Dominion Energy Kewaunee, Inc.
5000 Dominion Boulevard, Glen Allen, VA 23060

Dominion®

November 9, 2007

U. S. Nuclear Regulatory Commission
Attention: Document Control Desk
Washington, DC 20555

Serial No. 07-0465
KPS/LIC/BG: R0
Docket No. 50-305
License No. DPR-43

DOMINION ENERGY KEWAUNEE, INC.
KEWAUNEE POWER STATION
LICENSE AMENDMENT REQUEST 234 – REQUEST FOR REVIEW AND APPROVAL OF METHODOLOGY CHANGE REGARDING AUXILIARY BUILDING CRANE UPGRADE

Pursuant to 10 CFR 50.90 Dominion Energy Kewaunee, Inc. (DEK) requests Nuclear Regulatory Commission (NRC) approval of a proposed license amendment request (LAR) for the Kewaunee Power Station (KPS). The non-single-failure-proof auxiliary building crane is being upgraded to a single-failure-proof design through replacement of the crane trolley and modification of the existing crane bridge. The proposed amendment would allow the use of a methodology for performing the seismic qualification analysis of the auxiliary building crane. The methodology described in this proposed amendment is not currently described in the KPS Updated Safety Analysis Report (USAR) or the code of reference applicable to the crane.

Specifically, this LAR requests NRC approval of a seismic analysis method to be used for the auxiliary building crane. The new methodology includes rolling of the crane bridge and trolley wheels during a seismic event. This analysis will be used in determining the design loads for the new crane trolley and bridge, and the loads imparted from the crane bridge to the auxiliary building structure. DEK has reviewed the consideration of rolling, which occurs when the seismic load exceeds the crane holding brake torque, against the criteria contained in 10 CFR 50.59. Our review determined this change constitutes a departure from a method of evaluation as described in the KPS USAR. Because this particular analysis technique is not described in the USAR or in applicable NRC-endorsed guidance, DEK has determined NRC approval is required before use at KPS. This amendment request has been determined not to involve a significant hazards consideration because use of this seismic analysis methodology does not affect the operation or design functions of the crane as currently described in the KPS USAR.

Additional information and documents to support this license amendment request are provided as attachments to this letter. Attachment 1 provides a detailed description of the proposed change, background and technical analysis, no significant hazards consideration determination, and environmental review consideration. Attachment 2 provides marked-up USAR pages showing proposed changes. Attachment 3 provides a reference paper for the analysis technique, which is considered not publicly available. Also enclosed are non-proprietary general arrangement drawings of the upgraded KPS Auxiliary Building crane bridge and trolley.

A001
NRR

The KPS Plant Operation Review Committee has approved the proposed amendment and a copy of this submittal has been provided to the State of Wisconsin in accordance with 10 CFR 50.91(b).

DEK requests an expedited approval of the proposed amendment by May 30, 2008 in order to support the dry spent fuel storage loading schedule. DEK requests an expedited review because KPS will lose full core reserve (FCR) after the spring 2008 outage. To recover FCR, DEK has constructed an ISFSI using the NUHOMS system. The cask loading area in the north spent fuel pool contains a spent fuel rack seismic restraint that interferes with the placement of the storage cask in the pool for loading. The structure has been evaluated and can be removed. However, to remove the structure, KPS needs approval of this TS amendment, in conjunction with KPS License Amendment Request 227, to allow use of a single-failure-proof crane to move heavy loads in or over the pool. Following removal of the seismic structure, KPS can begin the dry runs required to demonstrate the capability of the ISFSI and the associated equipment and procedures. Assuming approval of this proposed amendment in late May, the expected time line for this effort is: (1) seismic restraint in the north pool removed by mid to late July, (2) dry runs completed by mid to late September, and (3) completion of four casks loads by late October, prior to the onset of winter weather. After NRC Staff approval of this amendment request, the KPS USAR will be revised as indicated in Attachment 2.

If you have any questions or require any additional information, please contact Mr. Gerald Riste at (920) 388-8424.

Very truly yours,

Gerald T. Bischof
Vice President-Nuclear Engineering

COMMONWEALTH OF VIRGINIA)
)
COUNTY OF HENRICO)

The foregoing document was acknowledged before me, in and for the County and Commonwealth aforesaid today by Gerald T. Bischof, who is the Vice President–Nuclear Engineering of Dominion Energy Kewaunee, Inc. He has affirmed before me that he is duly authorized to execute and file the foregoing document in behalf of that Company, and the statements in the document are true to the best of his knowledge and belief.

Acknowledged before me this 9th day of November, 2007.

Margaret B. Bennett
Notary Public

MARGARET B. BENNETT
Notary Public 354302
Commonwealth of Virginia
My Commission Expires Aug 31, 2008

Attachments:

1. Evaluation of License Amendment Request 234
2. USAR Markup Pages for License Amendment Request 234
3. "Response of Sliding Structures to Earthquake Response Motion," N. Mostaghel and J. Tanbakuchi, Earthquake Engineering and Structural Dynamics, Volume II, 729-748

Enclosures:

1. General arrangement drawings of upgraded KPS Auxiliary Building Crane

    a. D-20776-001, Single Failure Proof Trolley #2949 General Arrangement, Elevation View.
    b. D-20776-002, Single Failure Proof Trolley #2949 General Arrangement, Plan View.

Commitments made in this letter:

1. DEK will complete required modifications to the Auxiliary Building crane bridge girder to end-tie connections and confirm bolting material as described in Section 4.2.2.9 of this LAR.

cc: Regional Administrator
U.S. Nuclear Regulatory Commission
Region III
2443 Warrenville Rd.
Suite 210
Lisle, IL 60532-4532

Mr. P. D. Milano
Senior Project Manager
U.S. Nuclear Regulatory Commission
Mail Stop O-8-H-4a
Washington, DC 20555

NRC Senior Resident Inspector
Kewaunee Power Station

Public Service Commission of Wisconsin
Electric Division
P.O. Box 7854
Madison, WI 53707

ATTACHMENT 1

LICENSE AMENDMENT REQUEST 234
REQUEST FOR REVIEW AND APPROVAL OF METHODOLOGY CHANGE
REGARDING AUXILIARY BUILDING CRANE UPGRADE

EVALUATION OF LICENSE AMENDMENT REQUEST 234

KEWAUNEE POWER STATION

DOMINION ENERGY KEWAUNEE, INC.

EVALUATION OF LICENSE AMENDMENT REQUEST 234
Request for Review and Approval of Methodology Change Regarding Auxiliary Building Crane Upgrade

1.0 DESCRIPTION

This letter is a request to amend Operating License No. DPR-43 for the Kewaunee Power Station (KPS). KPS is currently in the process of implementing modifications to the auxiliary building crane that will make the crane single failure proof.

In accordance with 10CFR50.90 and 10CFR50.59, Nuclear Regulatory Commission (NRC) Staff review and approval is required for this proposed change to the KPS design and licensing basis. Specifically, Dominion Energy Kewaunee, Inc. (DEK) proposes to revise the KPS USAR to allow the use of a methodology for performing the seismic analysis of the upgraded Auxiliary Building (AB) crane. The new methodology is not currently described in the KPS licensing basis or in NRC-approved guidance for such analyses. KPS plans to use this crane for spent fuel cask loading operations in the spent fuel pool. A separate License Amendment Request (LAR), LAR 227, "Relocation of Spent Fuel Pool Crane Technical Specification to Technical Requirements Manual," will be submitted to the NRC to request review and approval of a Technical Specification change. LAR 227 will request modification and relocation of the spent fuel crane TS to the Technical Requirements Manual to support using the crane for spent fuel cask loading operations. The following provides a description of the proposed change (including the associated technical analysis), evaluation of significant hazards, and environmental impact evaluation.

2.0 PROPOSED CHANGE

The purpose of this amendment request is to modify the KPS design and licensing basis for the auxiliary building (AB) crane. A modification, currently in progress, is replacing the existing non-single-failure-proof crane trolley with a single-failure-proof design. The load rating of the crane will remain at 125 tons and the crane bridge will not be replaced. The bridge has been re-analyzed and will be modified as part of a combined upgraded bridge-and-trolley combination. The upgraded AB crane will meet applicable NRC and industry guidance for single-failure-proof-cranes provided in NUREG-0612 (reference 1) and NUREG-0554 (reference 2). The only exception to these guidance documents is the consideration of rolling for the crane seismic analysis.

Following NRC approval, Section 9.5.2.12, and Appendix B of the KPS USAR will be modified as shown in Attachment 2, and subsequently submitted in accordance with 10CFR50.71(e).

3.0 BACKGROUND

DEK intends to complete construction and start operation of an independent spent fuel storage installation (ISFSI) at the KPS site in mid-2008 under the general license provisions of 10 CFR 72, Subpart K. DEK will be using the Transnuclear standard NUHOMS dry spent fuel storage system in accordance with 10 CFR 72 Certificate of Compliance 1004. The standard NUHOMS system requires the use of a transfer cask weighing up to 125 tons during cask loading operations in the AB. The KPS AB crane will be used to lift and move the transfer cask between the cask loading area of the spent fuel pool and the truck bay. The existing AB crane is a Whiting Corporation 125-ton capacity, non-single-failure-proof, bridge and trolley design of late-1960s vintage. The AB crane bridge travels in the east-west direction on the AB runway rails and the trolley runs in the north-south direction on the bridge rails. To facilitate its use in dry spent fuel storage system loading operations, the AB crane is being upgraded to a single-failure-proof design by replacing the trolley with a single-failure-proof design and modifying the existing bridge. The design rated load and maximum critical load for the AB crane will remain 125 tons. The design of the new single-failure-proof trolley on the existing bridge requires a re-analysis of the existing bridge structure due to an increase in trolley weight and differences in the design and arrangement of the new trolley compared to the existing trolley.

The KPS AB crane is designated Service Class A in accordance with Crane Manufacturer's Association of America Standard 70 (CMAA 70) (reference 7), Section 2.2. The AB crane has a Class I* nuclear safety design classification in accordance with KPS USAR Appendix B, Section B.2.1, which is a mixed classification. Components designated as Class I* in the KPS licensing basis are designed to Class I design basis earthquake loading and treated as Class III in all other respects[1]. In summary, upon completion of this modification, the AB crane will prevent uncontrolled lowering of the load and the trolley and bridge wheels will remain on their respective rails under postulated DBE conditions with a suspended load up to and including the 125-ton rating of the AB crane. It is not required that the crane remain operational during or following a design basis event.

The existing AB crane was designed, fabricated, and qualified in accordance with Electrical Overhead Crane Institute (EOCI) Standard 61; American National Standard Institute (ANSI) Standard B30.2.0, 1967 Edition; and Pioneer Service and Engineering Company Specification for Powerhouse Overhead Electric Traveling Cranes. DEK has chosen not to postulate and analyze potential drops of a loaded transfer cask during dry spent fuel storage system loading operations. Instead DEK has chosen to replace the

[1] As stated in KPS USAR, Section B.2.1, Class I structures and components are those whose failure might cause or increase the severity of a loss-of-coolant accident or result in an uncontrolled release of substantial amounts of radioactivity, and those structures and components vital to safe shutdown and isolation of the reactor. Class III structures and components are not directly related to reactor operation or containment.

AB crane trolley with a single-failure-proof design meeting the guidance in NUREG-0612, "Control of Heavy Loads at Nuclear Power Plants," (reference 1), Section 5.1.6 and Appendix C; and NUREG-0554, "Single Failure Proof Cranes for Nuclear Power Plants" (reference 2). Furthermore, DEK has reviewed NRC Bulletin 96-02 (reference 3) and the guidance contained in NRC Regulatory Information Summary (RIS) 2005-25 (reference 4) and concludes the design addresses the issues and guidance described in them, as described in this submittal.

The original seismic analysis of the AB crane, which constitutes the current licensing basis, is an equivalent static analysis using accelerations provided in the original procurement specification. While the original seismic analysis was performed assuming the design rated load of 125 tons was attached to the main hoist, the seismic accelerations were applied only to the weight of the crane, not the suspended load. This was the customary approach for non-single-failure proof cranes designed and procured at the time KPS was constructed. However, current NRC guidance and construction codes and standards for single-failure-proof cranes require seismic analyses to be conducted assuming the design rated load is suspended on the crane hook. Therefore, a new seismic analysis approach and inputs were applied to the upgraded KPS AB crane. The discussions in Section 4.2.1 and 4.2.2 below provide a summary of the new seismic analysis approach and the application of KPS-specific seismic inputs.

4.0 TECHNICAL ANALYSIS

4.1 Applicability of Codes and Standards

The AB crane is described in KPS USAR Section 9.5.2.12. The existing crane is non-safety-related and non-single-failure-proof. It was designed and procured as a commercial grade item during initial plant construction. It was designed and constructed in accordance with EOCI Standard 61; ANSI B30.2.0, 1967 Edition; Pioneer Service and Engineering Company Specification for Powerhouse Overhead Electric Traveling Cranes; and other contemporaneous commercial codes and standards applicable to such components. KPS Safety Classification I* for the AB crane ensures that during an uncontrolled lowering of the lifted load, up to and including the 125-ton rated load of the crane, the bridge and trolley wheels will not leave their rails. The crane upgrade project will enhance the design of the AB crane by replacing the existing trolley with a single-failure-proof trolley constructed in accordance with the guidance in NUREG-0554.

DEK is aware that current NRC approved guidance for a complete crane replacement is ASME NOG-1-2004, "Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder)" (reference 5). This standard is referenced in Standard Review Plan Section 9.1.5, "Overhead Heavy Load Handling Systems," Revision 1

(reference 6), as meeting the intent of NUREG-0554. At KPS, the establishment of the governing codes and standards for the crane upgrade project is based on replacing the trolley and using the existing bridge. The design phase of the project was initiated, and the applicable codes governing the project were selected, before the release of SRP 9.1.5, Revision 1, in January, 2007. Therefore, the primary code of reference is CMAA 70 (reference 7) rather than ASME NOG-1. However, ASME NOG-1-2004 is used where CMAA 70 does not provide applicable guidance as described in further detail below.

NUREG-0554 either provides specific guidance or invokes the guidance in CMAA 70. If detailed guidance is not provided in NUREG-0554 or CMAA 70, the guidance in ASME NOG-1-2004 is used. KPS-specific licensing basis information is used to develop the seismic response spectra used in the seismic analysis. The damping values used in the seismic analysis are a combination of the existing KPS licensing basis values and the values suggested in ASME NOG-1-2004. The development of the seismic response spectra and damping values used is described in more detail in Section 4.2.

As stated above, the fundamental document applicable to the crane upgrade project is NUREG-0554. However, the trolley is being replaced and the existing bridge is being modified and re-used. Therefore, the governing codes and standards providing the specific construction[2] guidance for these components are slightly different from one another, as described below:

New Trolley Codes and Standards

Construction is in accordance with NUREG-0554 and CMAA 70 (2004). Seismic load combinations and stress analysis acceptance criteria, as well as guidance used to address two-blocking, load hang-up, and wire rope failure are taken from NUREG-0554 and the KPS licensing basis. In areas where these documents are silent ASME NOG-1-2004 is used.

Existing Bridge Codes and Standards

Construction is in accordance with NUREG-0554 and EOCI-61. Where NUREG-0554, and Electrical Overhead Crane Institute (EOCI) EOCI-61 for (reference 13) do not offer specific guidance, the American Institute of Steel Construction (AISC) Manual (9th and 13th Editions), CMAA 70 (2004) and ASME NOG-1-2004 are used in that order of hierarchy for the combined upgraded trolley/bridge analyses.

[2] The term "construction" as used herein is a simplifying term referring generally to all applicable code requirements pertaining to design, analysis, fabrication, inspection, and testing of the AB crane.

4.2 Upgraded AB Crane Seismic Analysis

This discussion is presented in the following manner: 1) the KPS licensing and design basis pertaining to seismic analysis of Class I structures is described; 2) the proposed new licensing and design basis used to analyze the upgraded AB crane is described; 3) appropriate technical justification is provided for any deviations from the KPS licensing basis and/or industry codes and standards. It is these deviations for which NRC review and approval is requested.

4.2.1 KPS Licensing Basis

4.2.1.1 KPS Seismic Accelerations and Response Spectra

The KPS AB crane is designated Class I* as stated in the KPS USAR, Appendix B, Table B.2-1. In accordance with USAR Section B.2.1, Class I* components must be designed to Class I DBE loading. This includes the trolley, bridge, and auxiliary building crane support system. The design basis earthquake at KPS is based upon a maximum horizontal ground acceleration of 0.12 g. The response spectra for seismic analysis performed for the KPS are documented in a report developed by John A. Blume and Associates for Pioneer Service and Engineering Company in 1971 (reference 8). Hereafter this report will be referred to as the "Blume Report."

The Blume Report describes an equivalent multi-mass mathematical model used to perform dynamic analyses of the various Class I structures at KPS. Maximum responses were developed for a mass point for each mode in the model using the operational basis earthquake (OBE) ground response and damping factors in KPS USAR Table B.6-5. The total response for each point was determined using the square-root-of-the-sum-of-squares (SRSS) method. From these data, a set of floor response spectra curves was developed showing the variation with height of the maximum translational accelerations, displacements, shears, and moments in each structure. Vertical acceleration equal to two-thirds the horizontal ground acceleration was applied to each structure. Stresses were computed for the various parts of the Class I plant structures using forces developed from the Blume Report data. The forces used for analysis of stresses for the DBE were taken to be twice the forces determined by the spectral analyses for the OBE.

4.2.1.2 Load Combinations for KPS Components

KPS USAR Appendix B, Table B.7-1 provides the load combinations applicable to plant components, by component class and condition. Class I* components such as the AB crane must be designed for the normal operating condition and DBE condition, as follows:

Normal Operating Condition: Dead + Live + Environmental Loads (Snow or Wind)*

DBE Condition: Dead + Live + DBE Loads

*Because the AB crane is located indoors, environmental loads are not applicable.

4.2.1.3 Allowable Stresses

KPS USAR Appendix B, Table B.7-5 refers to "normal" condition and "faulted" condition stress limits for Class I components (which apply to Class I* components for seismic analysis). For normal loading conditions, the structural steel allowables are those specified in the applicable standards of the American Institute of Steel Construction (AISC). For DBE loadings, the structural steel allowable stresses are 1-1/2 times the AISC allowable values. The applicable stress limits are summarized in Table 4-2.

4.2.1.4 Damping

KPS USAR Appendix B, Table B.6-5 provides the damping values to be used for seismic analysis. In general, for electrical and mechanical equipment evaluated in accordance with the Blume Report, 1.0 percent of critical damping should be used. However, as discussed in Section 3.0 the original crane seismic analysis was an equivalent static analysis rather than a dynamic analysis and the AB crane was not considered plant "electrical and mechanical equipment" at the time. Therefore, damping was not a part of the original seismic analysis that forms the AB crane's current licensing basis. The upgraded crane was analyzed using a dynamic analysis with damping applied to the seismic accelerations. The analysis uses 1 percent damping in the vertical direction in accordance with the KPS USAR and 7 percent damping in the horizontal direction in accordance with ASME NOG-1-2004 for the Safe Shutdown Earthquake. This is a conservative approach because ASME NOG-1-2004 allows 7 percent damping in both directions.

The application of these loading combinations, allowable stresses, damping values, and any unique requirements prescribed by NUREG-0554 for single-failure-proof cranes is discussed in Section 4.2.2.

4.2.2 Auxiliary Building Crane Upgrade Seismic Analysis

4.2.2.1 Response Spectra, Damping, and Accelerations

The seismic analysis of the new trolley and the existing bridge and AB crane support structure hinges on the development of appropriate inputs. These inputs include load combinations, seismic ground accelerations, damping, or structure response spectra, to

generate bounding forces to be used in analysis and design. The objective is to develop conservative, bounding inputs that are code-compliant.

The Blume Report does not include horizontal response spectra data for a mass point at the location of the AB crane rail. Therefore, a lumped-mass stick model of the auxiliary building steel structure was used to generate additional horizontal response spectra for use at the AB crane rail. These are referred to as the "extended Blume spectra."

In accordance with ASME NOG-1-2004, Section 4153.8, 7 percent damping for the safe shutdown earthquake condition is applied to the horizontal response spectra at the crane rail elevation. Because the vertical response spectra developed in the Blume Report are applicable for all higher elevations of the reactor, auxiliary and turbine building complex, 1 percent damping is appropriate for use in the vertical direction in accordance with the KPS USAR. For the seismic directional combination, the two-dimensional absolute sum method is employed and the more conservative combination of vertical with east-west or vertical with north-south is used, as applicable. For close mode combinations, the absolute sum addition method, per ASME NOG-1-2004 was applied for conservatism. The missing mass, as described in NRC Regulatory Guide 1.92, Revision 2, (reference 12), was taken into account in the analysis.

4.2.2.2 Application of Seismic Loads

Design analyses using the seismic accelerations developed in the east-west direction from the above application of response spectra and damping resulted in an unrealistic and overly conservative overstress condition for the existing AB bridge girders and structure. Significant modifications to strengthen the bridge girders, bridge structure, and possibly the auxiliary building structure would have been required if these design inputs were used. Therefore, consideration of rolling of the crane trolley and bridge on their respective rails during a seismic event was evaluated as a method to moderate the DBE design loadings in a realistic, yet still conservative manner. The crane wheel rolling occurs when the seismic load exceeds the crane holding brake torque. Consideration of crane rolling during a seismic event is an analytical technique used in seismic analyses to more accurately represent the true physics of the components' behavior in the dynamic model.

The existing AB crane bridge, which travels in the east-west direction, has eight bridge wheels, including two driven wheels (i.e., wheels with a motor and brakes to provide driving and retarding forces) and six idler wheels, which roll along the crane rails as the bridge is driven. The new AB crane trolley has four wheels, with two driven wheels and two idler wheels.

Conventional linear seismic analyses assume the drive wheels are restrained in the direction parallel to the respective rails. This assumes that the crane is pinned to the rails and the magnitudes of the resulting computed forces can be unrealistically large. A

more realistic yet conservative assumption is that the magnitude of the resultant forces is limited by the design holding brake torque of the driven wheels. When the limiting force needed to overcome the brake torque is reached, the crane bridge and/or trolley wheels will roll on the rails and/or roll through the brakes. During a seismic event, only the drive wheels roll with brake resistance while the idler wheels simply roll on the rails because they do not have brake resistance.

The lateral seismic force at the point of incipient rolling divided by the dead weight of the crane and trolley is the equivalent coefficient of friction. The method used to consider crane wheel rolling is consistent with that described in the Mostaghel and Tanbakuchi paper published in 1983 in "Earthquake Engineering and Structural Dynamics" (reference 9). The methodology in this paper provides guidance for simulating the non-linear time-history behavior of rolling with a conventional linear analysis method. Reference 9 is provided as Attachment 3 to this submittal.

4.2.2.3 Load Combinations

The load combinations for the analysis and design of the new trolley are based upon CMAA 70, Section 3.3.2.6, NUREG 0554, and the seismic requirements of KPS USAR Appendix B. They are provided in Table 4-1 below:

Table 4-1
KPS Auxiliary Building Crane Upgrade Load Combinations

CASE NUMBER	LOAD COMBINATION	STRESS LEVEL
CASE 1	$DL(DLF_B) + TL(DLF_T) + LL(1 + HLF) + IFD_T + IFD_B$	1
CASE 2	$DL(DLF_B) + TL(DLF_T) + LL(1 + HLF) + IFD_T + IFD_B + WLO + SK$	2
CASE 3A*	DL + TL + WLS	3
CASE 3B**	DL + TL + LL + CF	3
CASE 3C***	DL + TL + 1.25 LL	3
CASE 4a	DL + TL + LL + DBE	4a
CASE 4b	DL + TL + LL + DBA	4b

* This case will not be evaluated since the crane operates indoors.
** This case will not be evaluated for the trolley since the collision forces are negligible.
*** This case will not be evaluated since it has been determined that the Case 1 is more severe than Case 3C.

Note that load combination Case Numbers 1 and 2 above and the allowable stresses in Table 4-2 below bound the loading requirements and allowable stresses of EOCI Specification 61 for normal operation. The terms indicated in the loading combinations are defined as follows:

Term	Definition
DL	Bridge Dead Load = 106,000 lb
DLF_B	Dead Load Factor for Bridge = 1.1 (Max. speed = 50 fpm)* (reference 7)
TL	Trolley Dead Load = 112,000 lb
DLF_T	Dead Load Factor for Trolley = 1.1 (Max. speed = 40 fpm)* (reference 7)
LL	Lifted Load/Design Rated Load = 250,000 lb
HLF	Hoist Load Factor 0.15 (reference 7)
TAR	Trolley Acceleration Rate = 0.167 ft/sec^2 ** (calculated value)
BAR	Bridge Acceleration Rate = 0.250 ft/sec^2 ** (calculated value)
IFD_T	Inertia Force Factor for Trolley = 7.8 x TAR = 0.061 (see Section 4.2.2.4 below)
IFD_B	Inertia Force Factor for Bridge = 7.8 x BAR = 0.061 (see Section 4.2.2.5 below)
WLO	Operational Wind Loading***
WLS	Stored Wind Load***
SK	Skewing Forces – Bridge and Trolley****
CF	Collision Forces – Bridge and Trolley****
DBE	Design Basis Earthquake
DBA	Design Basis Accident Conditions - two-blocking, load hang-up, and broken wire rope****

* 1.15 is used for design.
** TAR = BAR = 0.250 ft/sec^2 for design. This is conservative. The value of 7.8 is divided by the acceleration of gravity, 32.2 ft/sec^2.
*** Not applicable. Crane is located indoors.
**** Considered in the design, but not part of this LAR scope.

