately increase the member buckling strength. Drawings C-121 and C-122 need to be modified to document this requirement.