4.2.2.4 Trolley Inertia Force from Drives

The trolley inertia force from the drives produces a tractive force on the trolley wheels. This force is based upon the live load and trolley dead load. The tractive force is resisted at the two trolley drive wheels.

Dead Load of Trolley = 112,000 lbs
Lifted Load = 250,000 lbs
Total = 362,000 lbs
Total Tractive Force = IFD_T x Total

Total Tractive Force = 0.061 (362,000) = 22,082 lbs; 24,000 or 12,000 lbs per drive wheel is assumed in the design.

For the auxiliary hoist, lifted load = 30,000 lbs
Total Load = 30,000 + 112,000 = 142,000 lbs
Total Tractive Force = 0.061 (142,000) = 8,662 lbs

Assuming 9,000 lbs or 4,500 lbs per drive wheel for the auxiliary hoist, it can be seen that the forces associated with the design rated load on the main hoist control the design.

4.2.2.5 Bridge Inertia Force from Drives

The bridge inertia force from drives produces a tractive force at the bridge drive wheels. The thrust force on the bridge wheels is based on the live load and dead load of the trolley and the bridge girders.

Dead Load of Trolley = 112,000 lbs
Lifted Load = 250,000 lbs
Bridge Girders = 85,000 lbs
Total = 447,000 lbs
Total Thrust = IFD_B x Total
Total Thrust = 0.061 x 447,000 = 27,267 lbs; Use 28,000 lbs or 14,000 lbs per drive wheel.

4.2.2.6 Allowable Stresses

The design of structural members utilizes the allowable stresses as outlined in CMAA 70, Section 3.4. The design of welds is based upon the requirements of AWS D1.1, Table 2.3 (reference 14). The allowable member stresses for case and stress levels 1, 2, and 3 are proportioned in accordance with CMAA 70, Sections 3.4.1, 3.4.2, and 3.4.3.

Stress level 4a is limited to 1.5 times AISC allowable stresses per KPS USAR Appendix B, Table B.6.2. The stress level 4b allowable is 1.6 times that of the AISC normal allowable, which is more conservative than NUREG 0554.

Table 4-2
Allowable Member and Weld Stresses for Trolley and Bridge

Stress Level	Allowable Compression Stress*	Allowable Tension Stress	Allowable Shear Stress	Allowable Bearing Stress	Allowable Weld Stress
1	0.60Fy	0.60Fy	0.35Fy	0.75Fy	0.30 Fu weld metal 0.40 Fy base metal
2	0.66Fy	0.66Fy	0.375Fy	0.80Fy	0.33 Fu weld metal 0.44 Fy base metal
3	0.75Fy	0.75Fy	0.43Fy	0.90Fy	0.375 Fu weld metal 0.50 Fy base metal
4a	0.90Fy	0.90Fy	0.58Fy**	1.35Fy	0.45 Fu weld metal 0.58 Fy** base metal
4b	0.96Fy	0.96Fy	0.58Fy**	1.44Fy	0.48 Fu weld metal 0.58 Fy** base metal

* Not Subject to buckling
** 1.5 or 1.6 times AISC allowable shear is 0.60Fy or 0.64Fy, which is greater than ultimate shear, $(Fy/3)^{1/2}$ = 0.58Fy. Thus, 0.58Fy is used, where Fy = Yield stress of material.

4.2.2.7 Upgraded Auxiliary Building Crane Seismic Analysis Approach

The seismic analysis of the upgraded KPS AB crane was performed by the designer of the upgraded trolley, American Crane and Equipment Company (ACECO). The STARDYNE modules of computer program STAAD.Pro-2001, a general-purpose finite element program available in the public domain, was used to perform the seismic analysis. The STARDYNE computer modules have been verified and validated in accordance with ACECO procedure AP-20.5, "Computer Program Verification and Validation," as described in the STAAD.PRO 2001 documentation.

The seismic analyses are documented in detail in an ACECO calculation package (reference 11). Key elements of the analysis methodology and the results of the analysis are summarized below.

Mathematical Model

The bridge assembly and trolley frames are represented by a generalized three-dimensional lumped mass system interconnected by weightless elastic members. The model's geometry reflects the overall size, length, connectivity, and stiffness of various structural members. Member section properties are either obtained from AISC Steel Construction Manual, 9th Edition, or calculated using actual member dimensions. Material properties are based on selected ASTM material designation.

The interconnecting elastic members represent the centroidal axes of the physical members. When the interconnecting members do not meet at the same points, the offsets are represented by rigid links. These are artificially stiffened members and are an order of magnitude stiffer than the connecting members. The dimensions are obtained from the appropriate crane design drawings.

The wire rope and the lower block with the lifted weight behave like a pendulum during a seismic event. The horizontal load due to the pendulum effect is negligibly small due to low frequency. This can be confirmed by calculating the pendulum frequency of the "hook up" condition.

From the "hook up" model, the distance from the hook to the center of the upper block is 157.875 inches. Including the lifting yoke and slings, the distance from the center of the upper block to the cask trunnion is at least 1.5 x 157.875 = 236.8 inches. The pendulum frequency can be calculated as:

$$F = (g / L)^{0.5}/6.2832 = (386.4/236.8)^{0.5}/6.2832 = 0.21 \text{ Hz}$$

This pendulum effect is excluded from the dynamic model to avoid numerical instability. However, the pendulum mode responses are still conservatively combined with the

dynamic model responses by the SRSS method. The response spectra are directly applicable to the pendulum mode because the lowest horizontal mode frequency of the dynamic model is very high in comparison with the pendulum frequency.

Dynamic degrees of freedom are assigned to a sufficient number of lumped mass points and in locations that simulate the actual mass distribution. Structural members subjected to concentrated loads are provided with additional nodes at points where concentrated loads or their equivalent masses are positioned. No uplift restraint is required for the trolley, while seismic clamps are already installed on the existing bridge end-trucks.

Boundary Conditions

Member releases are based on the STARDYNE user's manual. The boundary conditions for the crane and trolley are as shown in ASME NOG-1, Figure 4154.3-1 and Table 4154.3-1.

Analysis Techniques

For the seismic analysis, the response spectrum method was applied with input response spectra. For vertical seismic input, the 1 percent damped Blume response spectrum was applied. For the N-S and E-W input, the 7 percent damped extended Blume response spectra were applied as discussed in Section 4.2.2.1 above. For the DBE analysis, a factor of 2 was applied to the OBE spectra in accordance with the KPS licensing basis. The modal combination method is consistent with the guidance in NRC Regulatory Guide 1.92, Revision 1 (reference 12), with the absolute addition of close modes. All three components of the response spectra were individually used as input. Horizontal effects were handled as equivalent static loads with due consideration of wheel rolling, as explained in more detail below. Seismic responses are selected from the larger of vertical plus E-W seismic input or vertical plus N-S seismic input, taking into account sign reversal.

Only the "hook down" condition was considered because the vertical frequency is closer to the peak of the vertical spectrum. The "hook up" condition would result in a vertical frequency on the higher frequency side of the peak spectrum region and would be less critical. The live-load-on-hook condition controls the design because the "no-load" frequency is even higher than the "hook up" case with load, creating lower spectrum values. In addition, the live load is not in the loading combination for the "no-load" case. The missing mass option, as described in NRC Regulatory Guide 1.92, Revision 2[3], is applied in these analyses. This option provides more accurate prediction of responses of less flexible systems.

[3] The absolute addition of close modes, from RG 1.92, Revision 1, and the missing mass option from Revision 2 of the RG are used to produce a conservative combined approach to the problem.

Frequencies of mid-span, quarter-span, and end-span cases were compared and used to select the two most critical cases for design. The mid-span model was selected to obtain the highest bridge bending moments and stresses. The quarter-span model was considered to compare response spectrum values, and the end-span case was considered to obtain higher wheel reactions.

The static load analysis was applied to dead load, live load, inertia load, and pendulum load. The lateral loads due to pendulum modes in the E-W and N-S directions were considered as two load cases in the static analyses. The final loading combinations used in the analysis are as follows:

CASE 1: 1.15 x (DL + TL + LL) + IFD, Main hoist
CASE 2: 1.15 x (DL + TL + LL) + IFD, Auxiliary Hoist
CASE 3: DL + TL + LL + SEISMIC, seismic vertical and N-S
CASE 4: DL + TL + LL + SEISMIC, seismic vertical and E-W
CASE 5: -DL - TL - LL + SEISMIC, seismic vertical and N-S
CASE 6: -DL - TL - LL + SEISMIC, seismic vertical and E-W
CASE 7: DL + TL + LL + DBA

Where SEISMIC = $S_{vertical}$+ $((SEISMIC_{ew\ or\ ns})^2 + (PENDULUM_{ew\ or\ ns})^2)^{0.5}$. As discussed in further detail in the trolley and bridge wheel rolling evaluation below, seismic lateral analyses are equivalent static 0.3 g for the mid-span and 0.5 g for the end-span in the E-W direction, and 1.1 g for both the mid-span and the end-span in N-S direction for the bridge and 0.21 g for the trolley, due to rolling.

4.2.2.8 Upgraded Auxiliary Building Crane Seismic Analysis Results

The results of the bridge and trolley seismic analysis are summarized in this section.

The larger deflection at the girder mid-span location for the static analysis is shown below for the dead load (including bridge girder dead weight) plus live load.

LOAD	VERTICAL DEFLECTION East Girder	VERTICAL DEFLECTION West Girder
Dead	-0.383"	-0.377"
Live	-0.564"	-0.565"
TOTAL	-0.947"	-0.942"

With the bridge span at 990 inches, the span/deflection ratio = 990.0/0.947 = 1,045, which is greater than 1,000. This ratio satisfies the acceptance criteria of ASME NOG-1, Section 4341, and CMAA 70, Section 3.5.5.1. The ratio of 1,045 is conservative

because it includes the deflection caused by the bridge girder dead weight, which is not included in the acceptance criteria of ASME NOG-1, Section 4341, and CMAA 70, Section 3.5.5.1.

Frequency Responses

The results of the analysis show that the mid-span and quarter-span spectrum values are comparable. Because the mid-span location results in the largest bending moment in the bridge, that location is selected for design. The end-span case will also be selected for the largest wheel reactions. The dominant frequencies of these two cases are presented below:

	North/South	East/West	Vertical
C1 – MIDDNLAN	12.70 Hz	1.93 Hz	1.89 Hz
C3 - ENDDNLAN	14.98 Hz	4.04 Hz	2.12 Hz

Consideration of Rolling[4]

The response spectrum analysis for the mid-span case results in very high maximum crane lateral accelerations: 1.6 g in the E-W direction and 1.2 g in the N-S direction. These high responses are not realistic because the dry coefficient of friction between the trolley and bridge wheels and their respective rails is only 0.2, per CMAA 70, Table 5.2.9.1.2.1-B. In order to more accurately account for the total wheel friction force, the brake torque on the drive wheels must also be considered in determining the external force required to overcome the applied brake torque and cause wheel rolling. The bridge drive wheels will roll before they slide when the external horizontal force in the direction of the bridge runway girder is increased to 12.5 kips.

The external horizontal force in the direction of the bridge runway girder (east-west (EW) direction), above which the bridge drive wheels will roll (R), was determined from the equation:

$$F_{EWR} = (T_{SBB} \times (BRR / \eta)) / RA_{BW}$$

Where:

T_{SSB} = Static Torque Rating of Bridge Brake = 75 ft-lb

η = Gearing Efficiency of Bridge Drive System = 0.913

[4] To avoid confusion, the term "rolling" is used throughout this LAR as a more accurate representation of the physical phenomenon being modeled. Reference 9 refers to "sliding." However the technique in this paper is applicable to sliding and rolling.

BRR	= Existing Bridge Reducer Ratio (total)	= R_{CGB} x R_{OGB}
R_{CGB}	= Ratio of Center Gear Box	= 36.7
R_{OGB}	= Ratio of Outboard Gearboxes	= 4.63
RA_{BW}	= Bridge Wheel Radius	= D_{BW} / 2
DBW	= Bridge Wheel Diameter	= 27 in

Therefore:

$$BRR = R_{CGB} \times R_{OGB} = 36.7 \times 4.63 = 169.92$$

$$RA_{BW} = D_{BW} / 2 = 27 \text{ in.} / 2 = 13.5 \text{ in.}$$

$$F_{EWR} = (75 \text{ ft-lb} \times (169.92 / 0.913)) / (13.5 \text{ in}) (12 \text{ in/ft}) = 12{,}407 \text{ lbs}$$

or

$$F_{EWR} \approx 12{,}500 \text{ lbs } (12.5 \text{ kips})$$

The source of the rolling friction is brake torque rather than the wheel rolling along the rail. The efficiency factor included in the calculation accounts for the inertia and friction in the gearboxes. Since the lower block pendulum action has a very low frequency, it is decoupled from the crane motion during a seismic event. Thus, the maximum horizontal acceleration that can be sustained by the rolling friction force is based on the inertia of the trolley and bridge without the lower blocks as follows:

$$A = 12{,}500/(100{,}085 + 106{,}000) = 0.061 \text{ g}$$

Where 100,085 lbs and 106,000 lbs are the weight of the trolley without the lower blocks and the weight of the bridge, respectively. The acceleration calculated above is equivalent to a friction coefficient of 0.061.

A detailed horizontal response has been evaluated based on a paper by N. Mostaghel and J. Tanbakuchi, "Response of Sliding Structures to Earthquake Support Motion," published in Earthquake Engineering and Structural Dynamics, Volume 11, pages 729-748, 1983 (reference 9 and Attachment 3 to this submittal).

The trolley is represented by a flexible mass and a rolling base mass, which are functions of the trolley location along the bridge. The mid-span and end-span cases were evaluated separately.

<u>East-West Mid-Span Case:</u>

In the E-W direction, the flexible mass is the trolley and the center one-half portion of the bridge, or 100,085 lbs + 42,500 lbs = 142,585 lbs. The rolling base is the end-truck and one-half portion of the bridge near the end-trucks, or 21,000 lbs + 42,500 lbs =

63,500 lbs. The flexible-to-base mass ratio is 2.25 (142,585/63,500). Based on Figures 3 & 4 of Reference 9, the zero period case applies to a rigid system and responds the same as the rolling base with no amplification. Other elastic systems are amplified and respond higher than the rolling base. For example, for the case of a friction coefficient equal to 0.05, the flexible system normalized response is about 0.345 in Figure 3 of Reference 9 (for a mass ratio of 3.0) and 0.435 in Figure 4 of Reference 9 (for a mass ratio of 1.0) at 1.93 Hz, which is the lowest E-W frequency. The rigid case or rolling base normalized response is 0.147. Thus, the flexible mass amplification is 0.345/0.147 = 2.35 for a mass ratio = 3.0 and 0.435/0.147 = 2.96 for a mass ratio = 1.0. The interpolated value for the mass ratio of 2.25 is 2.35 + (2.96 – 2.35) x (3.0 – 2.25)/(3 – 1) = 2.58

The flexible mass portion, which includes the trolley and the center one-half of the bridge girders, responds at 2.58 x 0.05 g = 0.13 g for a friction coefficient of 0.05. From Figure 3 of Reference 9, the flexible mass normalized responses are 0.345 for a friction coefficient of 0.05 and 0.597 for a friction coefficient of 0.1 at a frequency 1.93 Hz. Thus, the normalized response at a friction coefficient of 0.061 will be 0.345 + (0.061 – 0.05) (0.597 – 0.345)/(0.1 – 0.05) = 0.40.

The flexible mass with a friction coefficient of 0.061 will respond at 0.13 g x 0.40/0.345 = 0.151 g. The rolling base portion, which includes the end-trucks and the rest of the bridge girders, responds at 0.061 g.

<u>East-West End-Span Case:</u>

When the trolley is at the end of the bridge, the rolling base includes both end trucks, the outside one-half of the bridge girders, and the trolley. The flexible mass is the center one-half of the bridge girders. Thus, the mass ratio is 42,500 lbs/(42,500 lbs + 21,000 lbs + 100,085 lbs) = 0.26. The E-W dominant frequency is 4.04 Hz.

From Figures 4 & 5 of Reference 9, the flexible system normalized response is about 0.435 in Figure 4 and 0.647 in Figure 5 at 4.04 Hz and a friction coefficient of 0.05. The rigid case or rolling base normalized response is 0.147. Thus, the flexible mass amplification is 0.435/0.147 = 2.96 for a mass ratio = 1.0 and 0.647/0.147 = 4.40 for a mass ratio = 0.33. The extrapolated value for flexible mass amplification for a mass ratio of 0.26 is 2.96 + (4.40 – 2.96) x (1.0 – 0.26)/(1.0 – 0.33) = 4.55.

The flexible mass portion, which includes the center one-half of the bridge girders, responds at 4.55 x 0.05 g = 0.23 g for a friction coefficient of 0.05. From Figure 5 of Reference 9, the flexible mass normalized responses are 0.647 for a friction coefficient of 0.05 and 1.01 for a friction coefficient of 0.1 at a frequency 4.04 Hz. Thus, the normalized response at a friction coefficient of 0.61 will be 0.647 + (0.061 – 0.05) (1.01 – 0.647)/(0.1 – 0.05) = 0.727.

The flexible mass with a friction coefficient of 0.061 will respond at 0.23 g x 0.727/0.647 = 0.258 g. The rolling base portion, which includes end-trucks, the outer one-half of the bridge girders and trolley responds at 0.061 g.

North-South Direction:

A similar calculation can be performed in the N-S direction, where only the trolley rolls, not the bridge. Consequently, there is very little difference between the mid span case and the end span case.

The pendulum action of the lower block and live loads is decoupled from the trolley and crane lateral motions. Thus, it is conservative to remove the live load and lower blocks from the horizontal inertia for an equivalent lateral acceleration calculation. The horizontal available inertia is 110,000 lbs – 8,765 lbs – 1,150 lbs = 100,085 lbs. The maximum trolley acceleration that can be sustained by the friction force in the N-S direction = 3,960/100,085 = 0.04 g, which is equivalent to a friction coefficient of 0.04. The 3,960 pounds is the force required to overcome the trolley drive brake torque at the two drive wheels.

The trolley is very stiff in the N-S direction; therefore, the flexible parts including the upper block and drums and the rolling base, including the trolley end trucks, have similar acceleration at 0.04 g. However, the bridge will not roll on the crane rail in the transverse direction. The dominant N-S frequency is 12.70 Hz. The corresponding DBE 7 percent damped spectrum acceleration value is 2 x 0.55 = 1.1 g, which is much higher than the sustainable trolley acceleration of 0.04 g. Thus, it is reasonable to apply an equivalent acceleration of 1.1 g in the N-S direction to both the trolley and the bridge.

In conclusion, after the actual brake torque on the drive wheels is taken into account, the drive wheels will roll when the external horizontal force in the direction of the runway girder is increased to 3.96 kips. Therefore, an acceleration of 1.1 g in the N-S direction was applied to the trolley and bridges, including the end trucks, in the equivalent static analysis.

For the purpose of performing the steel structural design check and the bridge girder to end truck connection design check, a reduced trolley acceleration value can be used in the N-S direction. The fundamental N-S frequency is at 12.70 Hz. Its corresponding mode shape indicates that this is a bridge mode. Thus, the next higher N-S mode at 18 Hz can be considered as the trolley mode. From Figure 3 of Reference 9, the amplification factor at 18 Hz is about 1.8. With a safety factor of 2.9, the amplification factor is 1.8 x 2.9 = 5.2. Hence, the trolley acceleration is assumed to be 0.04 x 5.2 = 0.21 g in the equivalent static analysis, which is higher than the 0.2 friction coefficient given in CMAA 70.

The equivalent static analysis shows results of 1.1 g for the bridge and 0.21 g for the trolley in the N-S direction at mid-span and end-span, respectively.

East-West Direction Mid and End-Spans

In the E-W direction, the equivalent static loads vary with trolley location. At mid-span, the flexible mass portion, which includes the trolley and the center one-half of the bridge girders, responds at 0.151 g. The rolling base portion, which includes the end trucks and the rest of the bridge girders, responds at 0.061 g. At the end span, the flexible mass portion, which includes the center one-half of the bridge girders, responds at 0.258 g. The rolling base portion, which includes end-trucks, the outer one-half of the bridge girders, and trolley, responds at 0.061 g.

The E-W response spectrum is then used to determine at what zero period acceleration (ZPA) value the crane starts to roll.

When the trolley is at mid-span, the dominant frequency is 1.93 Hz with the corresponding E-W 7 percent damped spectrum (used for ratios only, hence OBE and DBE gives the same result) value at 0.56 g and the ZPA value of 0.17 g. With a flexible-to-rolling mass ratio of 2.25, the equivalent lateral load at impending rolling is 2.25 x 0.56 + 1 x 0.17 = 1.43. The equivalent lateral load based on the above rolling evaluation is 2.25 x 0.151 + 1 x 0.061 = 0.40. Thus, with the trolley at mid-span, the crane starts to roll at a ZPA value of:

$$0.17 \text{ g} \times 0.40 / 1.43 = 0.048 \text{ g}$$

Because 0.048 is less than 0.061, the crane will roll at 0.048 g instead of 0.061 g. The flexible mass responds at 0.56 x 0.048/0.17 = 0.158 g, and the rolling base responds at 0.048 g, which is slightly more critical than 0.151 g at the flexible mass and 0.061 g at the rolling base in terms of bridge stress. For design purposes, 0.3 g is conservatively applied to both the flexible mass and the rolling base, which is about double the calculated value.

When the trolley is at the end span, the dominant frequency is 4.04 Hz with the corresponding E-W 7 percent damped spectrum value at 0.48 g and a ZPA value of 0.17 g. With a flexible-to-rolling mass ratio of 0.26, the equivalent lateral load at impending rolling is 0.26 x 0.48 + 1 x 0.17 = 0.295. The equivalent lateral load based on the rolling evaluation is 0.26 x 0.258 + 1 x 0.061 = 0.128. Thus, with the trolley at end-span, the crane starts to roll at a ZPA value of:

$$0.17 \text{ g} \times 0.128 / 0.295 = 0.074 \text{ g}$$

Because 0.074 is higher than 0.061, the previous rolling evaluation for the end span case remains valid, and the crane is assumed to start rolling at 0.061 g. The flexible

mass responds at 0.258 g. For design purposes, 0.5 g is applied to both the flexible mass and the rolling base, which is about double the calculated value.

Displacement Evaluations

From Figures 7, 8 and 9 of Reference 9, the maximum rolling displacement is less than 0.9 times the input displacement when the frequencies are equal to or larger than 1.93 Hz. The maximum input displacement from DEK's calculations is 0.16 inch in the E-W direction and 2.37 inches in the N-S direction. Thus, the crane rolling displacement in the E-W direction is less than 0.16 x 0.9 = 0.14 inch, and the trolley rolling displacement in the N-S direction is less than 2.37 x 0.9 = 2.13 inches.

The pendulum frequency is 0.21 Hz, which is for the "hook up" condition. Since the "hook down" condition is the possible position for interference with the building structure, the frequency at the "hook down" condition was evaluated. From the crane design drawings, the center of the upper block is 53.75 inches above the top of the rail, which is at elevation 679'-11", or 8,159 inches. The bottom of the spent fuel pool is at elevation 608' or 7,296 inches. Assuming the center-of-gravity of the lifted load (i.e., the spent fuel cask) is 60 inches above the bottom of the cask, the pendulum length is 8,159 inches +53.75 inches – 7,296 inches – 60 inches = 856.75 inches. The "hook down" pendulum frequency can be calculated as:

$$F = (g / L)^{0.5}/6.2832 = (386.4/856.75)^{0.5}/6.2832 = 0.109 \text{ Hz}$$

This frequency is so low that it is outside the range of the DBE seismic response spectra. Very rigid equipment responds at the same absolute acceleration as the input absolute acceleration, and very soft equipment responds at the same relative displacement as the input relative displacement. Thus, the E-W pendulum displacement is about 0.16 inch, and the N-S pendulum displacement is about 2.37 inches.

The rolling displacements are stop-and-go type non-linear displacements, and the pendulum displacements are low frequency displacements. It is very unlikely that the maximum pendulum displacement will occur at the same time as the maximum rolling displacement. In theory, they should be combined by the SRSS method. However, they are conservatively combined in this calculation by the absolute addition method.

The final E-W displacement is about 0.14 inch + 0.16 inch = 0.3 inch, and the final N-S displacement is about 2.13 inches + 2.37 inches = 4.5 inches. These displacements are less than the available clearances of 6.12 inches in the E-W direction and 8.6 inches in the N-S direction, and are, therefore, acceptable. There is no impact between the swinging cask during a seismic event and the spent fuel pool walls or spent fuel racks.

It should be pointed out that for a very flexible pendulum such as a load suspended on a long wire rope, the motion of the pendulum is relative to its support. In reality, the support is moving, and the very soft pendulum is isolated from its support and is almost stationary. Thus, the displacement of a very flexible pendulum is the same as its support motion.

Member Stress and Connection Checks

Member stress and connection checks were performed for different crane members and connections as discussed below. The bridge material is ASTM A36 as shown in the original crane bridge drawings and the upgraded trolley material is ASTM A572, Grade 50. The weld electrode used on the trolley is E70. For the bridge, E60 weld electrode is conservatively assumed. For the seismic cases, fatigue evaluations are not performed because the cycles are too few in number to be of concern.

Bridge Girder Seismic Case Stress Check

The maximum computed bridge girder stress for the seismic loading condition is 30.76 ksi, which is less than the allowable stress of 32.4 ksi, and is acceptable. For the welds between flanges and webs, the required weld size is 0.27 inch, which is less than the 0.3125-inch welds shown on the girder drawings for the weld between the top or bottom flange and the web. Therefore, the existing bridge girder welds are adequate.

Bridge End Truck Seismic Case Stress Check

The maximum computed seismic stress is 19.02 ksi, which is less than the acceptance criterion of 32.4 ksi, and is acceptable. These end-trucks have no welds between the flanges and the webs because they are formed from two WF27x94 beams. The flange butt welds are located at the flange centers, and are not exposed to tensile or shear stress.

Bridge End-Tie Seismic Case Stress Check

The maximum computed seismic stress is 24.86 ksi, which is less than the acceptance criterion of 32.4 ksi, and is acceptable. For the welds between flanges and webs, the weld stress is 2.88 ksi, which is less than the acceptance criterion of 12.73 ksi and is acceptable.

Trolley End Truck Seismic Case Stress Check

The maximum computed seismic stress is 31.09 ksi, which is less than the acceptance criterion of 45 ksi, and is acceptable. For the flange-to-web connection, the required weld size is 0.23 inch. The actual weld size is 0.3125 inch as shown on the trolley drawing, and is acceptable.

Trolley Main Load Girt Seismic Case Stress Check

The interaction coefficient is 0.77, which is less than 1.0 and is acceptable. For the welds between the flange and the webs, the required weld size is 0.15 inch. The design weld size is 0.25 inch as shown in the trolley drawing and is acceptable.

Check of Connections between Bridge Girder and End Tie under Seismic Load

The existing $^1/_4$" stitch welds were found to be undersized. The connection will be strengthened by adding a 1-inch thick plate, welded to the bridge girder with a minimum $^5/_8$-inch fillet weld using E70 low hydrogen electrode and bolted to the end-tie with 12 one-inch diameter ASTM A325 bolts.

The existing two exterior (outboard) 1 $^1/_8$-inch diameter bolts on the top flange are within allowable stress if they are ASTM A325 bolts. The existing two exterior bolts will either be verified to be ASTM A325 material or will be replaced with bolts made from ASTM A325 material.

Check of Connections between Bridge and End Trucks under Seismic Load

From the bridge drawing, it is shown that the connection at the top of the end-truck is through the 8" x 6" x ½" angle, which is bolted to the end-truck and welded to the bridge girder. The connection at the bottom of the end-truck is through the 5/8-inch plate and the 6" x 6" x ½" angle.

The vertical force, P, is resisted by the 5/16-inch web plates as compression down to the end truck's 19-inch wide top flange and the 7/16-inch thick, 20-inch wide stiffener plates in contact with the end-truck top flange. The resulting vertical stress is 11.2 ksi, which is less than the acceptance criterion of 32.4 ksi, and is acceptable. The maximum bolt shear stress is 13.98 ksi, which is less than the allowable stress of 15 ksi, and is acceptable.

4.2.2.9 Conclusions

As used in the KPS AB crane analysis, consideration of rolling is used conservatively and reflects the true physical configuration of the trolley and bridge assemblage. The analysis concludes that the trolley and bridge components are designed to satisfy the KPS design criteria for seismic loads from the DBE response spectra. The potential wheel uplifting condition was also evaluated and it was concluded that there was no uplift at the trolley. The existing bridge design includes seismic clamps to prevent uplift, which will remain in place, even though the seismic analysis indicates that there is no bridge uplift.

Because the vertical response spectrum dominant frequencies are low, the "hook down" (live load on lowered hoist) condition is limiting. Dominant natural frequencies for the "hook down" condition are as follows:

	North/South	East/West	Vertical
C1 – MIDDNLAN	12.70 Hz	1.93 Hz	1.89 Hz
C3 – ENDDNLAN	14.98 Hz	4.04 Hz	2.12 Hz

The maximum lower block vertical acceleration is 1.08 g. Maximum accelerations for rigidly mounted mechanical and electrical components are:

Maximum acceleration in the X direction (N-S): 1.1 g + 0.2 g = 1.3 g
Maximum acceleration in the Y direction (Vertical): 0.5 g
Maximum acceleration in the Z direction (E-W): 0.3 g + 0.4 g = 0.7 g (except at node 608 for the bevel drive, which is 0.5 g + 0.3 g = 0.8 g)

Based on the design calculations, the trolley frame, bridge girders, end-trucks, and end-ties are adequate to resist design loads with the maximum critical load on the crane hook, provided recommended modifications to the bridge girder to end-tie connections are implemented and certain bolting material can be assured. Specifically, at these connections, one-inch plates (ASTM A572 Grade 50) must be added to strengthen the design. These plates must be connected to the bridge girders with minimum 5/8-inch fillet welds (E70 low hydrogen electrodes) and to the end-ties with 12 one-inch diameter ASTM A325 bolts. In addition, the bolts located on the top flanges at these connections will be confirmed to be ASTM A325 bolts. If this cannot be confirmed, the two exterior (outboard) bolts will be replaced with ASTM A325 bolts.

Based on the above and contingent upon strengthening of the girder-to-end tie welds and confirming or replacing certain bolting material, the upgraded KPS AB crane will meet all acceptance criteria for applicable loading combinations for a new trolley installed with the existing crane bridge.

5.0 REGULATORY SAFETY ANALYSIS

5.1 No Significant Hazards Consideration

Dominion Energy Kewaunee, Inc. (DEK) proposes to modify the Kewaunee Power Station (KPS) licensing basis for seismic evaluation of the Auxiliary Building (AB) crane to permit the consideration of trolley and bridge rolling. The seismic analysis is otherwise consistent with the methods for seismic analysis recommended in ASME NOG-1-2004, which does not address the consideration of rolling.

DEK has evaluated whether or not a significant hazards consideration is involved with the proposed amendment by focusing on the three standards set forth in 10 CFR 50.92, "Issuance of Amendment," as discussed below:

1. **Does the proposed amendment involve a significant increase in the probability or consequences of an accident previously evaluated?**

   Response: No.

   This amendment request pertains solely to an analysis method supporting the upgrade of the KPS AB crane from a non-single-failure-proof design to a single-failure-proof design. The AB crane is used to lift and handle loads in the KPS spent fuel pool and truck bay areas. The AB crane does not interface with operating plant equipment. The design rated load of the AB crane remains the same as previously approved. The proposed amendment does not change the current heavy load handling practices that are in use at KPS. Upgrading the AB crane to a single-failure-proof design will reduce the probability of a heavy load drop in the areas where the AB crane lifts and handles loads.

   The seismic analysis method proposed for use recognizes the inherent propensity for structures not fixed to one another (e.g., steel wheels on steel rails) to roll if sufficient lateral force is applied to either object. This seismic analysis method is proposed for use solely on the AB crane upgrade and not for any other plant structures, systems, or components. The recognition of wheel rolling between the AB crane trolley and bridge and their respective rails reflects the true nature of the installed equipment and its response to horizontal forces generated by a seismic event. Consideration of rolling reduces the projected analyzed loads on the crane and building structures and eliminates the need for unnecessary modifications to both.

   Therefore, the proposed amendment does not involve a significant increase in the probability or consequences of an accident previously evaluated.

2. **Does the proposed amendment create the possibility of a new or different kind of accident from any accident previously evaluated?**

   Response: No.

   This amendment request pertains to an analysis method supporting the upgrade of an existing plant component. Specifically, the existing AB crane trolley is being replaced with a state-of-the-art design that is single-failure-proof. The AB crane does not interface with operating plant equipment. This seismic analysis method is proposed for use solely on the AB crane upgrade and not for any other plant structures, systems, or components.

The design rated load of the AB crane remains the same at 125 tons. This load controls the design and supporting analysis. The auxiliary hook design rated load is being increased from 10 tons to 15 tons. The proposed amendment does not change the currently acceptable heavy load handling practices in use at KPS. The number and types of lifts made using this crane in support of KPS plant operations are not significantly changed from that contemplated during original plant licensing. Furthermore, the basic operations of the crane (i.e., hoisting and horizontal travel) remain the same, although the electronic controls will be upgraded to current standards.

Therefore, the proposed amendment does not create a new or different kind of accident from any accident previously evaluated in the KPS licensing basis.

3. Does the proposed amendment involve a significant reduction in a margin of safety?

Response: No.

Although the proposed change is made specifically to support the upgrade of the KPS AB crane from a non-single-failure-proof to a single-failure-proof design, the margin of safety under consideration in this evaluation is mainly based on that contained within the safety analysis (seismic analysis).

The purpose of this methodology is to determine the stress placed on the AB cranes' structural components. The stresses determined by this methodology are then compared to the yield strength values contained in CMAA-70. If the stresses the structural component are analyzed to receive during a postulated seismic event are less than the values contained in CMAA-70 the structural integrity of the crane is maintained and a suspended load will remain suspended during a seismic event. Additional margin has been added by reducing the analysis acceptance criteria to 90% of the acceptance criteria values contained in CMAA-70, modifying the crane support structure through additional welds and material, and confirming the bolts are of the proper material.

DEK is modeling the AB crane to roll during a seismic event when the postulated forces exceed the brake holding force. This provides a more realistic approach because the crane trolley is not fixed to the bridge rails. DEK has provided additional conservatisms by doubling the calculated force needed to overcome the brake holding force.

Therefore, the proposed amendment does not involve a significant reduction in a margin of safety.

5.2 Applicable Regulatory Requirements/Criteria

The U.S. Atomic Energy Commission (AEC) issued their Safety Evaluation (SE) of the Kewaunee Power Station on July 24, 1972 with supplements dated December 18, 1972 and May 10, 1973. Section 3.1, "Conformance with AEC General Design Criteria," of the AEC's SE described the conclusions the AEC reached associated with the General Design Criteria in effect at the time. The AEC SE stated:

> *"The Kewaunee plant was designed and constructed to meet the intent of AEC's General Design Criteria, as originally proposed in July 1967. Construction of the plant was about 50% complete and the Final Safety Analysis Report (Amendment 7) had been filed with the Commission before publication of the revised General Design Criteria in February 1971 and the present version of the criteria in July 1971. As a result, we did not require the applicant to reanalyze the plant or resubmit the FSAR. However, our technical review did assess the plant against the General Design Criteria now in effect and we are satisfied that the plant design generally conforms to the intent of these criteria."*

As such, the appropriate General Design Criteria KPS is licensed to from the Final Safety Analysis Report (Amendment 7), which has now been updated and is entitled the Updated Safety Analysis Report (USAR), are listed below.

5.3 Kewaunee Design Criteria

Criterion 2 – Performance Standards

Those systems and components of reactor facilities which are essential to the prevention of accidents which could affect the public health and safety or to mitigation of their consequences shall be designed, fabricated, and erected to performance standards that will enable the facility to withstand without loss of the capability to protect the public. The additional forces that might be imposed by natural phenomena include those such as earthquakes, tornadoes, flooding conditions, winds, ice, and other local site effects. The design bases so established shall reflect:

a) appropriate consideration of the most severe of these natural phenomena that have been recorded for the site and surrounding area, and

b) an appropriate margin for withstanding forces greater than those recorded to reflect uncertainties about the historical data and their suitability as the basis for design.

5.4 Precedent

The proposed change to incorporate rolling/sliding in the seismic analysis methodology is similar to a seismic analysis method approved for the Diablo Canyon Nuclear Power Station in NUREG-0675 supplement 9 (reference 15). In this NUREG supplement, Section 9.1, "General – Cranes," there is a discussion about the intake structure crane analysis. The discussion states that sliding of a few inches along the tracks can be expected which serves to prevent development of sufficient overturning moments to cause instability, this is not the case when the crane is parked at the end of its travel preventing sliding. Similarly, DEK is requesting approval to use rolling/sliding in its seismic analysis for the AB crane.

In conclusion, based on the considerations discussed above, (1) there is reasonable assurance that the health and safety of the public will not be endangered by operation in the proposed manner, (2) such activities will be conducted in compliance with the Commission's regulations, and (3) the issuance of the amendment will not be inimical to the common defense and security or to the health and safety of the public.

6.0 ENVIRONMENTAL CONSIDERATION

A review has determined that the proposed amendment would change a requirement with respect to installation or use of a facility component located within the restricted area, as defined in 10 CFR 20, or would change an inspection or surveillance requirement. However, the proposed amendment does not involve (i) a significant hazards consideration, (ii) a significant change in the types or a significant increase in the amount of any effluent that may be released offsite, or (iii) a significant increase in the individual or cumulative occupational radiation exposure. Accordingly, the proposed amendment meets the eligibility criterion for categorical exclusion set forth in 10 CFR 51.22(c)(9). Therefore, pursuant to 10 CFR 51.22(b), no environmental impact statement or environmental assessment need be prepared in connection with the proposed amendment.

7.0 REFERENCES

1. NUREG-0612, "Control of Heavy Loads at Nuclear Power Plants," dated July 1980.
2. NUREG-0554, "Single Failure Proof Cranes for Nuclear Power Plants," dated May 1979.
3. NRC Bulletin 96-02, "Movement of Heavy Loads Over Spent Fuel, Over Fuel in the Reactor Core, or Over Safety-Related Equipment," dated April 11, 1996.

4. NRC Regulatory Issue Summary 2005-25, "Clarification of NRC Guidelines for Control of Heavy Loads," dated October 31, 2005; and Supplement 1 dated May 29, 2007.

5. ASME NOG-1, "Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder)," 2004 Edition.

6. NUREG-0800, "Standard Review Plan," Section 9.1.5, "Overhead Heavy Load Handling Systems," Revision 1.

7. Crane Manufacturers Association of America (CMAA) Specification 70, "Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes," 2004.

8. John A. Blume and Associates Report to Pioneer Service & Engineering Company, "Kewaunee Nuclear Power Plant Earthquake Analysis: Reactor-Auxiliary-Turbine Building Response Acceleration Spectra," April 1971.

9. "Response of Sliding Structures to Earthquake Response Motion," N. Mostaghel and J. Tanbakuchi, Earthquake Engineering and Structural Dynamics, Volume II, 729-748, Copyright, John Wiley and Sons, Ltd., 1983.

10. DIR-20776-SE-001, "Dominion Energy Kewaunee Auxiliary Building Crane Upgrade - Structural, Seismic, and Design Basis Accident (DBA) Design Requirements," American Crane and Equipment Company, Revision 2.

11. CAL-20776-SE-001, "125/12-Ton Auxiliary Building Crane Trolley Structural and Seismic Design," American Crane and Equipment Company, Revision 2.

12. NRC Regulatory Guide 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," Revisions 1 and 2.

13. Electrical Overhead Crane Institute (EOCI) EOCI-61.

14. American Welding Society (AWS) D1.1, "Structural Welding Code," – 2004.

15. NUREG-0675, Supplement 9, "Safety Evaluation Report Related to the Operation of Diablo Canyon Nuclear Power Station Units 1 and 2," June 1980.

ATTACHMENT 2

LICENSE AMENDMENT REQUEST 234
REQUEST FOR REVIEW AND APPROVAL OF METHODOLOGY CHANGE REGARDING AUXILIARY BUILDING CRANE UPGRADE

USAR MARK-UP PAGES FOR LICENSE AMENDMENT REQUEST 234

(For Information Only)

9.5-9
9.5-10
B-20
B-21
B-29
B-41
B-63
B-79
B-80
B-81
B-91

KEWAUNEE POWER STATION

DOMINION ENERGY KEWAUNEE, INC.

Suitable restraints are provided between the bridge and trolley structures and their respective rails to prevent derailing. The manipulator crane is designed to prevent disengagement of a fuel assembly from the gripper in the event of a Design Basis Earthquake.

9.5.2.11 Spent Fuel Pool Bridge

The spent fuel pool bridge is a gantry crane spanning the spent fuel pool, which carries electric monorail hoists on an overhead structure. The fuel assemblies are moved within the spent fuel pool by means of a long-handled tool suspended from the hoist. The hoist travel and tool length are designed to limit the maximum lift of a fuel assembly to a safe shielding depth.

9.5.2.12 Auxiliary Building Crane

The Auxiliary Building (AB) crane, which is used for handling spent fuel shipping and transfer casks, is designed to minimize the possibility of dropping such a cask. The AB crane was upgraded from its original design by replacing the trolley with a single-failure-proof design and modifying the existing crane bridge. The replacement AB crane trolley, including new main and auxiliary hoists, is constructed to meet the guidance of NUREG-0612, Section 5.1.6 and NUREG-0554. The design, fabrication, inspection, and testing of the AB crane trolley is in accordance with NUREG-0554, Crane Manufacturer's Association of America Specification 70 (CMAA-70), 2004 Edition, and, in areas where NUREG-0554 or CMAA-70 does not provide guidance, ASME NOG-1-2004. The ~~fuel handling~~ existing AB crane bridge was~~has been~~ designed, fabricated, and qualified in accordance with the Electric Overhead Crane Institute Standard No. 61, American National Standard Institute Standard B-30.2.0, 1967 Edition (and Pioneer Service and Engineering Company Standard Specification for Powerhouse Overhead Electrical Traveling Cranes). The design rated load and maximum critical load on the AB crane main hook and cable hoist is 125 tons. The design rated load and maximum critical load on the auxiliary hoist is 15 tons. All AB crane structural members have been designed to withstand impact loads per applicable specifications. A seismic evaluation has been performed for the loaded condition for both the main and auxiliary hoist hooks. Numerous safety features have been incorporated in the design of the AB crane. Among these are the following:

1. design of the hoist cables incorporates a design safety factor of five, based on the rated load and efficiency of lifting tackle. Both the main and auxiliary hoists include dual reeving systems. Each dual reeving system has a minimum design safety factor of five, based on the maximum critical load for the respective hoist, to provide an overall design safety factor of ten for the maximum critical load~~.~~;
2. all parts subjected to dynamic forces, such as gears, shafts, drums, blocks and other integral parts, have a minimum design safety factor of five unless otherwise specified by the applicable design specification, codes and standards. Single-failure-proof components are

provided with a design safety factor of ten based on the maximum critical load for that load path, as required by the design specification;

3. ~~two separate magnetic holding brakes are provided as well as an eddy-current control brake. Each magnetic brake provides a braking force in excess of 150 percent of rated load. The eddy-current brake assures that smooth lowering and hoisting speeds can be maintained regardless of the load on the hook and that the load can be safely lowered even if both magnetic brakes fail~~the main hoist is provided with two holding brakes located on the high-speed shafting and one emergency braking system that applies stopping force directly to the drum flanges. The auxiliary hoist is provided with two holding brakes located on the high-speed shafting. Each brake on both hoists has a minimum static torque rating of 125 percent of the full rated load hoisting torque at the point of brake application. The main and auxiliary hoist control braking is provided by the respective flux vector frequency drives, which are sized to provide a minimum braking torque equal to 150 percent of full rated motor torque. The hydraulic drum brakes on the main hoist and the holding brakes on the auxiliary hoist can be manually modulated to lower the rated load in the event of a hoisting equipment failure~~.~~;

4. the crane is capable of raising, lowering and transporting occasional loads of 125 percent of rated load without damage or distortion to any crane part;

5. ~~the crane is provided with a type-302 stainless steel cable with a high-strength steel core. The cable is made up of 6 strands consisting of 37 wires to each strand.~~ The AB crane main and auxiliary hoists are provided with high-strength lubricated carbon steel wire ropes in accordance with the guidance in NUREG-0554 and CMAA-70 (2004)~~.~~;

6. during acceptance inspection, the main and auxiliary hooks w~~as~~ere subjected to a 200 percent overload test followed by magnetic particle inspection;

7. motor controllers with at least five uniformly proportional steps for control in each direction are provided for all crane motions. Built-in time delays are provided between steps. The main and auxiliary hoists employ infinitely variable speed frequency drive controls and the bridge and trolley employ variable speed frequency drive controls. This will provide a smooth uniform acceleration, which eliminates sudden jerking of the load and other components~~eask~~;

8. a fail-safe remote radio control system is provided for the crane. A selector switch determines whether the radio controls or master pendant controls are used. All of the safety features built into the control system apply when either the radio transmitter or master control is used. The radio has a system for transmitting and receiving signals so it is not credible to duplicate this signal by other means. The signals used are different for every crane, so the transmitter for other cranes will not actuate the receiver panel for this crane.

B.3 DESIGN CODES

The design and construction of this plant has been in accordance with the following codes and regulatory guidance, as applicable:

1. American Concrete Institute Code ACI 318-63
2. American Institute of Steel Construction "Specification for the Design, Fabrication and Erection of Structural Steel Buildings," 1963 Edition[3]
3. American Welding Society Code D 1.0 "Standards for Arc and Gas Welding in Building Construction"
4. International Conference of Building Officials "Uniform Building Code," 1967 Edition
5. Atomic Energy Commission publication TID 7024 "Nuclear Reactors and Earthquakes"
6. American Society of Mechanical Engineers "Boiler and Pressure Vessel Code," Section III, Nuclear Vessels, 1968 Edition through Summer 1968 Addenda[4]
7. Piping Code, USAS B31.1.0-1967 with applicable N-code cases to ASA B31.1-1955[5] [6]
8. Welding Research Council Bulletin No. 107, 1965 Edition
9. Wisconsin Administrative Code: "Rules of Department of Industry, Labor & Human Relations"
10. Crane Manufacturers Association of America Specification 70, "Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes," 2004 Edition.
11. ASME NOG-1, "Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder)," 2004 Edition.

---

[3]A later edition may be used for plant physical changes provided appropriate reconciliation is documented.

[4]Replacement Steam Generator lower units are designed and manufactured to ASME III, Division 1, Subsection NB, Class 1, 1986 Edition through 1987 Addenda. The steam domes were designed to ASME III, Class C, 1965 Edition through Summer 1966 Addenda. They have been reanalyzed for a 40-year design life (beginning December 2001) in accordance with the design code of the replacement lower units.

[5]An alternative Design Code to USAS B31.1 is ASME Section III (Post 1980 Editions Approved by NRC, reference Table B.7-6).

[6]During RFO 28 tubing for penetrations 1, 3, 21, 27E, 27EN, 27N, 27NE, and 36, located between containment and the shield building, was analyzed to ASME Section III, reference Table B.7-6. Analyses were performed to reconcile thermal stresses that may occur during sampling and differences in displacement of the containment and shield buildings due to annual temperature variations and periodic ILRT testing.

12. Electrical Overhead Crane Institute (EOCI) Standard 61.

13. NUREG-0612, "Control of Heavy Loads at Nuclear Power Plants," dated July 1980.

14. NUREG-0554, "Single Failure Proof Cranes for Nuclear Power Plants," dated May 1979.

These criteria, as applied to tornado winds, and to the DBA condition in combination with DBE loads, will permit some permanent deformation but will not permit loss of structural function. In this sense, structural function is defined to mean that structures will remain intact and continue to support their normal operating loads after an earthquake and/or DBA, but may require repair or replacement for future continued use.

Tornado missiles may result in large local deformations, but the criteria will not permit the missile to breach the barrier so that essential safety features functions are jeopardized.

B.6.3 Structural Design Basis

Class I Structures

The designs of Class I structures for seismic, tornado winds, tornado missiles, etc., are given in subsequent paragraphs.

Seismic Design

For dynamic analysis, an equivalent multi-mass mathematical model was constructed to approximate the structural system. The effect of the foundation soils was included in the model by means of equivalent springs. The spectral method was then used to determine the maximum response of each mass point for each ~~n~~mode, using as input the OBE (Plate 8-A in Appendix A) and the damping factors given in Table B.6-5. The total response for each point was determined by the square-root-~~mean~~of-the-sum-of-squares (SRSS) method. From this, a set of curves was developed which show the variation with height of the maximum translational accelerations, displacements, shears and moments in the structure. All of the above work was performed by John A. Blume and Associates and is reported in detail in a separately submitted Topical Report JAB-PS-01(s) (hereinafter referred to as "The Blume Report"). Vertical acceleration equal to two-thirds the horizontal ground acceleration was applied to the structure.

Operational Basis Earthquake

Using the data presented in the Blume Report, stresses were computed for the various parts of the plant structures. The stresses resulting from both horizontal and vertical acceleration were combined to obtain the total earthquake stresses. Earthquake stresses were then added linearly and directly to stresses caused by DBA, snow, dead loads, and the appropriate operating loads to obtain the total stresses. The total stresses were reviewed to ensure that they were within the maximum stress limits as established in Table B.6-2 and B.6-3. Direct superposition of stresses has been used for all loads except missile impact and contact points of pipe rupture restraints.

Table B.6-5
Damping Factors

Item	Percent of Critical Damping*
Reactor Containment vessel	1.0
Shield Building	2.0
Reactor containment vessel internal concrete	5.0
Steel frame structures	2.0
Reinforced concrete construction	2.0
Piping systems	0.5
Electrical and mechanical equipment evaluated in accordance with the Blume Report (John A. Blume & Associates, Engineers, "Kewaunee Nuclear Power Plant-Earthquake An)	1.0
Foundation soils	5.0
Electrical and mechanical equipment evaluated in accordance with the SQUG GIP**	5.0
Auxiliary Building Crane	***

* The maximum percent of critical damping factors given is applied to both the OBE and the DBE.

** See Seismic Design and Verification of Modified, New and Replacement Equipment.

*** Damping is 1% for vertical acceleration per the Blume Report and 7% for horizontal acceleration per ASME NOG-1-2004, Section 4153.8.

Note: At and below the mezzanine floor level, the Shield Building, Auxiliary Building, and Containment System are interconnected so as to comprise a monolithic structure. The many shear walls below this level in the Auxiliary Building, the grout under the Reactor Containment Vessel, and the shear walls in the Containment System all combine to form a very stiff connection between the Basement level and the Mezzanine level. For this reason, the mathematical model used for the dynamic analysis of these buildings considers that they are rigid between these two levels. Above mezzanine floor level these concrete buildings are not interconnected and the individual damping values are used (i.e., 5 percent for Auxiliary Building, 2 percent for Shield Building, and 5 percent for Reactor Containment Vessel internal concrete construction).

Table B.7-1
Load Combinations For Components
Class Of Components

Condition of Loading	Class I[1, 2]	Class I*[3]	Classes II and III*	Class III

1. The replacement steam generator lower units are designed and analyzed to loading combinations defined in Design Specification 414A03, consistent with ASME Code, Section III, Division 1, Subsection NB, Class 1, 1986 Edition through 1987 Addenda. The original steam domes are analyzed in the same manner as the replacement lower units.
2. The replacement reactor vessel head is designed and analyzed to loading combinations defined in WCAP-16237-P, Rev 1, Addendum 2, consistent with ASME Code, Section III, Division 1, Subsection N.3, Class 1, 1998 Edition through 2000 Addenda. This methodology is approved by NRC for application to KPS under Amendment 172, dated February 27, 2004.
3. The upgraded Auxiliary Building crane is also designed to withstand two-blocking, load hang-up, and broken wire rope without an uncontrolled lowering of the load, in accordance with NUREG-0554.

B.8 PROTECTION AGAINST CRANE TOPPLING AND CONTROL OF HEAVY LOADS

The Auxiliary Building crane and the Turbine Building crane are located in areas where they are subject to possible damage from tornado and earthquake. These crane bridges and trolleys are protected against tipping, derailment, and uncontrolled movements that could possibly create damage.

To assure stability of the crane, the bridge and trolley are equipped with fixed, fitted rail yokes that allow free rolling movement but prevent the wheels from being lifted or derailed. The bridge and trolley wheels are equipped with electrically activated, spring set brakes. Upon loss of power or when the crane or trolley are not under operator control, the springs activate the brakes, locking the wheels firmly in place to prevent rolling out of position. The positive wheel stops and bumpers provided to prevent over-travel of the trolley and bridge will prevent the trolley and bridge from leaving the rails, even in the unlikely event of brake failure.

As a result of Generic Task A-36, "Control of Heavy Loads Near Spent Fuel," the NRC issued NUREG-0612, "Control of Heavy Loads at Nuclear Power Plants." NUREG-0612 was to be implemented in two phases. Phase I addressed Section 5.1 of NUREG-0612 and established seven basic guidelines for all nuclear power plants, which detailed provisions for the handling of heavy loads in the area of the reactor vessel near stored spent fuel, in other areas where an accidental load drop could damage equipment required for safe shutdown or decay heat removal. Phase II (Sections 5.1.2 through 5.1.6) was intended to cover the need for electrical interlocks/mechanical stops, or alternatively, single failure-proof cranes or load drop analyses.

In Letter from H. L. Thompson (NRC) to Licensees, Letter No. K-85-132, June 28, 1985 the NRC concluded that satisfaction of the Phase I guidelines would provide adequate assurance that, due to improvements in heavy load handling procedures and training, and crane and handling tool inspection and testing, the potential for a load drop is extremely small. Letter from H. L. Thompson (NRC) to Licensees, Letter No. K-85-132, June 28, 1985 also included a cost-benefit analysis for Phase II of NUREG-0612 which concluded that, because of the reduced potential of a load handling accident provided by Phase I, the high cost of implementing Phase II could not be justified by the comparatively small associated increase in plant safety.

Therefore, since Kewaunee has satisfied the requirements of Phase I and since Phase II compliance is no longer required, the NRC has determined that Kewaunee has adequately addressed NUREG-0612 and has significantly reduced the probability of a heavy load handling accident to an acceptably small value (see NRC SER in NRC SER, S. A. Varga (NRC) to C.W. Giesler (WPS), Letter No. K-84-61, March 16, 1984).

The Auxiliary Building (AB) crane was upgraded in support of dry spent fuel storage cask loading operations. This upgrade involved the replacement of the original trolley with a single-failure-proof design, replacement of the trolley controls, and an upgrade to the existing AB crane bridge. The upgrade of the AB crane meets the guidance in Section 5.1.6 of NUREG-0612, "Control of Heavy Loads at Nuclear Power Plants," and NUREG-0554, "Single Failure Proof Cranes for Nuclear Power Plants," as applicable.

Design Criteria for Upgraded Auxiliary Building Crane

The AB crane is designated as Class I* per Table B.2-1 and therefore is designed to meet Class I seismic standards. The crane is designed to stay on its rails and not allow an uncontrolled lowering of the load as a result of a seismic event. It is not required to be operational during or after a seismic event. The AB crane is also designed to withstand the crane design basis accident events described in NUREG-0554: two-blocking, load hang-up, and wire rope failure.

Because the replacement AB crane trolley is a new component and the crane bridge is an existing component, the construction codes applicable to the two are not identical. The construction codes for the trolley and bridge are as follows:

AB Crane Trolley Codes and Standards

Construction is in accordance with NUREG-0554 and, where NUREG-0554 does not offer specific guidance (e.g., normal condition load combinations and stress acceptance criteria), construction is in accordance with Crane Manufacturers Association of America Specification 70 (CMAA-70), 2004 Edition is used. Seismic load combinations and stress analysis acceptance criteria, as well as guidance used to address two blocking, load hang-up, and wire rope failure are taken from ASME NOG-1-2004.

AB Crane Bridge Codes and Standards

Construction is in accordance with NUREG-0554 and Electrical Overhead Crane Institute Standard 61 (EOCI-61). CMAA-70 (2004) and ASME NOG-1-2004 are used, in that hierarchy, where NUREG-0554, and EOCI-61 do not offer specific construction guidance.

Upgraded AB Crane Seismic Response Spectra, Damping, and Accelerations

The Blume Report, which forms the basis for seismic analyses at the Kewaunee Power Station, does not include horizontal response spectra data for a mass point at the location of the AB crane rail, appropriate for use in analyzing the upgraded crane. Therefore, a lumped-mass stick model of the Auxiliary Building steel structure was used to generate additional horizontal response spectra applicable for use at the AB crane rail. These are referred to as the "extended Blume spectra."

In accordance with ASME NOG-1-2004, Section 4153.8, 7 percent damping for the Safe Shutdown Earthquake[8] condition was applied to the horizontal response spectra at the crane rail elevation. Because the vertical response spectra developed in the Blume Report are applicable for all higher elevations in the Reactor-Auxiliary-Turbine Building complex, 1 perecent damping is applied in the vertical direction. For the seismic directional combination, the two-dimensional absolute sum method is employed and the more conservative combination of vertical with east-west or vertical with north-south is used, as applicable. For close mode combinations, the absolute sum addition method, per ASME NOG-1-2004 was applied for conservatism.

Rolling [Sliding] of the AB crane and bridge wheels was considered in the seismic analysis. The method used to consider crane wheel sliding is consistent with that described in the Mostaghel and Tanbakuchi paper published in 1983 in "Earthquake Engineering and Structural Dynamics" (Reference 9). The methodology in this paper provides guidance for simulating the non-linear time-history behavior of sliding with a conventional linear analysis method.

B.9 TURBINE MISSILE EFFECTS

B.9.1 General Discussion

In the original analysis on the effects of turbine-disc missiles by Westinghouse and based on information contained in Westinghouse's "Report Covering the Effects of a High Pressure Turbine Rotor Fracture and Low Pressure Turbine Disc Failures at Design Overspeed" ("Report Covering the Effects of a High Pressure Turbine Rotor Fracture and Low Pressure Tu). Five of the six discs were contained when the overspeed was limited to 120 percent of the rated speed. A more recent analysis by Westinghouse ("Methodology for Calculating the Probability of a Missile Generation from Rupture of a Lo, "Results of Probability Analyses of Disc Rupture and Missile Generation," Revision 0, Au) shows that only one of the six discs is contained when the overspeed was limited to 120 percent of the rated speed. Su is a summary of the exit missile properties for both the original analysis and the revised analysis. These analyses are for the original LP2 rotor.

The Number 2, 3, 4 and 5 discs are assumed to exit as four quadrants in any direction radially, 90 degrees apart, and ± 5 degrees from the vertical radial plane. The weights, exit velocities, and exit energies are listed in Su. The impact velocity varies with the height of the impact point above the source of the missile. Analyses for the effect of this missile are based on a 90-degree impact angle.

The missile trajectory and the effects of the turbine missiles on Class I structures and components described below pertain to the original turbine missile design parameters from

[8] Safe Shutdown Earthquake (SSE) is considered synonymous with design basis earthquake (DBE) when applying ASME NOG-1 rules.

25. Seismic Qualification Utility Group (SQUG), "Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Power Plant Equipment," Revision 3, May 16, 1997

26. Supplemental Safety Evaluation Report No. 3 (SSER No. 3) on the Review of Revision 3 to the Generic Implementation Procedure for Seismic Verification of Nuclear Power Plant Equipment updated May 16, 1977, (GIP-3), (TAC No. M93624)

27. "Response of Sliding Structures to Earthquake Response Motion," N. Mostaghel and J. Tanbakuchi, Earthquake Engineering and Structural Dynamics, Volume II, 729-748, Copyright, John Wiley and Sons, Ltd. 1983

ATTACHMENT 3

LICENSE AMENDMENT REQUEST 234
REQUEST FOR REVIEW AND APPROVAL OF METHODOLOGY CHANGE
REGARDING AUXILIARY BUILDING CRANE UPGRADE

"RESPONSE OF SLIDING STRUCTURES TO EARTHQUAKE RESPONSE MOTION,"
N. MOSTAGHEL AND J. TANBAKUCHI,
EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS, VOLUME II, 729-748

KEWAUNEE POWER STATION

DOMINION ENERGY KEWAUNEE, INC.

RESPONSE OF SLIDING STRUCTURES TO EARTHQUAKE SUPPORT MOTION

Revised 14 February 1983
Earthquake Engineering & Structural Dynamicx, Vol. 11, 729-748 (1983)

Terms and Conditions of Use

The following Terms and Conditions of Use dated as of August 2003, govern use by all Users of **Wiley InterScience**, an online service, and the electronic edition of the Products subscribed to by the User or the User's institution (the "Electronic Product(s)"). Please read the following carefully because by using **Wiley InterScience** you are agreeing to abide by these Terms and Conditions of Use.

Your institution may have subscribed to a License which permits you additional uses; contact your institution's librarian/administrator for more information.

Copyright

The entire contents of the Electronic Product, and each article and abstract and other material available through **Wiley InterScience**, are protected by copyright (unless otherwise indicated on the title page of the article). As a User, you have certain rights specified below; all other rights are reserved.

Rules of Use

A. To make **Wiley InterScience** and the Electronic Product useful, the following rules apply:

1. All Users, including Guest Users, will have access to all Content websites within **Wiley InterScience**. You may browse all Tables of Content and for all Electronic Product on **Wiley InterScience** regardless of what Electronic Product your institution subscribes to, and search the entire database of Tables of Content and abstracts. For most publications, you may browse the abstracts as well.
2. Users at institutions which have subscribed to Electronic Product and users who have access under a Society or Personal License will have access to the full text of such Electronic Product ("Authorized Users"). Authorized Users may download, view, copy and save to hard disk or diskette and store or print out single copies of individual articles or items for your own personal use, scholarly, educational or scientific research or internal business use. Authorized Users may transmit to a third-party colleague in hard copy or electronically, a single article or item from **Wiley InterScience** for personal use or scholarly, educational, or scientific research or professional use but not for re-sale. In addition, Authorized Users have the right to use, with appropriate credit, figures, tables and brief excerpts from individual articles in the Electronic Product(s) in your own scientific, scholarly and educational works.
3. All Users, including Guest Users, have the option to create a My Profile Page which will allow them to create links to Electronic Product (s), articles of interest and search criteria which may be reused by them and manage their custom e-mail alerting services. In order to do so, Users must register. During the registration process, Users will select and register a user name and password which they must keep confidential and not disclose to or share with anyone else.

B. Except as provided above, you may not copy, distribute, transmit or otherwise reproduce material from Electronic Products (s); store such material in any form or medium in a retrieval system; download and/or store an entire issue of a Electronic Product or its equivalent; or transmit such material, directly or indirectly, for use in any paid service such as document delivery or list serve, or for use by any information brokerage or for systematic distribution, whether or not for commercial or non-profit use or for a fee or free of charge.

C. In order to protect the integrity and attribution of **Wiley InterScience** and the Electronic Product(s), you agree not to remove, obscure or modify any copyright or proprietary notices, author attribution or disclaimer contained therein or on any screen display. You may not integrate material from the Electronic Product(s) with other material or otherwise create derivative works in any medium. This is not meant to prohibit quotations for purposes of comment, criticism or similar scholarly purposes.

D. You may not do anything to restrict or inhibit any other User's access to or use of **Wiley InterScience** and the Electronic Product (s).

E. If you refuse or fail to abide by these Terms and Conditions or violate any other terms of this Agreement, Wiley reserves the right in its sole discretion to suspend or terminate your access to **Wiley InterScience** and the Electronic Products immediately without notice, in addition to any other available remedies.

Additional Terms

Wiley is not responsible for any charges associated with accessing **Wiley InterScience** and the Electronic Products, including, but not limited to, any computer equipment, telephone or Internet connections and access software.

Wiley may modify any of these terms and conditions at any time by providing notice in **Wiley InterScience**. Your continued use of

Wiley InterScience shall be conclusively deemed acceptance of such modification.

Wiley InterScience may provide links to third-party websites. Where such links exist, Wiley disclaims all responsibility and liability for the content of such third-party websites. Users assume the sole responsibility for the accessing of third-party websites and the use of any content appearing on such websites.

Privacy and Data Protection Policy

John Wiley & Sons, Inc. and its related companies (collectively, "Wiley") recognize the importance of protecting the information collected from Users in the operation of this website, and take reasonable steps to maintain the security, integrity and privacy of any information in accordance with this Privacy and Data Protection Policy. By submitting your information to Wiley you consent to the practices described herein.

Wiley will ensure that any identifiable personal information it collects through, for example, the registration and purchasing processes, will be adequate, relevant and not excessive for purposes of operating this website or any of the services operated hereunder, it will be kept accurate and up-to-date based on information provided and it will be deleted when no longer needed. Except where necessary in connection with services provided by appropriate intermediaries, who will be required to comply with the confidentiality provisions of this policy, Wiley will not disclose any personal information identifying Users to any third party, unless required by law or to enforce the **Wiley InterScience** Terms and Conditions of Use or otherwise needed in connection with legal proceedings or unless we have received your consent.

If use of an electronic product, feature or service offered by Wiley on this website requires a User to provide a name and password or other personally identifiable information, the User will have the option to opt in or out of providing such information and using the product, feature or service. Any private communications or information supplied to Wiley by a User may be used by Wiley for internal business and research purposes and to help enhance and develop our websites but will not be shared with third parties unless we receive your consent.

Wiley may disclose to carefully chosen third parties navigational and transactional information in the form of anonymous, aggregate usage statistics (including "page views" on this website and the products herein) and demographics but only in forms that do not reveal a User's identity or confidential information, except as required by law, as needed in connection with legal proceedings, or unless we have received your consent.

Links may be provided from this website to other third party websites. In using such links, Users should be aware that each third party website will vary in terms of its privacy and data protection policies, and Wiley does not take responsibility for the privacy policies of, and usage of, personal information collected by others including, without limitation, those of any partner or affiliate.

You should be aware that identifiable personal information--such as your name or e-mail address--that you voluntarily disclose that are accessible to other Users (e.g. on bulletin boards or in chat areas) could be collected and disclosed by others. Wiley cannot take any responsibility for such collection and disclosure.

Please note that Wiley's Privacy and Data Protection Policy is reviewed periodically. Wiley reserves the right to modify its Privacy and Data Protection Policy at any time without notice. Any changes to the policy will be posted on this page. Your continued use of this website will be deemed conclusive acceptance of such modification. Any comments or questions concerning this policy should be addressed to **Wiley InterScience** Privacy Policy at privacy@wiley.com. If at any point you wish to access your personal information to (1) change your preferences, (2) review the accuracy, or (3) correct, supplement or modify your information, you may either send an e-mail to the address set forth above, make in a written request to John Wiley & Sons, Inc., 111 River Street, Hoboken, NJ 07030, or update the information directly via the My Profile section of **Wiley InterScience**.

Additional Services: In addition **Wiley InterScience** may offer additional services, including optional E-mail Alerting Services, which are opted into by registration and may be opted out of at any time. By choosing to take advantage of this and other services offered by **Wiley InterScience**, you consent to the use of your information for the purposes of providing these services.

Use of Cookies:. A cookie is a small piece of information that a website puts on a hard drive so that it can remember information about a user at a later date. In **Wiley InterScience** cookies are used in two ways: as a temporary, Session Control cookie to validate access privileges and, at the user's option, as a permanent cookie to facilitate login to **Wiley InterScience** via the Remember Me option. Cookies are not used to gather or provide usage statistics. By choosing to take advantage of these services, you consent to the use of your information for the purposes of providing these services.

Session Control Cookie: Session control cookies are used on most Web sites today because they facilitate navigation. In order to use **Wiley InterScience** users must have their browsers set to accept cookies.

Once a user is signed onto **Wiley InterScience**, our Web server uses a temporary cookie to help us manage the visit to the service. This Session Control Cookie is deleted when the user clicks on the Logout button in **Wiley InterScience** or when the user quits the browser. This cookie allows us to quickly determine access control rights and personal preferences during the online session. The Session Control Cookie is set out of necessity and not out of convenience.

Remember Me Cookie: When a user registers to gain access to **Wiley InterScience** or its personalization features, whether as a Guest User or as an Authorized User from a subscribing institution, he is asked to choose a user name and password. Every time the user logs into **Wiley InterScience**, he/she will be asked to enter this unique user name and password. Users have the option of storing this information on their computers as a permanent cookie by selecting the 'Remember Me' option when logging in. This feature is optional and user controlled. The Remember Me cookie can be deleted at any time by clicking on the Logout button in **Wiley InterScience**.

Users must be careful not to store their user name and password as a cookie on any computer that is not their own, since doing so would enable other users coming to that computer to access **Wiley InterScience** under their name and to have access to their **Wiley InterScience** My Profile page.

Warranty Limitations

WILEY INTERSCIENCE AND THE ELECTRONIC JOURNALS AND ALL MATERIALS CONTAINED THEREIN ARE PROVIDED ON AN "AS IS" BASIS, WITHOUT WARRANTIES OF ANY KIND, EITHER EXPRESS OR IMPLIED, INCLUDING, BUT NOT LIMITED TO, WARRANTIES OF TITLE, OR IMPLIED WARRANTIES OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE; THE USE OF THE ELECTRONIC JOURNALS, WILEY INTERSCIENCE AND ALL MATERIALS IS AT THE USER'S OWN RISK; ACCESS TO WILEY INTERSCIENCE AND THE ELECTRONIC JOURNALS MAY BE INTERRUPTED AND MAY NOT BE ERROR FREE; AND NEITHER WILEY INTERSCIENCE, THE ELECTRONIC JOURNALS, WILEY, OR ANYONE ELSE INVOLVED IN CREATING, PRODUCING OR DELIVERING THE ELECTRONIC JOURNALS OR THE MATERIALS CONTAINED IN WILEY INTERSCIENCE, SHALL BE LIABLE FOR ANY DIRECT, INDIRECT, INCIDENTAL, SPECIAL, CONSEQUENTIAL OR PUNITIVE DAMAGES ARISING OUT OF THE USER'S USE OF OR INABILITY TO USE WILEY INTERSCIENCE, THE ELECTRONIC JOURNALS AND ALL MATERIALS CONTAINED THEREIN.

EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS, VOL. 11, 729–748 (1983)

RESPONSE OF SLIDING STRUCTURES TO EARTHQUAKE SUPPORT MOTION

N. MOSTAGHEL* AND J. TANBAKUCHI†

Department of Civil Engineering, The University of Utah, U.S.A.

SUMMARY

To study the effectiveness of sliding supports in isolating structures from damaging earthquake ground motions, a mathematical model of a single degree of freedom structure supported on a sliding foundation and subjected to the N–S component of the El Centro 1940 earthquake is considered. Spectra for absolute accelerations, relative displacements, relative-to-ground displacements, sliding displacements and residual sliding displacements are evaluated for three mass ratios, four coefficients of friction and a damping of 5 per cent critical. It is observed that, for structures with periods less than 1·8 s, for the coefficients of friction considered, the suprema of relative-to-ground displacements, sliding displacements and residual sliding displacements are only of the order of 1·25 times the peak ground displacement. To study the response sensitivities, the spectra for absolute acceleration and sliding displacement of the 1949 Olympia earthquake (S86E component) are also presented. It is concluded that sliding supports can be quite effective in isolating structures from support excitations.

INTRODUCTION

A promising scheme of protecting structures against damaging earthquake ground motion is base isolation. Base isolators are devices which are incorporated in the structural support and essentially control the level of excitations which are transmitted to the superstructure. Besides the classical method,[1] there are many proposed isolation schemes such as the flexible first storey concept,[2–4] the soft storey concept[5,6] and specially shaped rollers.[7,8] Considerable work has been done to show the effectiveness of steel plate laminated rubber bearings and their variants,[9–17] and there are several structures which have been isolated using this scheme.[18]

Sliding structures, whose response to earthquake base excitations is the topic of this paper, are structures which can slide on their supports. The maximum ground accelerations that can be transmitted to the superstructures are controlled by the coefficient of sliding friction at their supports. Although, due to rigid plastic behaviour of the sliding support, the system is non-linear, in each sliding and non-sliding phase the system's behaviour is linear. The conditions for the determination of the transition point between any two phases are presented. These points are obtained as the solution process progresses. The responses at the end of each phase are used as initial conditions for the next phase. Through this matching technique, the non-linear problem is transformed to linear ones which are solved analytically for each phase.

The above solution scheme has been verified by the use of harmonic excitations for which closed-form solutions for sliding and non-sliding phases are available.[19]

A single degree of freedom system on top of a foundation raft is supported on the sliding element (see Figure 1). The base of the structure is subjected to the (N–S) component of the El Centro 1940 earthquake. Considering a damping of $\xi = 5$ per cent of critical, normalized response spectra (normalized with respect to the corresponding peaks of the input excitations) are calculated and plotted for absolute accelerations, relative displacements, sliding displacements relative to ground displacements and cumulative sliding displacements for coefficients of friction $\mu = 0{\cdot}05, 0{\cdot}10, 0{\cdot}15, 0{\cdot}20$ for three mass ratios $\alpha = m/(m+M) = 0{\cdot}75$,

* Professor of Civil Engineering.
† Research Assistant.

0098–8847/83/060729–20$02.00
© 1983 by John Wiley & Sons, Ltd.

Received 21 October 1982
Revised 14 February 1983

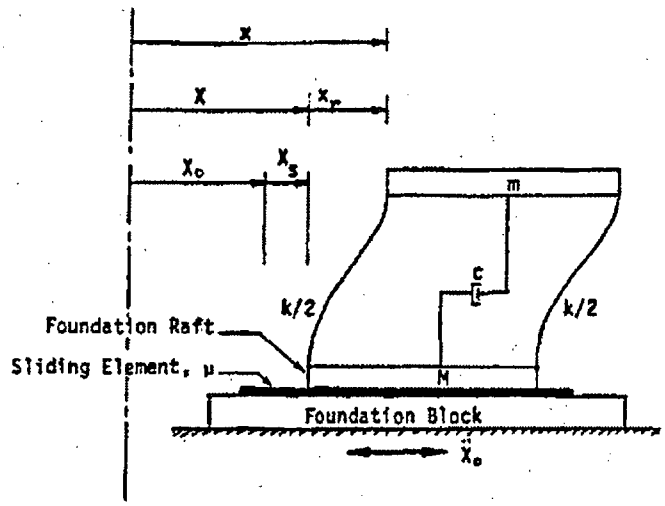


Figure 1. Single degree of freedom structure on sliding support

0·50, 0·25 (implying $m/M = 3$, 1 and $\frac{1}{3}$) where m is the mass of the structure and M is the mass of the foundation raft. As expected, these plots show that both the absolute acceleration and the relative displacement of the superstructure are smaller for smaller coefficients of friction, and the cumulative sliding movement is larger for smaller coefficients of friction. The residual sliding displacement, that is, the dislocation of the structure from its original position, due to sliding, when the ground motion stops, is also plotted.

By comparing different plots, it is observed that the level of acceleration response of sliding structures, especially for larger mass ratios, is almost independent of the period of the input excitations and depends only on the base's coefficient of friction. It can be reduced considerably by reducing the base's coefficient of friction. Also, the maximum (supremum) sliding displacement, for all the structures and coefficients of friction considered, is about 1·25 times the peak ground displacement. To further elucidate the performance of sliding systems, the spectra for the absolute acceleration and sliding displacement for the 1949 Olympia earthquake (S86E component) are also presented. The effects of vertical ground motions are not considered in this study. Generally, structures have much greater reserve capacity for vertical forces than for horizontal ones, but vertical excitations may be consequential in design for overhangs and flexible girders. However, it is noted that the horizontal isolation, by reducing the relative displacement, reduces the P–Δ effect and in this way reduces the effects of vertical excitations significantly. The vertical ground motion can either be considered directly or its effects may be taken into account by using an effective coefficient of friction.[20, 21] In the following sections the details of formulations, solutions and results are presented.

FORMULATION

A single degree of freedom structure of mass m, damping c and stiffness k supported by a foundation raft that can slide horizontally is shown in Figure 1. The coefficient of sliding friction is μ. If the ground moves with an acceleration $\ddot{X}_0(t)$, the system moves, and applications of Newton's second law yield the equations of dynamic equilibrium, which are

$$m\ddot{\chi} + c\dot{\chi}_r + k\chi_r = 0 \tag{1}$$

$$M\ddot{X} = F - m\ddot{\chi} \tag{2}$$

where χ is the total displacement of mass m in absolute frame, χ_r is the displacement of the mass m relative to the foundation raft, X is the total displacement of the foundation raft in absolute frame, and F is the interface

force between the foundation raft and its support. The maximum value of F which occurs when the system is in a sliding phase is given by

$$F = \mu g(m+M)\,\varepsilon \tag{3}$$

where μ is the coefficient of sliding friction and

$$\varepsilon = \frac{\alpha\ddot{\chi}_r + \ddot{X}_0}{|\alpha\ddot{\chi}_r + \ddot{X}_0|} \tag{4}$$

As is noted from the above expression, ε can only be $+1$ and -1. Also from Figure 1,

$$\chi = \chi_r + X_s + X_0 \tag{5}$$

$$X = X_s + X_0 \tag{6}$$

$$\chi_r = \chi - X \tag{7}$$

Substitutions of the above relations into equilibrium equations (1) and (2) yield

$$\ddot{\chi}_r + 2\xi\omega\dot{\chi}_r + \omega^2\chi_r = -X_s - \ddot{X}_0 \tag{8}$$

$$\ddot{X}_s = \varepsilon\mu g - \alpha\ddot{\chi}_r - \ddot{X}_0 \tag{9}$$

where ξ is the percentage of critical damping, $\omega = \sqrt{(k/m)}$ and

$$\alpha = \frac{m}{m+M} \tag{10}$$

It should be noted that expression (9) is valid only in sliding phases.

Solution

In the interval $t_i \leqslant t \leqslant t_{i+1}$ the input ground acceleration $\ddot{X}_0(t)$ and its integrals, i.e. ground velocity $\dot{X}_0(t)$ and ground displacement $X_0(t)$, may be represented as follows (see Figure 2):

$$\ddot{X}_0(t) = \ddot{X}_0(t_i) + \alpha_i(t - t_i) \tag{11}$$

$$\dot{X}_0(t) = \frac{\alpha_i}{2}t^2 + \beta_i t + \dot{X}_0(t_i) \tag{12}$$

$$X_0(t) = \frac{\alpha_i}{6}t^3 + \frac{\beta_i}{2}t^2 + t\dot{X}_0(t_i) + X_0(t_i) \tag{13}$$

where

$$\alpha_i = \frac{\ddot{X}_0(t_{i+1}) - \ddot{X}_0(t_i)}{t_{i+1} - t_i} \tag{14}$$

$$\beta_i = \ddot{X}_0(t_i) - \alpha_i t_i \tag{15}$$

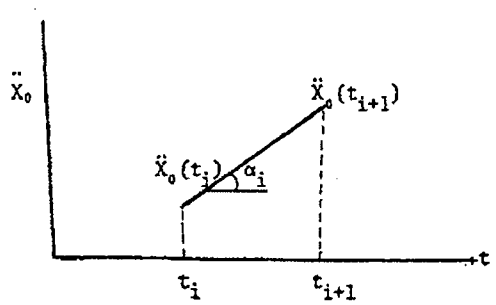


Figure 2. Ground motion representation in $t_i \leqslant t \leqslant t_{i+1}$

and $\ddot{X}_0(t_i)$ is the value of ground acceleration at time t_i. The solution of equations (8) and (9) involves two sets of phases, non-sliding and sliding.

(i) *Non-sliding phases*, $\bar{t}_j \leqslant t \leqslant \bar{t}_{j+1}$, $j = 1, 3, 5, \ldots, \bar{t}_1 = 0$. In these phases $\ddot{X}_s = \dot{X}_s = 0$. Therefore, equation (8) can be written as

$$\ddot{\chi}_r + 2\xi\omega\dot{\chi}_r + \omega^2\chi_r = -\ddot{X}_0 \tag{16}$$

for the non-sliding time intervals

$$\bar{t}_j \leqslant t < \bar{t}_{j+1}, \quad j = 1, 3, 5, \ldots, \bar{t}_1 = 0 \tag{17}$$

The $\bar{t}_j$'s represent the starting times of non-sliding and sliding phases, while the t_i's represent the times of digitization. The solution of equation (16) may be represented by

$$\chi_r(t) = \frac{-1}{\omega_d} S(t) + k(t) \tag{18}$$

$$\dot{\chi}_r(t) = -C(t) + \frac{\xi}{\sqrt{(1-\xi^2)}} S(t) + \dot{k}(t) \tag{19}$$

$$\ddot{\chi}_r(t) = -\ddot{X}_0(t) + \frac{\omega(1-2\xi^2)}{\sqrt{(1-\xi^2)}} S(t) + 2\xi\omega C(t) + \ddot{k}(t) \tag{20}$$

where ω_d is the damped natural circular frequency, ξ is the percentage of critical damping and

$$S(t) = \int_{\bar{t}_j}^{t} \ddot{X}_0(\tau)\, e^{-\xi\omega(t-\tau)} \sin\omega_d(t-\tau)\, d\tau = e^{-\xi\omega t}[A(t)\sin\omega_d t - B(t)\cos\omega_d t] \tag{21}$$

$$C(t) = \int_{\bar{t}_j}^{t} \ddot{X}_0(\tau)\, e^{-\xi\omega(t-\tau)} \cos\omega_d(t-\tau)\, d\tau = e^{-\xi\omega t}[A(t)\cos\omega_d t + B(t)\sin\omega_d t] \tag{22}$$

$k(t)$, $\dot{k}(t)$ and $\ddot{k}(t)$ represent the effects of initial conditions at the start of the non-sliding phases. They are defined by

$$k(t) = e^{-\xi\omega(t-\bar{t}_j)}\left[\frac{\dot{\chi}_r(\bar{t}_j) + \xi\omega\chi_r(\bar{t}_j)}{\omega_d}\sin\omega_d(t-\bar{t}_j) + \chi_r(\bar{t}_j)\cos\omega_d(t-\bar{t}_j)\right] \tag{23}$$

$$\dot{k}(t) = -\xi\omega k(t) + \omega_d e^{-\xi\omega(t-\bar{t}_j)}\left[\frac{\dot{\chi}_r(\bar{t}_j) + \xi\omega\chi_r(\bar{t}_j)}{\omega_d}\cos\omega_d(t-\bar{t}_j) - \chi_r(\bar{t}_j)\sin\omega_d(t-\bar{t}_j)\right] \tag{24}$$

$$\ddot{k}(t) = -(1-2\xi^2)\omega^2 k(t) - 2\xi\omega\omega_d e^{-\xi\omega(t-\bar{t}_j)}\left[\frac{\dot{\chi}_r(\bar{t}_j) + \xi\omega\chi_r(\bar{t}_j)}{\omega_d}\cos\omega_d(t-\bar{t}_j) - \chi_r(\bar{t}_j)\sin\omega_d(t-\bar{t}_j)\right] \tag{25}$$

For at rest initial conditions, at $\bar{t}_1 = 0$

$$\chi_r(\bar{t}_1) = \dot{\chi}_r(\bar{t}_1) = 0 \tag{26}$$

The quantities $A(t)$ and $B(t)$ are defined by

$$A(t) = \int_{\bar{t}_j}^{t} \ddot{X}_0(\tau)\, e^{\xi\omega\tau} \cos\omega_d\tau\, d\tau = -A_n(\bar{t}_j) + \sum_{n=1}^{m-1} A_n(t_{n+1}) + A_m(t) \tag{27}$$

$$B(t) = \int_{\bar{t}_j}^{t} \ddot{X}_0(\tau)\, e^{\xi\omega\tau} \sin\omega_d\tau\, d\tau = -B_n(\bar{t}_j) + \sum_{n=1}^{m-1} B_n(t_{n+1}) + B_m(t) \tag{28}$$

where

$$\left.\begin{aligned} -A_n(\bar{t}_j) &= \int_{\bar{t}_j}^{t_n} \ddot{X}_0(\tau)\, e^{\xi\omega\tau} \cos\omega_d \tau \, d\tau, \quad t_n \geqslant \bar{t}_j \\ A_n(t_{n+1}) &= \int_{t_n}^{t_{n+1}} \ddot{X}_0(\tau)\, e^{\xi\omega\tau} \cos\omega_d \tau \, d\tau \\ A_m(t) &= \int_{t_m}^{t} \ddot{X}_0(\tau)\, e^{\xi\omega\tau} \cos\omega_d \tau \, d\tau \end{aligned}\right\} \tag{29}$$

$B_n(\bar{t}_j)$, $B_n(t_{n+1})$ and $B_m(t)$ are defined similarly except that $\cos\omega_d\tau$ in expression (29) is replaced by $\sin\omega_d\tau$. Utilizing the ground motion representation given by expressions (11) and (12), it can be shown that for $t_i \leqslant t \leqslant t_{i+1}$

$$\begin{aligned} A_i(t) = \frac{e^{\xi\omega t}}{\omega}\Big\{&(\alpha_i t+\beta_i)(\xi\cos\omega_d t+\sqrt{(1-\xi^2)}\sin\omega_d t) \\ &+\frac{\alpha_i}{\omega}[(1-2\xi^2)\cos\omega_d t-2\xi\sqrt{(1-\xi^2)}\sin\omega_d t]\Big\} \\ -\frac{e^{\xi\omega t_i}}{\omega}\Big\{&(\alpha_i t_i+\beta_i)(\xi\cos\omega_d t_i+\sqrt{(1-\xi^2)}\sin\omega_d t_i) \\ &+\frac{\alpha_i}{\omega}[(1-2\xi^2)\cos\omega_d t_i-2\xi\sqrt{(1-\xi^2)}\sin\omega_d t_i]\Big\} \end{aligned} \tag{30}$$

$$\begin{aligned} B_i(t) = \frac{e^{\xi\omega t}}{\omega}\Big\{&(\alpha_i t+\beta_i)(\xi\sin\omega_d t-\sqrt{(1-\xi^2)}\cos\omega_d t) \\ &+\frac{\alpha_i}{\omega}[(1-2\xi^2)\sin\omega_d t+2\xi\sqrt{(1-\xi^2)}\cos\omega_d t]\Big\} \\ -\frac{e^{\xi\omega t_i}}{\omega}\Big\{&(\alpha_i t_i+\beta_i)(\xi\sin\omega_d t_i-\sqrt{(1-\xi^2)}\cos\omega_d t_i) \\ &+\frac{\alpha_i}{\omega}[(1-2\xi^2)\sin\omega_d t_i+2\xi\sqrt{(1-\xi^2)}\cos\omega_d t_i]\Big\} \end{aligned} \tag{31}$$

(ii) *Sliding phases*, $\bar{t}_j \leqslant t \leqslant \bar{t}_{j+1}$, $j = 2, 4, 6, \ldots$. To obtain $\chi_r(t)$ and $X_s(t)$, equations (8) and (9) should be solved simultaneously. To this end $\ddot{X}_s$, as given by equation (9), is substituted into equation (8) to yield

$$\ddot{\chi}_r + 2\xi_1\omega_1\dot{\chi}_r + \omega_1^2\chi_r = \frac{-\varepsilon\mu g}{1-\alpha} \tag{32}$$

where

$$\omega_1 = \frac{\omega}{\sqrt{(1-\alpha)}}, \quad \xi_1 = \frac{\xi}{\sqrt{(1-\alpha)}} \tag{33}$$

The solution of equation (32) in each sliding phase for relative displacement, velocity and acceleration may be represented by

$$\chi_r(t) = \upsilon R_s(t) + L(t), \quad \bar{t}_j \leqslant t \leqslant \bar{t}_{j+1}, \quad j = 2, 4, 6, \ldots \tag{34}$$

$$\dot{\chi}_r(t) = \upsilon \dot{R}_s(t) + \dot{L}(t) \tag{35}$$

$$\ddot{\chi}_r(t) = \upsilon \ddot{R}_s(t) + \ddot{L}(t) \tag{36}$$

where

$$v = -\frac{1}{\omega_{1d}}\frac{\varepsilon\mu g}{1-\alpha}, \quad \omega_{1d} = \omega_1\sqrt{(1-\xi_1^2)} \tag{37}$$

$$R_s(t) = \int_{\bar{t}_j}^{t} e^{-\xi_1\omega_1(t-\tau)}\sin\omega_{1d}(t-\tau)\,d\tau$$
$$= \frac{\sqrt{(1-\xi_1^2)}}{\omega_1} - \frac{e^{-\xi_1\omega_1(t-\bar{t}_j)}}{\omega_1}[\xi_1\sin\omega_{1d}(t-\bar{t}_j)+\sqrt{(1-\xi_1^2)}\cos\omega_{1d}(t-\bar{t}_j)] \tag{38}$$

$$\dot{R}_s(t) = -\xi_1\omega_1 R_s(t)+\omega_{1d}R_c(t) \tag{39}$$

$$\ddot{R}_s(t) = \omega_{1d}-(1-2\xi_1^2)\omega_1^2 R_s(t)-2\xi_1\omega_1\omega_{1d}R_c(t) \tag{40}$$

$$R_c(t) = \int_{\bar{t}_j}^{t} e^{-\xi_1\omega_1(t-\tau)}\cos\omega_{1d}(t-\tau)\,d\tau = \frac{\xi_1}{\omega_1}+\frac{e^{-\xi_1\omega_1(t-\bar{t}_j)}}{\omega_1}[\sqrt{(1-\xi_1^2)}\sin\omega_{1d}(t-\bar{t}_j)-\xi_1\cos\omega_{1d}(t-\bar{t}_j)] \tag{41}$$

$L(t)$, $\dot{L}(t)$ and $\ddot{L}(t)$ represent the effects of initial conditions at the start of the sliding phases. They are defined by

$$L(t) = e^{-\xi_1\omega_1(t-\bar{t}_j)}\left[\frac{\dot{\chi}_r(\bar{t}_j)+\xi_1\omega_1\chi_r(\bar{t}_j)}{\omega_{1d}}\sin\omega_{1d}(t-\bar{t}_j)+\chi_r(\bar{t}_j)\cos\omega_{1d}(t-\bar{t}_j)\right] \tag{42}$$

$$\dot{L}(t) = -\xi_1\omega_1 L(t)+\omega_{1d}e^{-\xi_1\omega_1(t-\bar{t}_j)}$$
$$\times\left[\frac{\dot{\chi}_r(\bar{t}_j)+\xi_1\omega_1\chi_r(\bar{t}_j)}{\omega_{1d}}\cos\omega_{1d}(t-\bar{t}_j)-\chi_r(\bar{t}_j)\sin\omega_{1d}(t-\bar{t}_j)\right] \tag{43}$$

$$\ddot{L}(t) = -(1-2\xi_1^2)\omega_1^2 L(t)-2\xi_1\omega_{1d}\omega_1 e^{-\xi_1\omega_1(t-\bar{t}_j)}$$
$$\times\left[-\chi_r(\bar{t}_j)\sin\omega_{1d}(t-\bar{t}_j)+\frac{\dot{\chi}_r(\bar{t}_j)+\xi_1\omega_1\chi_r(\bar{t}_j)}{\omega_{1d}}\cos\omega_{1d}(t-\bar{t}_j)\right] \tag{44}$$

It should be noted that $\chi_r(\bar{t}_j)$ and $\dot{\chi}_r(\bar{t}_j)$ are the relative displacement and the relative velocity at the beginning of sliding phase $\bar{t}_j$.

(iii) *Sliding velocity and displacement*, $\bar{t}_j \leqslant t \leqslant \bar{t}_{j+1}$, $j = 2, 4, 6, \ldots$. Substituting for $\ddot{\chi}_r(t)$ from expression (36) into expression (9) yields the sliding acceleration

$$\ddot{X}_s = \varepsilon\mu g-\ddot{X}_0-\alpha v\ddot{R}_s(t)+\alpha\ddot{L}(t) \tag{45}$$

Integrating the sliding acceleration $\ddot{X}_s$ and noting that at the beginning of any sliding phase j, the sliding velocity is

$$\dot{X}_s(\bar{t}_j) = 0 \tag{46}$$

one obtains

$$\dot{X}_s(t) = -[\dot{X}_0(t)-\dot{X}_0(\bar{t}_j)]-\alpha[\dot{\chi}_r(t)-\dot{\chi}_r(\bar{t}_j)]+\varepsilon\mu g(t-\bar{t}_j) \tag{47}$$

The sliding displacement X_s, which is obtained by integrating the sliding velocity, is given by

$$X_s(t) = -[X_0(t)-t\dot{X}_0(\bar{t}_j)]-\alpha[\chi_r(t)-t\dot{\chi}_r(\bar{t}_j)]+\varepsilon\mu g\left(\frac{t^2}{2}-t\bar{t}_j\right)+C_j \tag{48}$$

where

$$C_j = X_s(\bar{t}_j)+[X_0(\bar{t}_j)-\bar{t}_j\dot{X}_0(\bar{t}_j)]+\alpha[\chi_r(\bar{t}_j)-\bar{t}_j\dot{\chi}_r(\bar{t}_j)]+\frac{\varepsilon\mu g}{2}\bar{t}_j^2 \tag{49}$$

(iv) *The starting times of sliding phases*, $\bar{t}_j$, $j = 2, 4, 6, \ldots$. Substituting for χ and X from equations (5) and (6), and for F from equation (3) into the equilibrium equation (2), it can be shown that during any sliding phase

$$\mu g-|\alpha\ddot{\chi}_r+\ddot{X}_0+\ddot{X}_s| = 0 \tag{50}$$

where α is defined by relation (10). In any non-sliding phase $\dot{X}_s = 0$, and the magnitude of the interface force $|F| < (m+M)\mu g$. Considering these facts, it can be shown that, during any non-sliding phase, the equilibrium equation (2) may be replaced by the following inequality:

$$\mu g - |\alpha \ddot{\chi}_r + \ddot{X}_0| > 0 \tag{51}$$

If $\bar{t}_j$ is the time of initiation of sliding phase j, then $\bar{t}_j^-$, which is infinitesimally less than $\bar{t}_j$, is the end time of the preceding non-sliding phase. Therefore, according to expression (50), the end times of non-sliding phases are the roots of

$$\mu g - |\alpha \ddot{\chi}_r + \ddot{X}_0| = 0 \tag{52}$$

which are obtained as the solution process progresses.

(v) *The end times of sliding phases*, $\bar{t}_{j+1}$, $j = 2, 4, 6, \ldots$. These times are the times for which the sliding velocity $\dot{X}_s$ becomes equal to zero. Considering relation (47), these times are the roots of

$$[\dot{X}_0(t) - \dot{X}_0(\bar{t}_j)] + \alpha[(\dot{\chi}_r(t) - \dot{\chi}_r(\bar{t}_j)] - \varepsilon\mu g(t - \bar{t}_j) = 0 \tag{53}$$

which are obtained as the solution process progresses.

EVALUATION OF RESPONSE

Considering the first 18·7 s of the (N–S) component of the El Centro 1940 earthquake as an input excitation[22] and starting at $t = \bar{t}_1 = 0$ with initial conditions $\chi_r(0) = \dot{\chi}_r(0) = 0$, the non-sliding relative displacement χ_r and relative acceleration $\ddot{\chi}_r$ are evaluated from expressions (18) and (20) for a damping ratio of $\xi = 5$ per cent, a mass ratio $\alpha = 0{\cdot}75$ (which according to expression (10) implies $m/M = 3{\cdot}0$) and four coefficients of friction $\mu = 0{\cdot}05, 0{\cdot}10, 0{\cdot}15, 0{\cdot}20$. The quantity $\mu g - |\alpha \ddot{\chi}_r + \ddot{X}_0|$, which is the left-hand side of inequality (51), is also evaluated. As long as this quantity is positive, the system is responding in the non-sliding phase. Equation (52) yields $\bar{t}_2$, the starting time of the first sliding phase. This time, $\bar{t}_2$, is substituted into expressions (18) and (19) to obtain $\chi_r(\bar{t}_2)$ and $\dot{\chi}_r(\bar{t}_2)$. These initial values are substituted into relations (34) to (49), and the relative displacement χ_r, relative acceleration $\ddot{\chi}_r$, the sliding acceleration $\ddot{X}_s$, sliding velocity $\dot{X}_s$ and sliding displacement X_s are determined as functions of time. The relative velocity $\dot{\chi}_r$ and the ground velocity $\dot{X}_0(t)$ are substituted into equation (53), and time $\bar{t}_3$, the time of the start of the second non-sliding phase, is calculated. In all cases the value of ε is calculated from the right-hand side of equation (4) at the end times of non-sliding phases. The process is continued over the total duration t_d of ground acceleration, and the maxima of χ_r, $\ddot{\chi}$, X_s and X_{rg} and the quantities X_{rs} and X_{cs} are determined for structural periods $T = 0{\cdot}10, 0{\cdot}15, 0{\cdot}20, \ldots, 1{\cdot}00, 1{\cdot}10, 1{\cdot}20, 1{\cdot}30, \ldots, 1{\cdot}60, 1{\cdot}80$ and $2{\cdot}00$ s. Here, X_{rg} represents the relative-to-ground displacement. X_{rs} represents the residual sliding displacement when the earthquake stops and X_{cs} represents the cumulative sliding or total sliding movement during the entire duration of ground motion.

Normalized responses are defined by:

$$\text{Absolute acceleration} = \frac{\ddot{\chi}(T, \xi, \mu)}{a} \tag{54}$$

$$\text{Relative displacement} = \frac{\chi_r(T, \xi, \mu)}{D} \tag{55}$$

$$\text{Sliding displacement} = \frac{X_s(T, \xi, \omega)}{D} \tag{56}$$

$$\text{Relative to ground displacement} = \frac{X_{rg}(T, \xi, \mu)}{D} \tag{57}$$

$$\text{Residual sliding displacement} = \frac{X_{rs}(T, \xi, \mu)}{D} \tag{58}$$

$$\text{Cumulative sliding displacement} = \frac{X_{cs}(T, \xi, \mu)}{D} \tag{59}$$

where

$$\ddot{\chi}(T,\xi,\mu) = \max_t |\ddot{\chi}(t,T,\xi,\mu)| \tag{60}$$

$$\chi_r(T,\xi,\mu) = \max_t |\chi_r(t,T,\xi,\mu)| \tag{61}$$

$$X_s(T,\xi,\mu) = \max_t |X_s(t,T,\xi,\mu)| \tag{62}$$

$$X_{rg}(T,\xi,\mu) = \max_t |\chi_r(t,T,\xi,\mu)+X_s(t,T,\xi,\mu)| \tag{63}$$

$$X_{rs}(T,\xi,\mu) = |X_s(t_d,T,\xi,\mu)| \tag{64}$$

$$X_{cs}(T,\xi,\mu) = |X_{cs}(t_d,T,\xi,\mu)| \tag{65}$$

D is the peak ground displacement, A is the peak ground acceleration and t_d is the duration of ground motion.

To display the effectiveness of sliding supports in reducing the maximum levels of response acceleration and relative displacement, and to show the variations of the maximum sliding displacements and other response quantities with respect to coefficient of friction and mass ratio, the normalized responses are plotted against period T in Figures 3 to 22 for four coefficients of friction $\mu = 0{\cdot}05, 0{\cdot}10, 0{\cdot}15$ and $0{\cdot}20$, and for three mass ratios $\alpha = 0{\cdot}75$ (meaning $m/M = 3{\cdot}0$), $\alpha = 0{\cdot}50$ (meaning $m/M = 1{\cdot}0$) and $\alpha = 0{\cdot}25$ (meaning $m/M = \frac{1}{3}$). It should be noted that Figures 6 and 13 are the normalized absolute acceleration spectra and the normalized sliding displacement spectra for the S86E component of the 1949 Olympia earthquake. The absolute acceleration response for rigid structures (zero period) as presented in Figures 3 to 6 is calculated by considering the fact that for a rigid structure the maximum response acceleration is equal to μg ($\mu g \leqslant A$ in this case), i.e. the base's coefficient of friction times the gravitational acceleration. It should be noted that the response quantities are very sensitive to the exact times of starts and ends of sliding phases defined by the

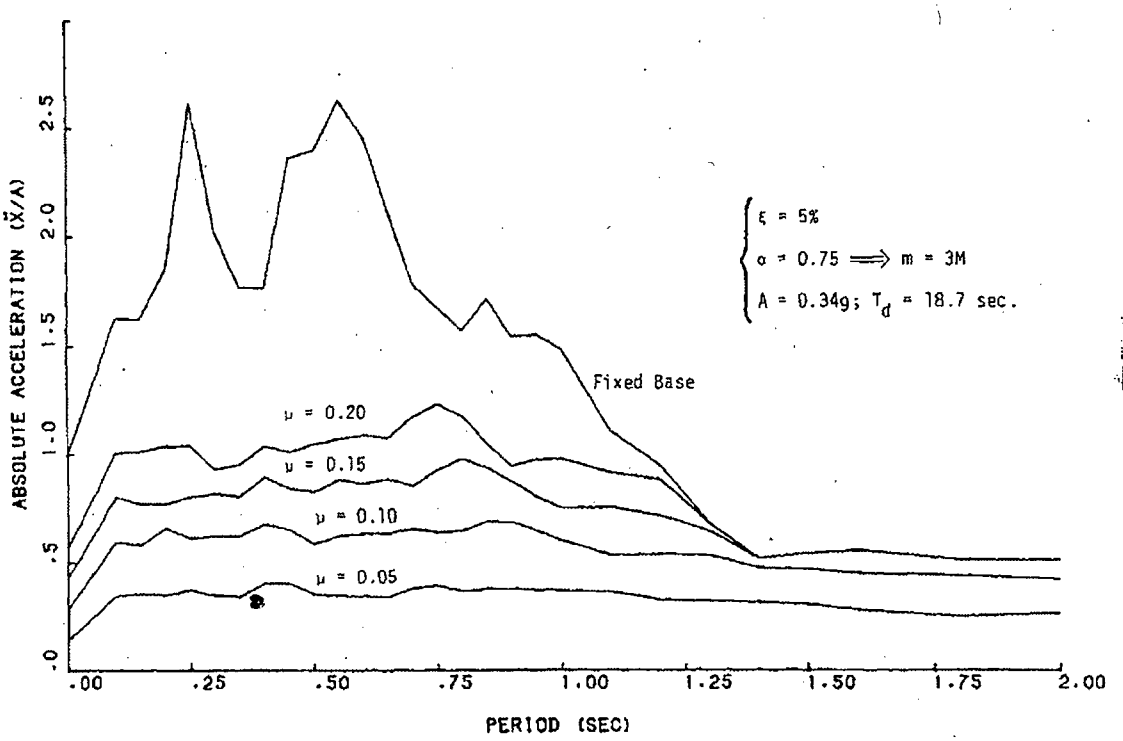


Figure 3. Acceleration response spectrum for El Centro 1940 earthquake (N–S)

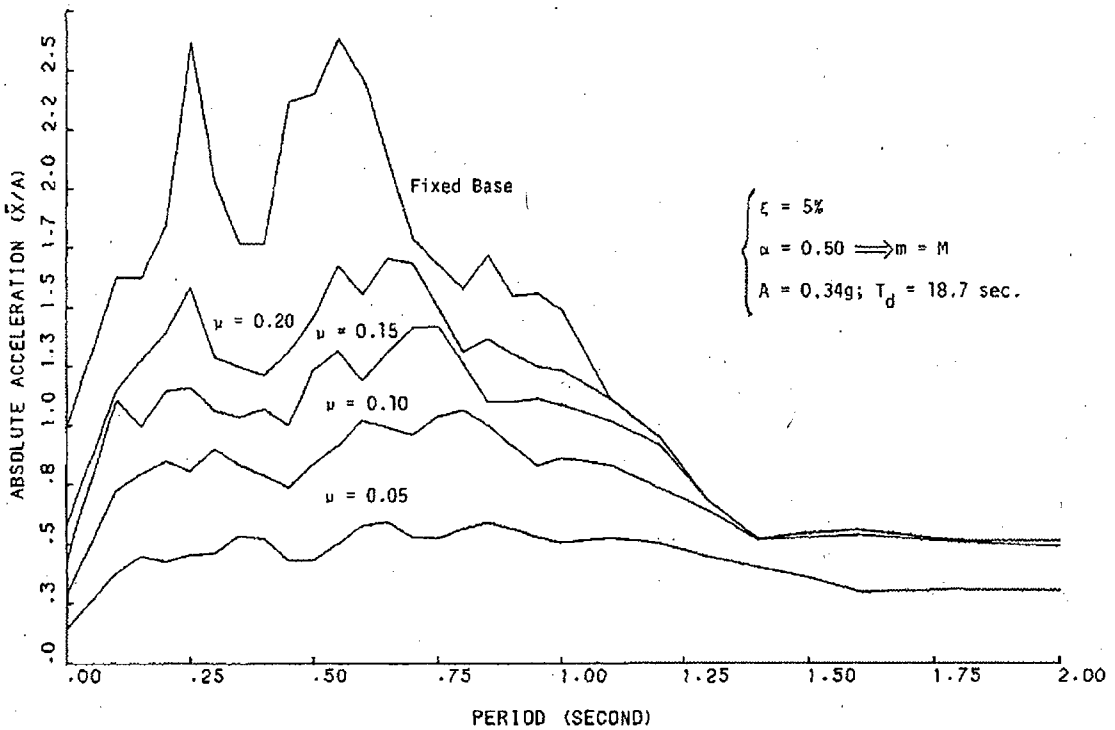


Figure 4. Acceleration response spectrum for El Centro 1940 earthquake (N–S)

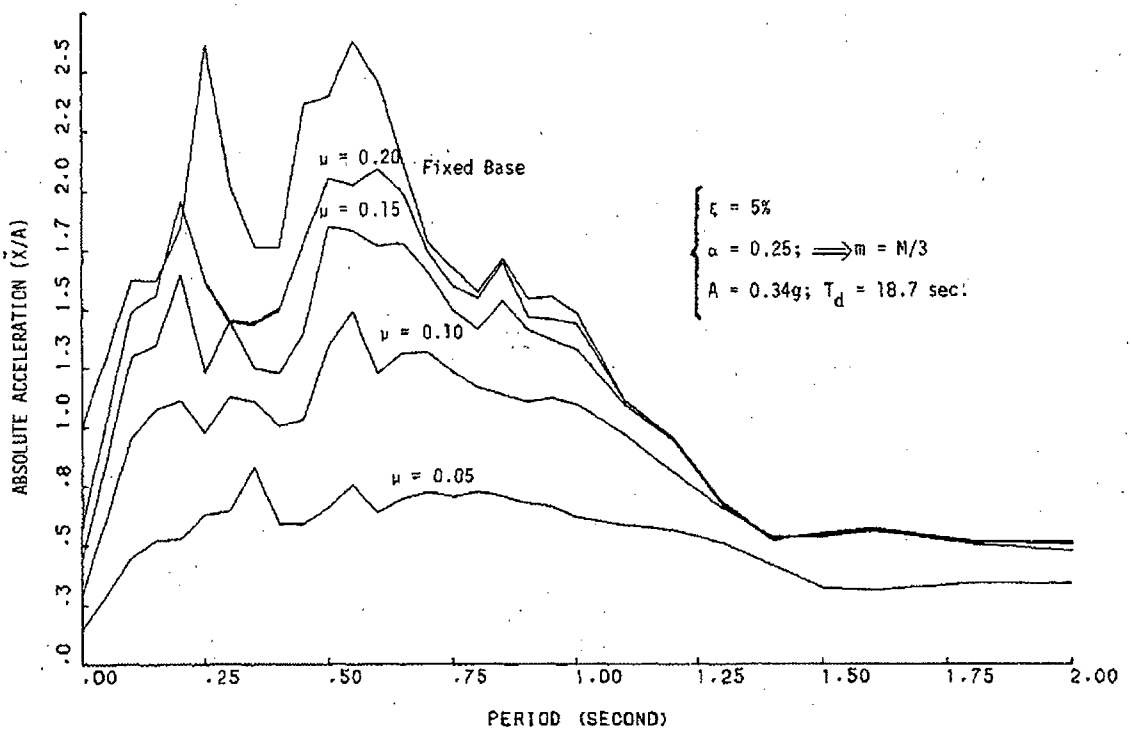


Figure 5. Acceleration response spectrum for El Centro 1940 earthquake (N–S)

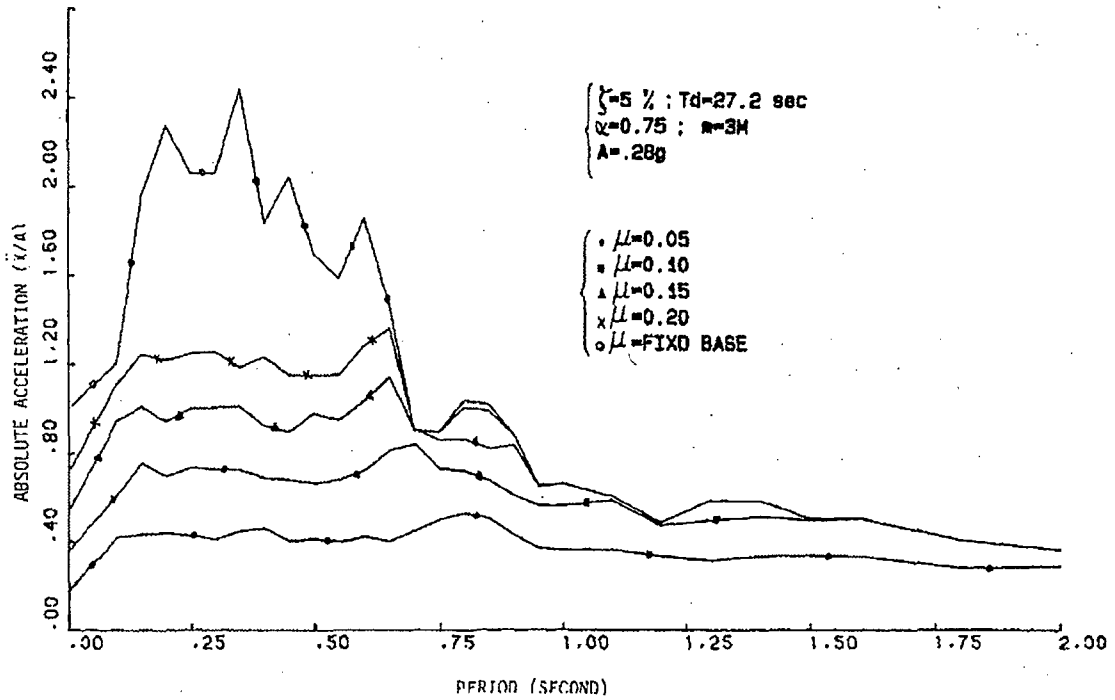


Figure 6. Acceleration response spectrum for 1949 Olympia earthquake (S86E)

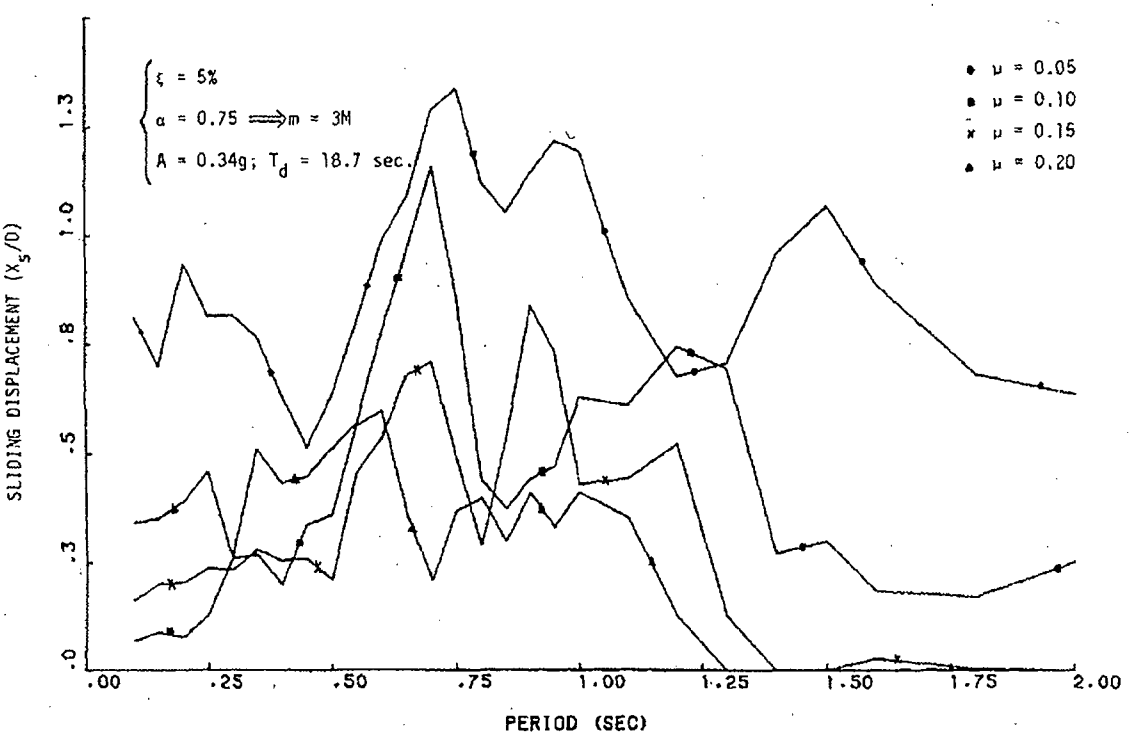


Figure 7. Sliding displacement response spectrum for El Centro 1940 earthquake (N–S)

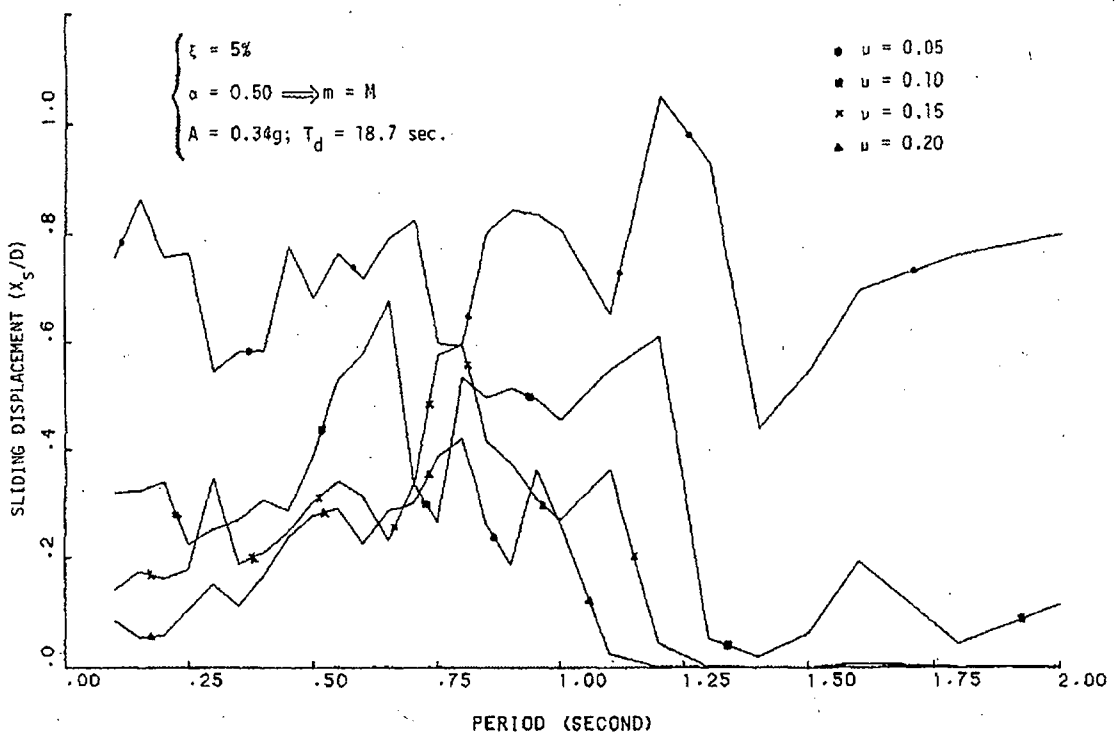


Figure 8. Sliding displacement response spectrum for El Centro 1940 earthquake (N–S)

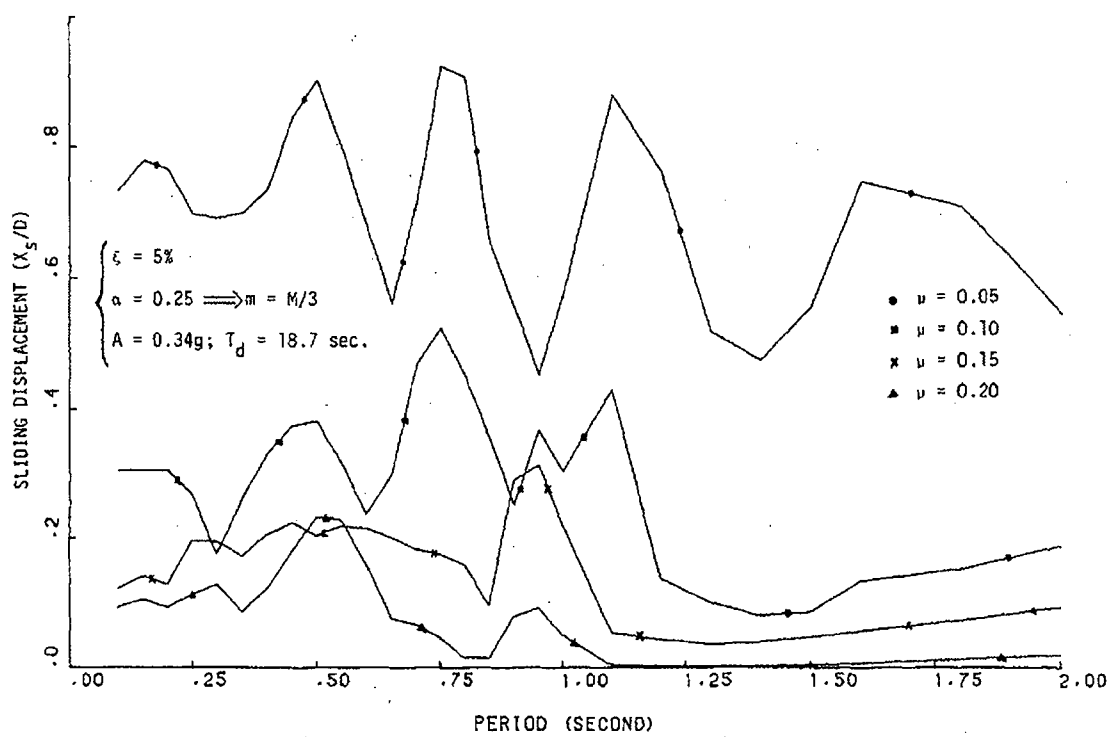


Figure 9. Sliding displacement response spectrum for El Centro 1940 earthquake (N–S)

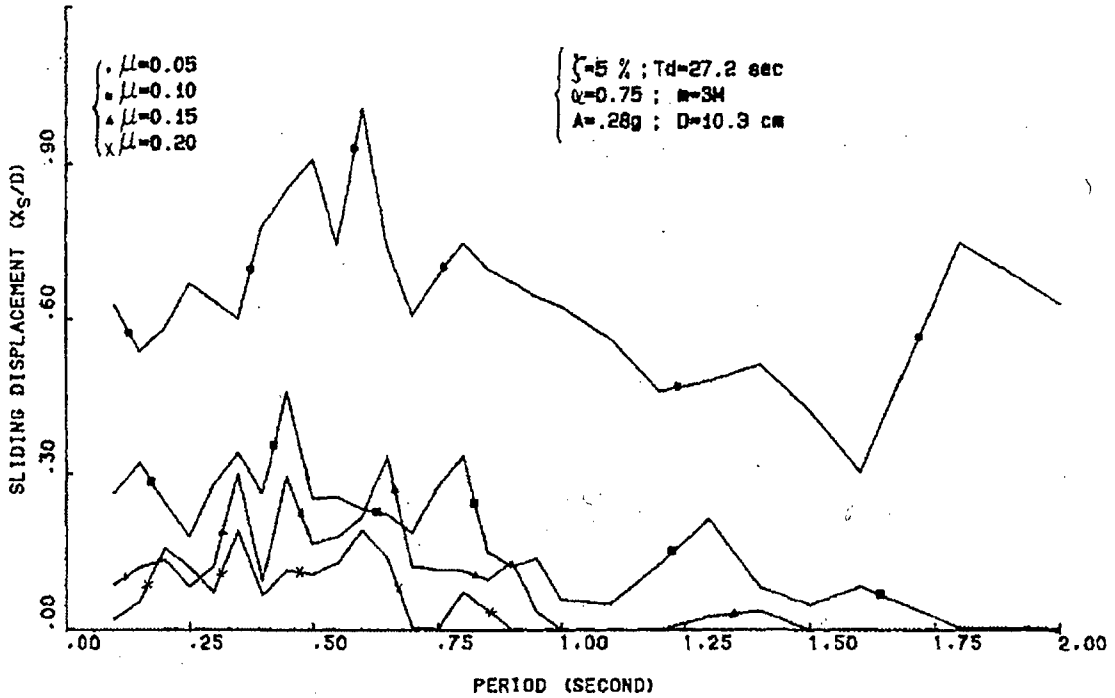


Figure 10. Sliding displacement response for 1949 Olympia earthquake (S86E)

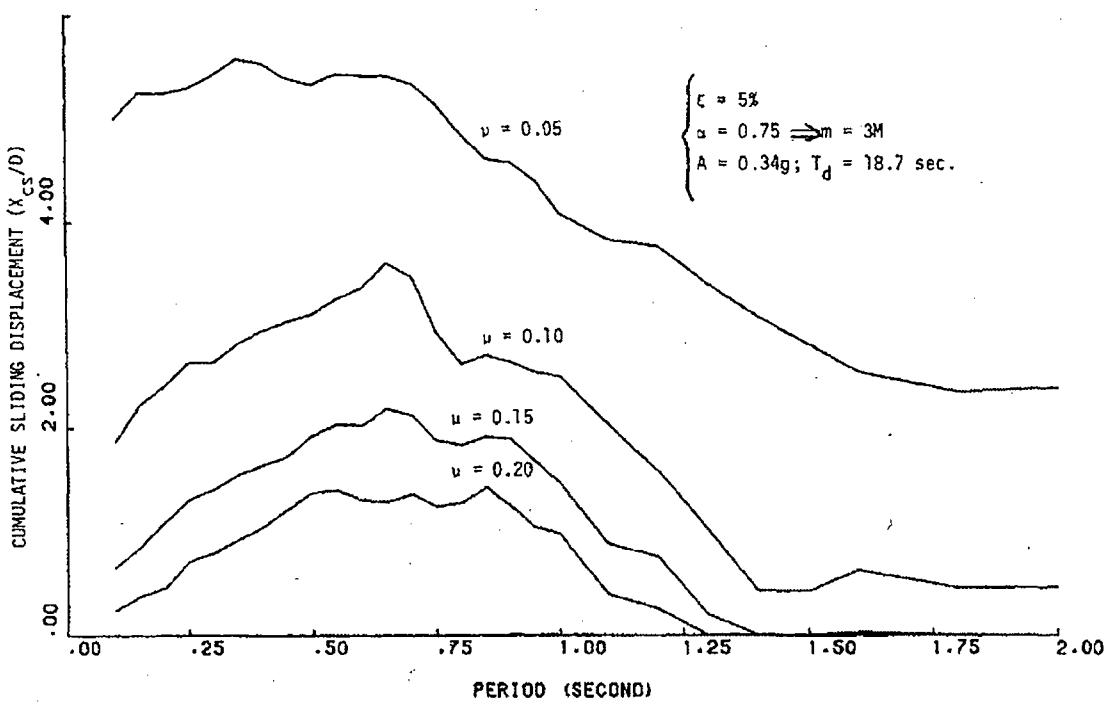


Figure 11. Cumulative sliding displacement spectrum for El Centro 1940 earthquake (N–S)

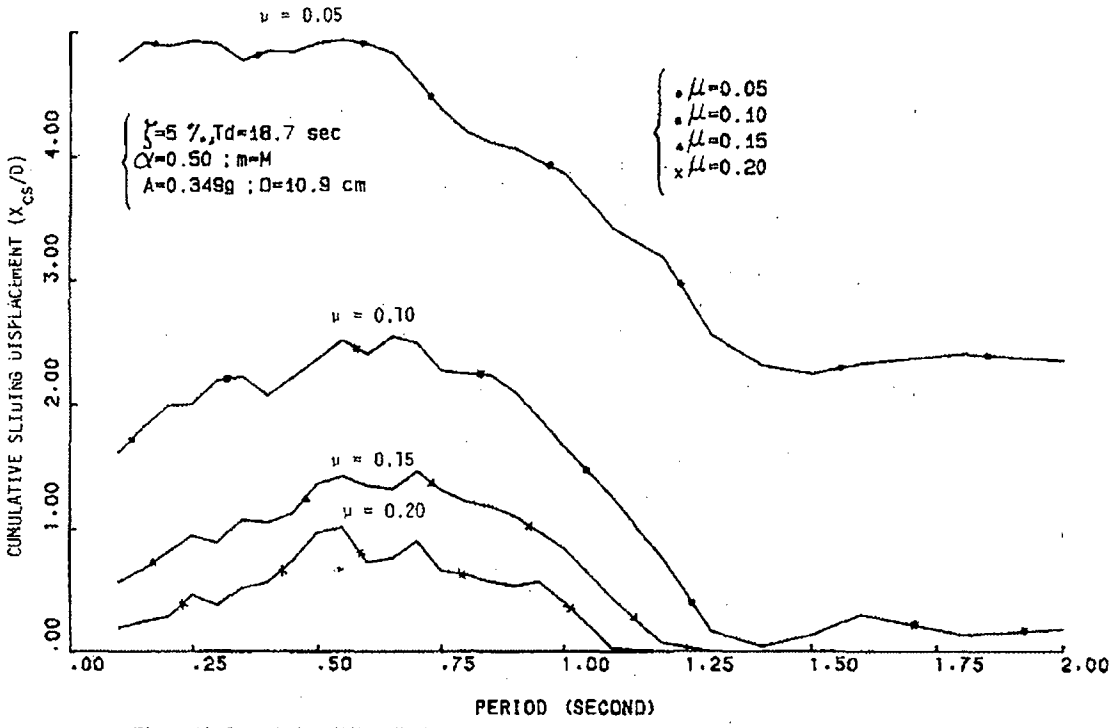


Figure 12. Cumulative sliding displacement response spectrum for El Centro 1940 earthquake (N–S)

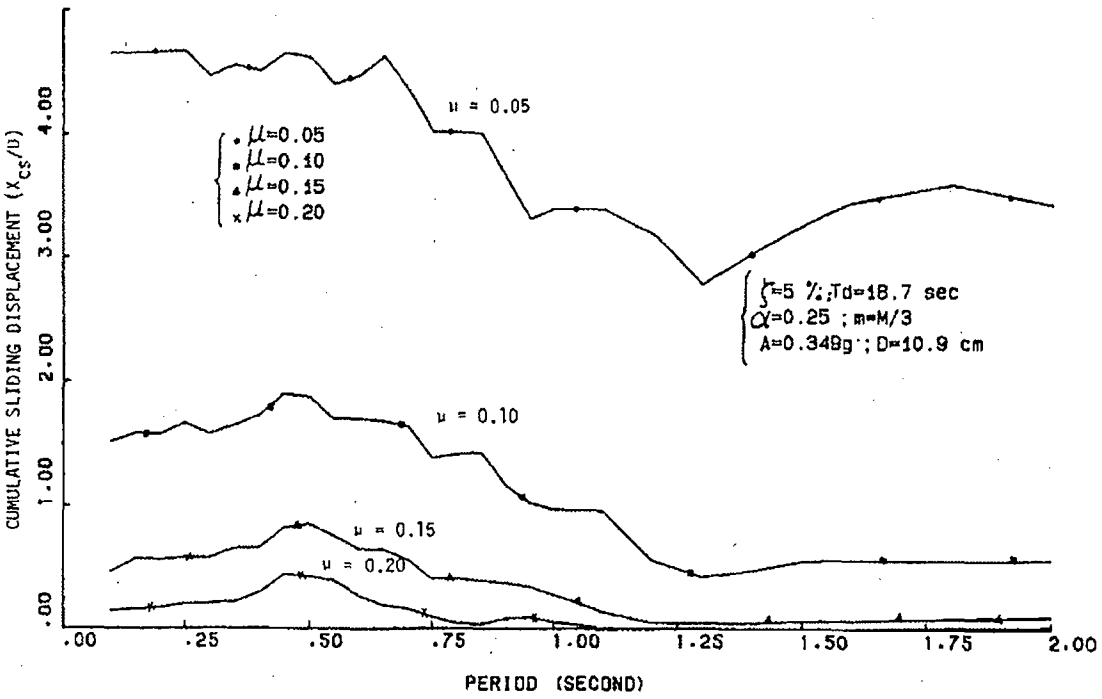


Figure 13. Cumulative sliding displacement response spectrum for El Centro 1940 earthquake (N–S)

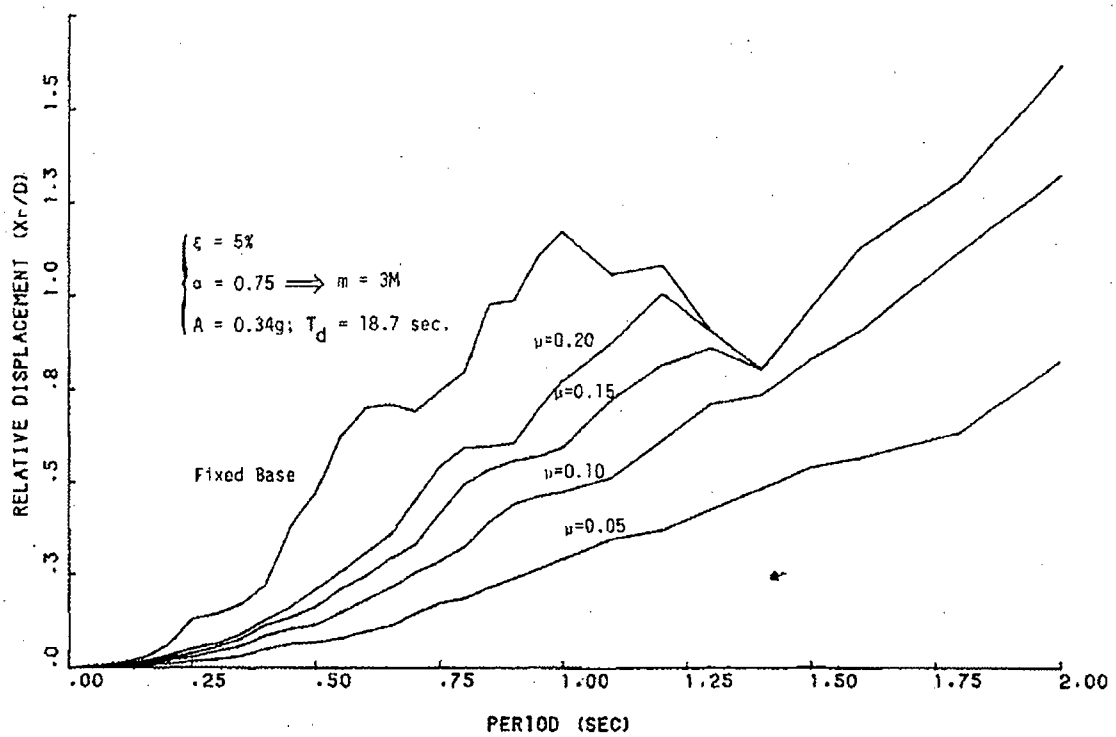


Figure 14. Displacement response spectrum for El Centro 1940 earthquake (N–S)

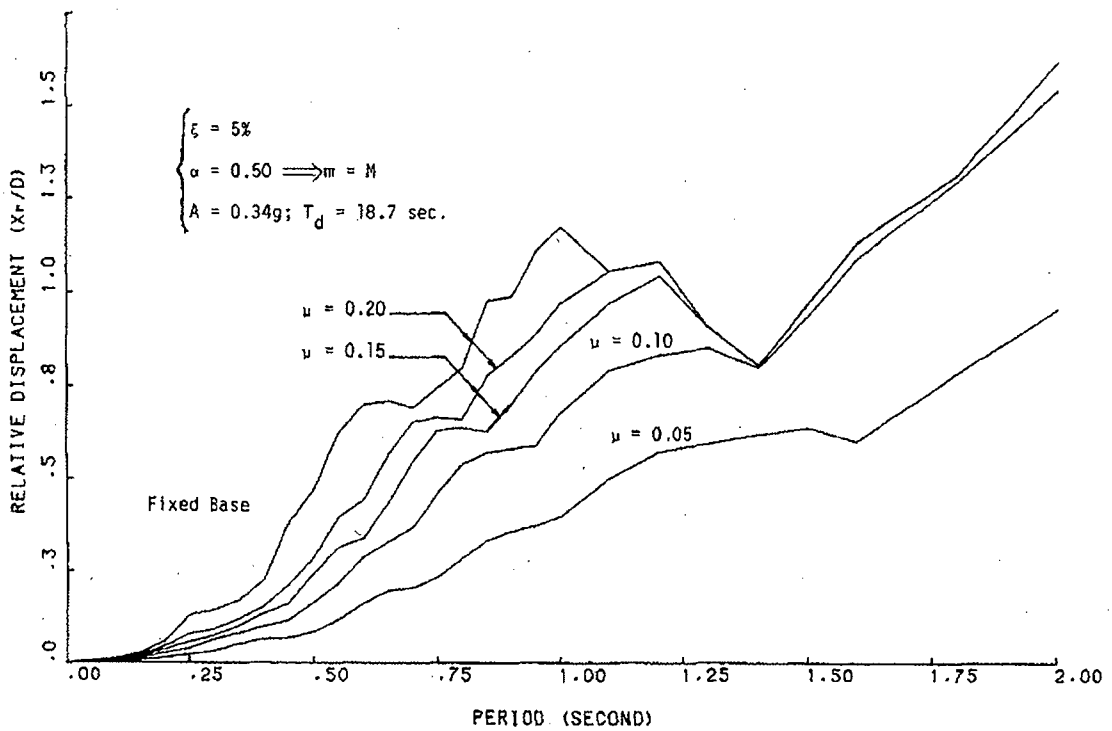


Figure 15. Displacement response spectrum for El Centro 1940 earthquake (N–S)

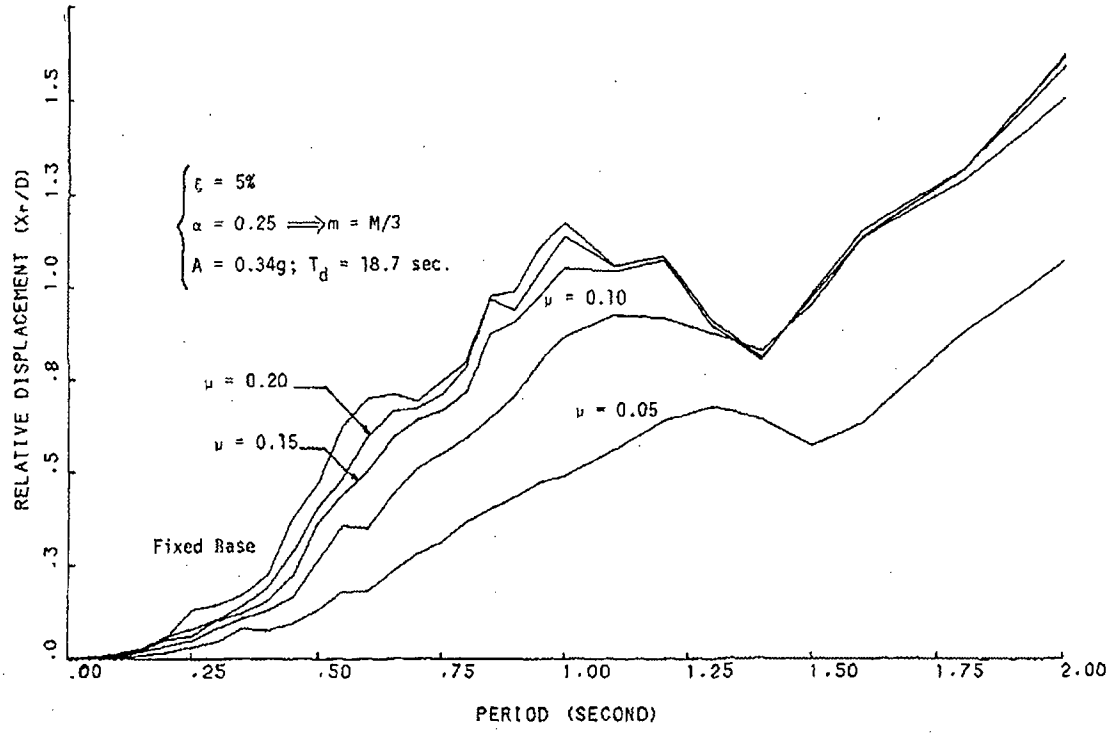


Figure 16. Displacement response spectrum for El Centro 1940 earthquake (N–S)

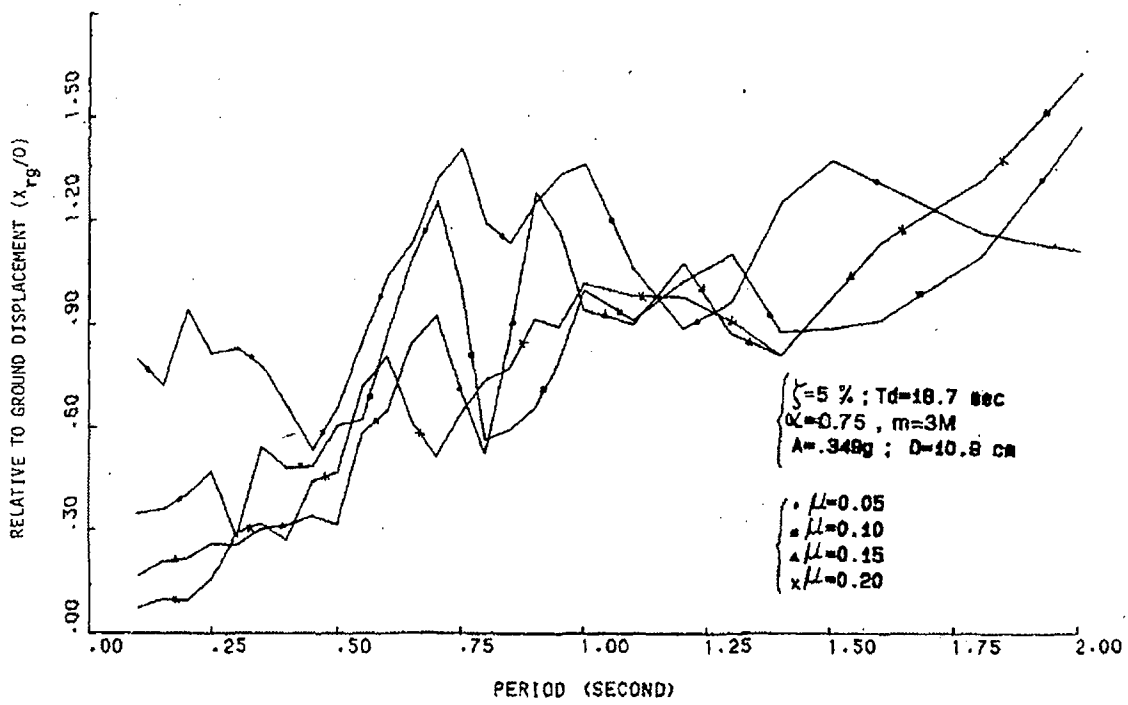


Figure 17. Relative-to-ground displacement response spectrum for El Centro 1940 earthquake (N–S)

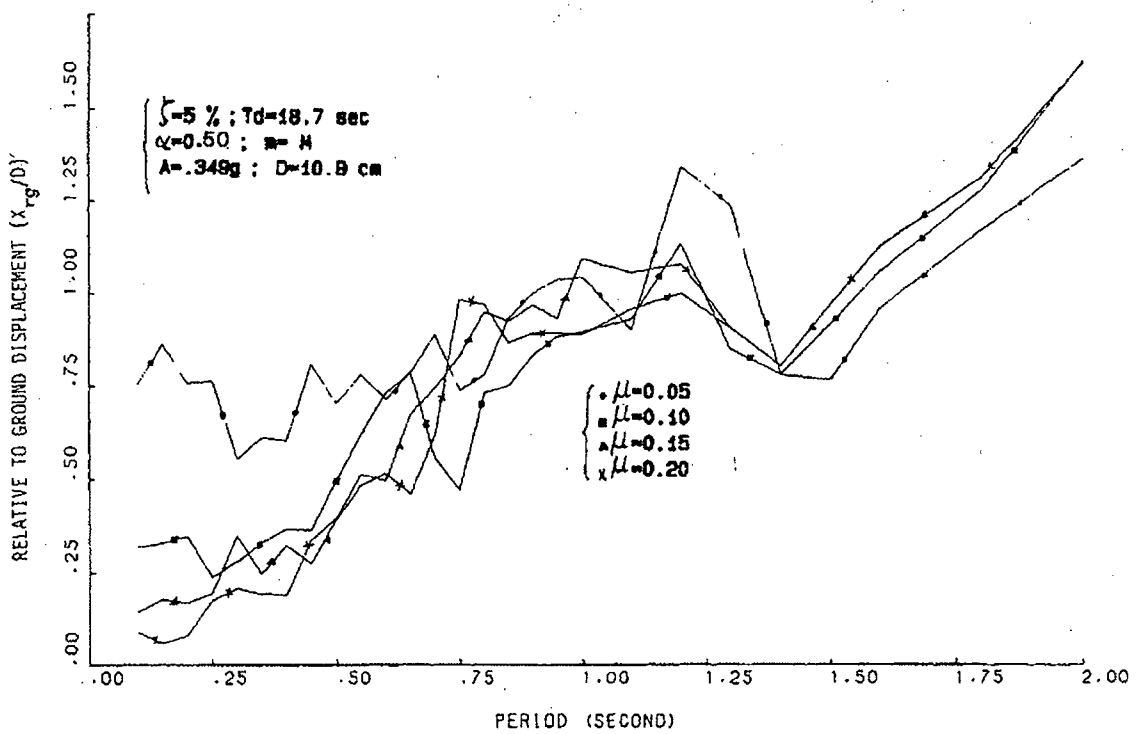


Figure 18. Relative-to-ground displacement response spectrum for El Centro 1940 earthquake (N–S)

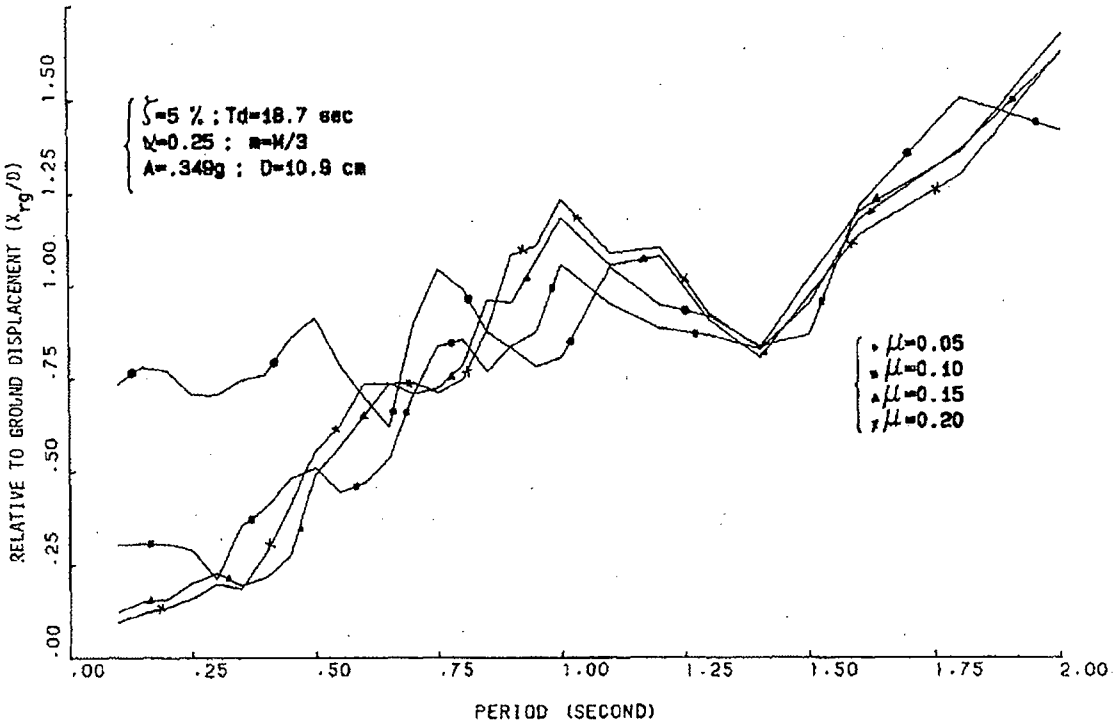


Figure 19. Relative-to-ground displacement response spectrum for El Centro 1940 earthquake (N–S)

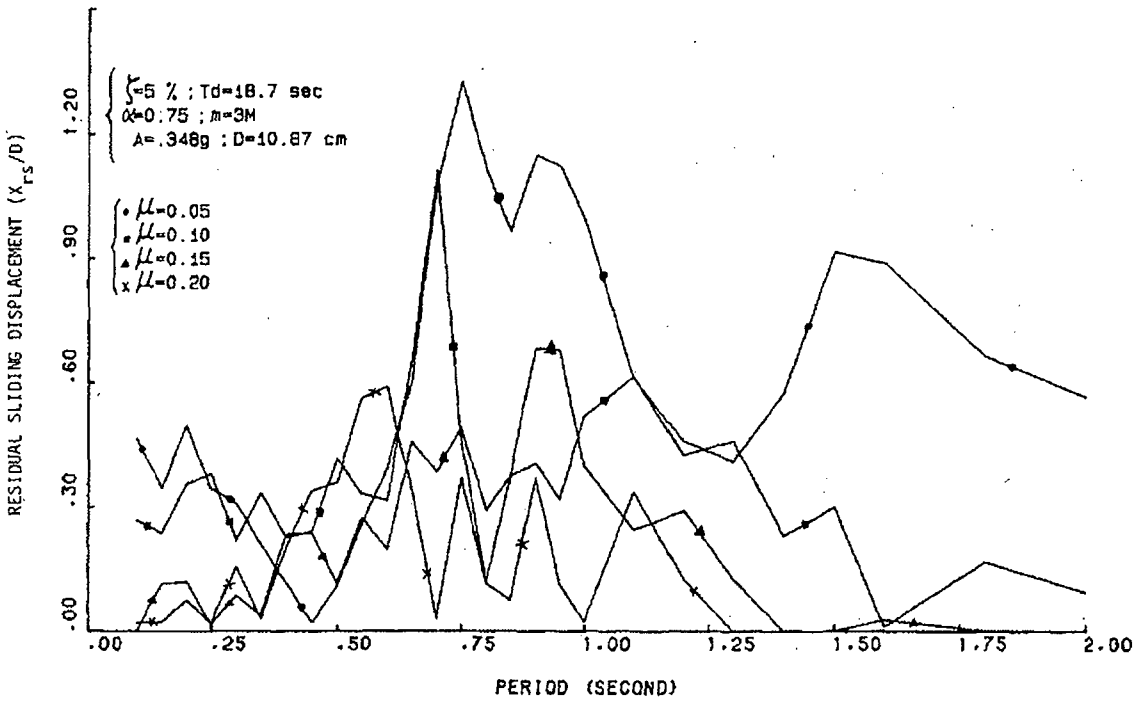


Figure 20. Residual sliding displacement spectrum for El Centro 1940 earthquake (N–S)

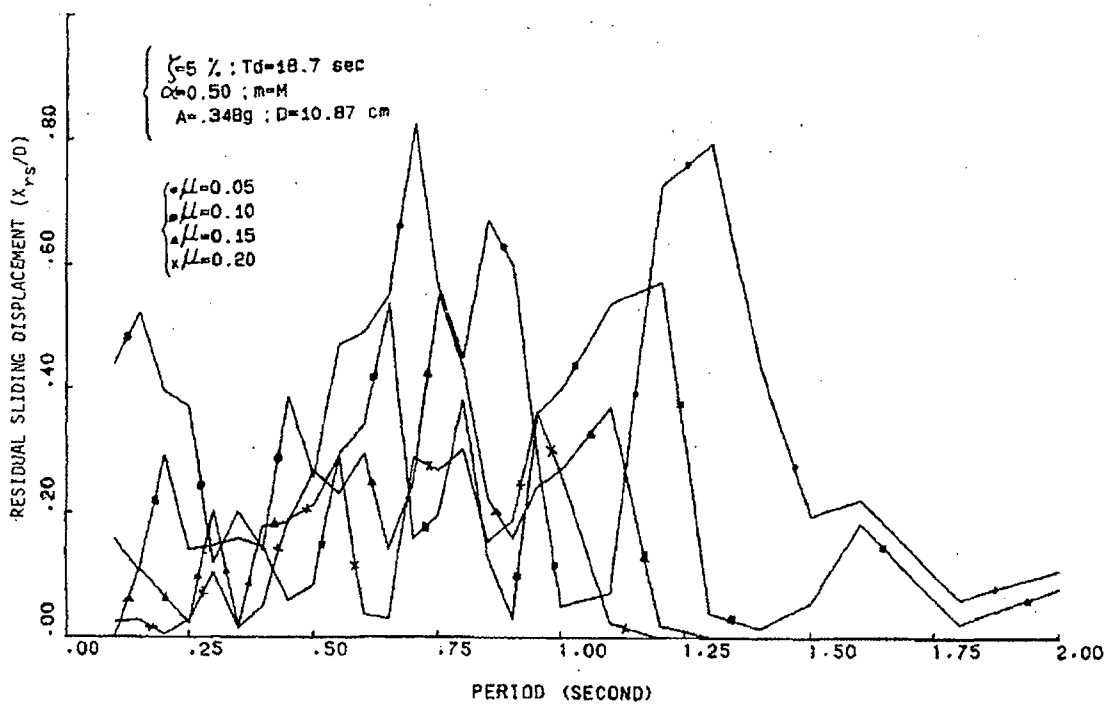


Figure 21. Residual sliding displacement spectrum for El Centro 1940 earthquake (N–S)

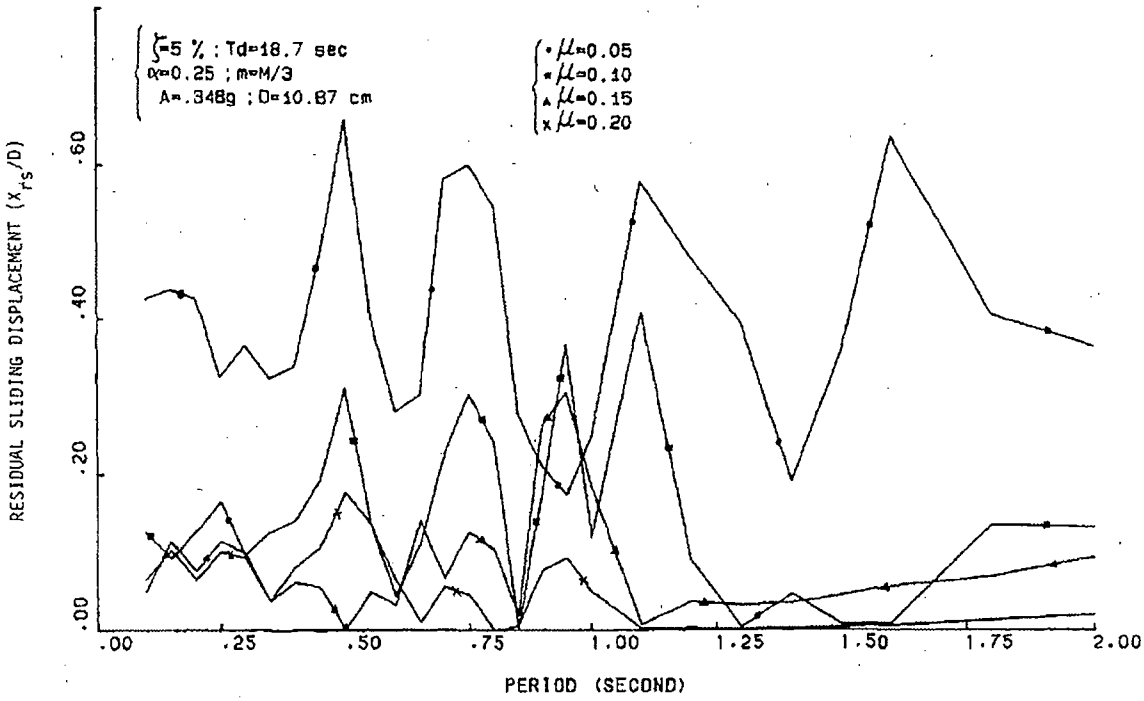


Figure 22. Residual sliding displacement spectrum for El Centro 1940 earthquake (N–S)

roots of equations (52) and (53) respectively. It was found that the response quantities can be obtained with sufficient accuracy if either the roots of these equations are evaluated with an accuracy of $\pm(10^{-6})$s or if the absolute values of the left hand sides of these equations are less than 10^{-4}.

DISCUSSION OF RESULTS

Figure 3 represents the normalized acceleration spectrum for structures in which the mass of the structure is three times the mass of the foundation raft. It is observed that:

(i) the spectral response of isolated structures appears to be almost independent of frequency of excitations, especially for lower coefficients of friction;

(ii) the level of response depends on the base's coefficient of friction. As expected the smaller the coefficient of friction the lower the response;

(iii) the level of response of isolated structures is considerably lower than the level of response of corresponding fixed base structures.

Figures 4 and 5 represent the normalized acceleration spectra for structures in which the mass of the structure is the same as that of the foundation raft and is one-third of the mass of the foundation raft, respectively. The observations made for Figure 3 hold, except that the response tends to be more frequency dependent, especially for larger coefficients of friction. Comparing Figures 3, 4 and 5, it may be concluded that the larger the mass of the structure as compared to the mass of the foundation raft, the lower the level of acceleration response. Figure 6 represents the normalized absolute acceleration spectrum for the S86E component of the 1949 Olympia earthquake for a mass ratio $\alpha = 0{\cdot}75$ (meaning $m/M = 3$). The same observations as those for Figure 3 apply. It is noted, as in the case of harmonic excitations,[19] that increase of the levels of the input excitations increases the isolation effectiveness in cutting down the acceleration response.

Figure 7 represents the normalized sliding displacement spectrum for structures in which the mass of the structure is three times the mass of the foundation raft. Contrary to expectations which were substantiated by harmonic excitations,[19] for some structures the maximum sliding displacement is lower for lower coefficients of friction. This is due to the disorderly arrangement of pulses in earthquake ground motion which controls the direction of sliding. However, as may be noted from Figure 11, the cumulative sliding displacement, which is the total sliding movement during earthquake, is higher for lower coefficients of friction. This, of course, matches the expectation. To study the effects of mass ratio on sliding displacement, sliding displacement spectra for mass ratios of $\alpha = 0{\cdot}5$ ($m = M$) and $\alpha = 0{\cdot}25$ ($m = M/3$) are presented in Figures 8 and 9 respectively. By comparing Figures 7, 8 and 9 it may be observed (as expected) that in general the lighter the superstructure, as compared to the weight of the foundation raft, the less is the maximum sliding amplitude. As may be noted from these figures, the maximum (supremum) sliding even for the lowest coefficient of friction considered, i.e. $\mu = 0{\cdot}05$, is of the order of 1·25 times the peak ground displacement. There is a theoretical limit to the amount of sliding as the coefficient of friction is reduced to zero. For zero coefficient of friction, theoretically, no acceleration is transferred to the superstructure. That is, the structure remains stationary in the inertial frame, and the maximum sliding displacement is equal to the peak ground displacement. In real cases, however, there is a momentum transfer before sliding, and the structure will not remain stationary. To elucidate the sensitivity of the sliding displacement to the input excitations, the sliding displacement spectrum for the S86E component of the 1949 Olympia earthquake for a mass ratio $\alpha = 0{\cdot}75$ ($m/M = 3$) is presented in Figure 10. Comparing Figures 7 and 10, it is noted, similar to the case of harmonic excitations,[19] that reduction of the levels of the input excitations, in general, reduces the sliding displacement response.

The cumulative sliding displacement response spectra for mass ratios $\alpha = 0{\cdot}75$, 0·50 and 0·25 are given in Figures 11, 12 and 13 respectively. Comparing these figures, it may be observed that in general, for the cases considered, the level of cumulative sliding displacement response does not significantly vary with the mass ratio. This implies that the higher the mass ratio (the heavier the structure as compared to the foundation raft) the larger the amount of energy dissipated at the support level. This is consistent with the lower levels of acceleration and relative displacement responses obtained for larger mass ratios.

The normalized relative displacement spectrum for the mass ratio $\alpha = 0{\cdot}75$ ($m = 3M$) is given in Figure 14. As expected, the relative displacements, that is, the displacements of the mass relative to the foundation raft, are much lower than the fixed base response. This is, of course, expected, because, according to Figure 3, the mass is subjected to much lower accelerations than if the base of the structure were fixed. Of course, the lower levels of relative displacements imply lower levels of deformations in the structures, i.e. deformations which may be much lower than the structure's damage threshold. The same conclusions may be drawn from Figures 15 and 16 which are for mass ratios $\alpha = 0{\cdot}50$ ($m = M$) and $\alpha = 0{\cdot}25$ ($m = M/3$), except that because of the increased levels of acceleration response, the level of relative displacement response is in general higher than that for $\alpha = 0{\cdot}75$.

Relative-to-ground displacement is of interest in structures which have elements such as pipings connected to the ground. Normalized spectra for relative-to-ground displacements for three different mass ratios are given in Figures 17, 18 and 19. These figures suggest that, in general, for the ground motion, the structures and the coefficients of friction considered, the maximum (supremum) of the relative-to-ground displacement is of the order of 1·25 times the peak ground displacement.

The residual sliding displacement, that is, the displacement due to sliding which remains when the ground motion is over, is of practical interest for any later re-centring operation. The normalized residual sliding displacement spectra for the three mass ratios are given in Figures 20, 21 and 22. From these figures it may be observed that for the ground motion, the structures and the coefficients of friction considered, the maximum (supremum) residual displacement is of the order of 1·25 times the peak ground displacement.

The large reduction in the level of response acceleration and the relatively 'small' relative-to-ground displacements, sliding displacements and residual sliding displacements suggest that sliding supports have the potential of being a very effective and inexpensive isolation system. The above results which are based on two ground excitations are also supported by the results from harmonic excitations.[19]

CONCLUSIONS

Through the study of response of sliding structures to the N–S component of the El Centro 1940 earthquake, and the S86E component of the 1949 Olympia earthquake, it has been established that sliding supports can be quite effective in controlling the level of acceleration response of structures. Since, for the coefficients of friction considered, the suprema of relative-to-ground displacements, sliding displacements and residual sliding displacements are all only of the order of 1·25 times the peak ground displacement, it is concluded that sliding supports can effectively isolate structures from support excitations and are practical.

For low coefficients of friction, the acceleration response does not vary with the frequency content of the ground motion. This implies that sliding supports can be effectively used for all kinds of sites, whether hard or soft soil, whether close or far from causative faults. The above conclusions are also supported by the studies of harmonic excitations.

ACKNOWLEDGEMENT

The support of the U.S. National Science Foundation under Grant No. CEE–8112580 is gratefully acknowledged.

REFERENCES

1. R. W. Clough and J. Penzien, *Dynamics of Structures*, McGraw-Hill, New York, 1975.
2. R. R. Martel, 'The effect of earthquake on buildings with a flexible first story', *Bull. seism. soc. Am.* **19**, 167–178 (1929).
3. N. B. Green, 'Flexible first story construction for earthquake resistance', *Trans. ASCE* **100**, 645–674 (1935).
4. L. S. Jacobsen, 'Effect of flexible first story in a building located on vibrating ground', in *S. Timoshenko 60th Anniversary Volume*, Macmillan, New York, 1938.
5. M. Fintel and R. R. Khan, 'Shock-absorbing soft-story concept for multistory earthquake structures', *J. Am. concrete inst.* **66**, 381–390 (1969).
6. A. K. Chopra, D. P. Clough and R. W. Clough, 'Earthquake resistance of building with a "soft" first story', *Earthquake eng. struct. dyn.* **1**, 347–355 (1973).
7. K. Matsushita and M. Izumi, 'Studies on mechanisms to decrease earthquake forces applied to buildings', *Proc. 4th world conf. earthquake eng.*, Santiago de Chile **1** (1969).
8. M. S. Caspe, 'Earthquake isolation of multistory concrete structures', *J. Am. concrete inst.*, **67**, 923–933 (1970).
9. J. M. Kelly, J. M. Eidinger and C. J. Derham, 'A practical soft story isolation system', *Report No. UCB/EERC–77/27*, Earthquake Engineering Research Center, University of California, Berkeley, CA, 1977.
10. J. M. Kelly and D. E. Chitty, 'Control of seismic response of piping systems and components in power plants by base isolation', *Proc. ASCE pressure vessels piping conf., 79–PVP55*, San Francisco (25–29 June 1979).
11. C. J. Derham, A. G. Thomas and J. M. Kelly, 'A rubber bearing system for seismic protection of structures', in *Engineering Design for Earthquake Environments*, I. Mech. E. Conf. Publ. 1978–12, London, 1979, pp. 53–58.
12. R. I. Skinner, J. M. Kelly and A. J. Heine, 'Hysteretic dampers for earthquake-resistant structures', *Earthquake eng. struct. dyn.*, **3**, 287–296 (1975).
13. J. M. Kelly, 'Control devices for earthquake-resistant structural design', in *Structural Control* (Ed. H. H. E. Leipholz), North-Holland and SM Publications, Amsterdam, 1980, pp. 391–413.
14. J. M. Kelly and S. B. Hodder, 'Experimental study for lead and elastomeric dampers for base isolation systems', *Report No. UCB/EERC–81/16*, Earthquake Engineering Research Center, University of California, Berkeley, CA, 1981.
15. J. M. Kelly, 'The influence of base isolation on the seismic response of light secondary equipment', *Report No. UCB/EERC–81/17*, Earthquake Engineering Research Center, University of California, Berkeley, CA, 1982.
16. J. H. Williams, Jr., 'Designing earthquake-resistant structures', *Techn. rev.*, **76**, 37–43 (1973).
17. C. Plichon and F. Jolivet, 'Aseismic foundation systems for nuclear power plants', *Proc. S.M.I.R.T. conf.*, London, England, Paper No. C 190/1978 (1978).
18. J. M. Kelly, 'Aseismic base isolation', *Shock vib. dig.*, **14**, No. 5, 17–25 (1982).
19. N. Mostaghel, M. Hejazi and J. Tanbakuchi, 'Response of sliding structures to harmonic support motion', *Earthquake eng. struct. dyn.*, **11**, 355–366 (1983).
20. D. Chen and R. W. Clough, 'Earthquake response of structures with friction sliding motion', Earthquake Engineering Research Center, University of California, Berkeley, CA, 1981.
21. D. Chen, 'Earthquake response control by sliding friction', *Proc. US–PRC bilateral workshop earthquake eng.*, Harbin, China, 27–30 August (1982).
22. *Strong Motion Earthquake Accelerograms*, Vol. II, Part A, California Institute of Technology, EERL 71–50, 1971.

ENCLOSURE 1

LICENSE AMENDMENT REQUEST 234
REQUEST FOR REVIEW AND APPROVAL OF METHODOLOGY CHANGE REGARDING AUXILIARY BUILDING CRANE UPGRADE

GENERAL ARRANGEMENT DRAWINGS OF UPGRADED KPS AUXILIARY BUILDING CRANE

D-20776-001

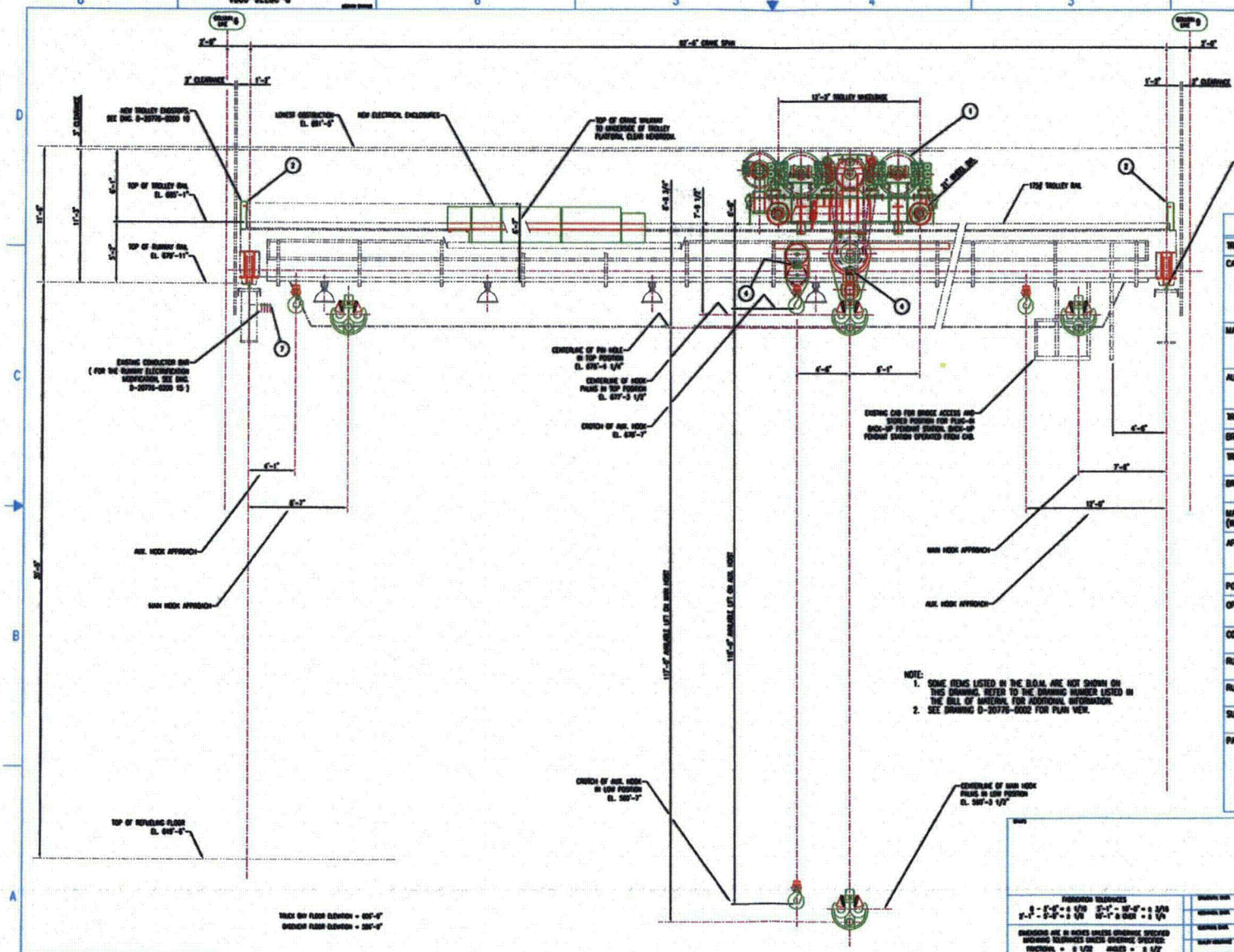
Single Failure Proof Trolley #2949 General Arrangement, Elevation View

D-20776-002

Single Failure Proof Trolley #2949 General Arrangement, Plan View

KEWAUNEE POWER STATION

DOMINION ENERGY KEWAUNEE, INC.



NOTE:
1. SOME ITEMS LISTED IN THE B.O.M. ARE NOT SHOWN ON THIS DRAWING. REFER TO THE DRAWING NUMBER LISTED IN THE BILL OF MATERIAL FOR ADDITIONAL INFORMATION.
2. SEE DRAWING D-20776-0002 FOR PLAN VIEW.

CRANE DATA

TROLLEY NO.	2949
CAPACITIES	MAIN HOIST: DRL = MCL = 125 TONS AUX. HOIST: DRL = MCL = 15 TONS TROLLEY: DRL = MCL = 125 TONS BRIDGE: 125 TONS
MAIN HOIST SPEED	0.25–5 F.P.M. VARIABLE (CRITICAL LOAD) 0.25–7.5 F.P.M. VARIABLE (NON CRITICAL LOAD)
AUX. HOIST SPEED	0.25–20 F.P.M. VARIABLE (CRITICAL LOAD) 0.25–30 F.P.M. VARIABLE (NON CRITICAL LOAD)
TROLLEY SPEED	1–40 F.P.M. VARIABLE
BRIDGE SPEED	1.25–50 F.P.M. VARIABLE
TROLLEY WEIGHT	112,000 LBS.
BRIDGE WEIGHT	EXISTING
MAXIMUM BRIDGE WHEEL LOAD (WITHOUT IMPACT)	98,500 LBS. @ 125 TON CAPACITY (TO BE CONFIRMED)
APPLICATION	NUREG 0554, CMAA CLASS A, INDOORS
POWER SUPPLY	460–3–60 AC
OPERATOR CONTROLS	RADIO REMOTE CONTROL WITH BACK-UP PENDANT STATION
CONTROL ENCLOSURE	NEMA 4 (FOR NEW ENCLOSURES)
RUNWAY COLLECTORS	CONDUCTOR BAR – 3 EXISTING ACECO TO ADD 4TH FOR GROUND
RUNWAY LENGTH	150 FT. APPROX.
SURFACE PREPARATION	SSPC–SP6
PAINT	PRIMER: 2.5–6.0 MILS DFT, KEELER & LONG KL3200, WHITE FINISH: 1.5–2.5 MILS DFT, KEELER & LONG POLY–SILICONE ENAMEL– SAFETY ORANGE TO MATCH EXISTING.

DOMINION ELECTRIC KEWAUNEE
KEWAUNEE POWER STATION
CUSTOMER P.O. NO. 70153240 SAFETY RELATED

AMERICAN CRANE & EQUIPMENT CORPORATION

SINGLE FAILURE PROOF TROLLEY #2949
GENERAL ARRANGEMENT, ELEVATION VIEW

UPGRADE OF 4PS AUX. BUILDING CRANE

DRAWN BY J. PEREZ — DATE 3/28/06 — SCALE 1/4" = 1' - 0"

D-20776-0001 — REV 0

FABRICATION TOLERANCES
0 - 6'-0" = ± 1/16 — 6'-1" - 18'-0" = ± 3/16
7'-1" - 12'-0" = ± 1/8 — 18'-1" & OVER = ± 1/4

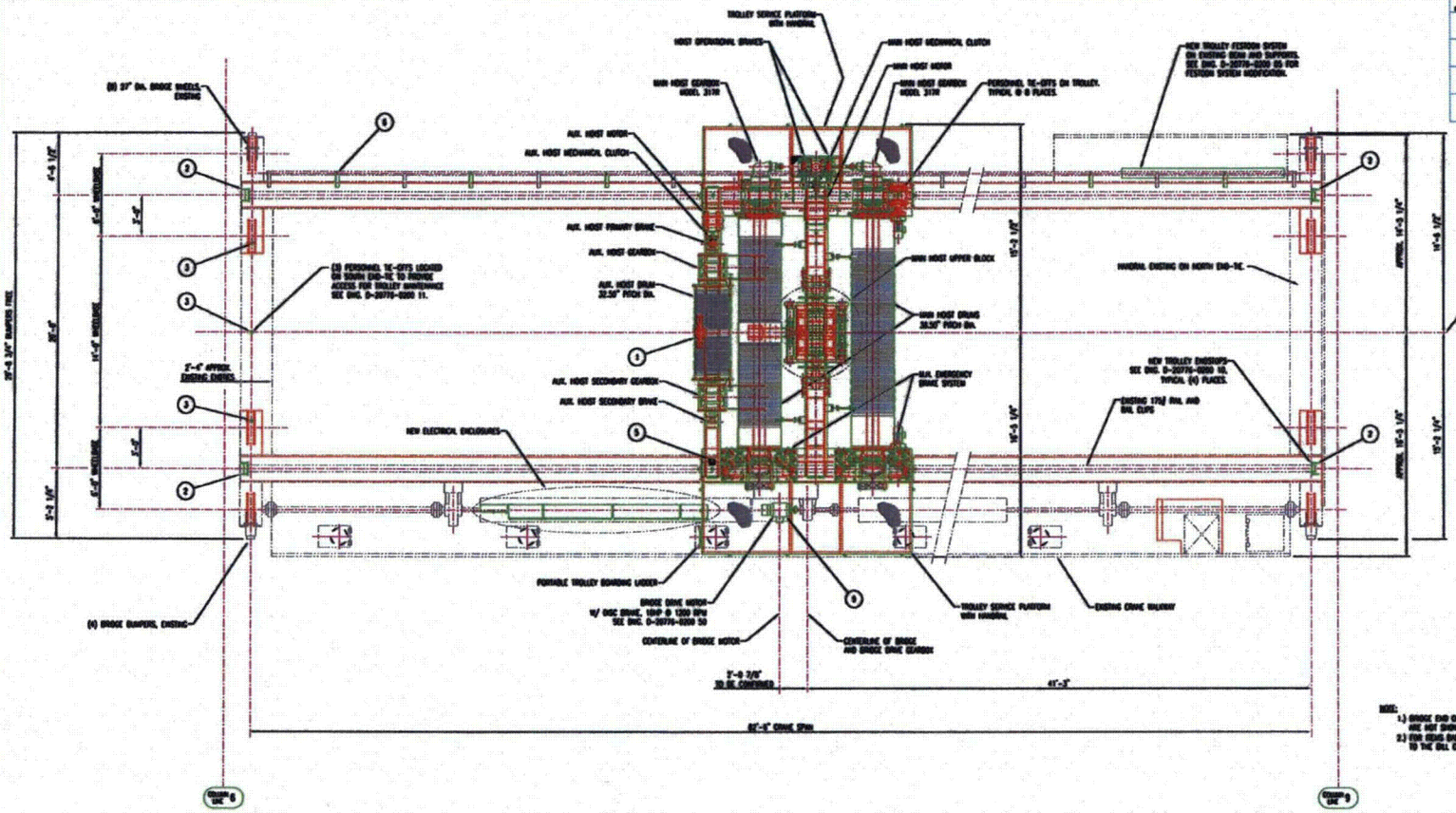
DIMENSIONS ARE IN INCHES UNLESS OTHERWISE SPECIFIED
MACHINING TOLERANCES UNLESS OTHERWISE SPECIFIED:
FRACTIONAL = ± 1/32 — ANGLES = ± 1/2°
X.XX = ± .015
X.XXX = ± .005
BREAK ALL SHARP EDGES

TOLERANCES

N/A

D-20776-0002

REVISIONS



NOTE:
1.) BRIDGE END OF TRAVEL LIMIT SWITCHES VIEW (B) ARE NOT SHOWN ON THIS DRAWING, SEE DRAWING D-20776-0200 00.
2.) FOR ITEMS DIMENSIONED ON THIS DRAWING, REFER TO THE BILL OF MATERIAL FOR DRAWING D-20776-0001.

DOMINION ELECTRIC KEWAUNEE
KEWAUNEE POWER STATION
CUSTOMER P.O. NO. 70153249 SAFETY RELATED

AMERICAN CRANE & EQUIPMENT CORPORATION

SINGLE FAILURE PROOF TROLLEY #2949
GENERAL ARRANGEMENT, PLAN VIEW

UPGRADE OF SFS AUX. BUILDING CRANE

J. PEREZ 3/26/08 1/8" = 1' - 0"

20776 CRANE CHARGEUPS

20776

D-20776-0001

D-20776-0002 0

TOLERANCES