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SEISMIC EVALUATION OF K BASIN BRIDGE CRANES
HOI-320 & HOI-418 & SUPPORTING STRUCTURE [VOL 1]

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16

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Page 1 of 2

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11. Receiver Remarks: JCS work packages 1K-96-00228 replaces bridge rail bolts and clamps for Hoists HOI-320 & HOI-418 respectively. This action will provide the basis for qualifying the cranes and hoists for a peak ground acceleration of 0.2 g 5000 year return period.				12. Major Assm. Dwg. No.: NA	
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Page 2 of 2

15. DATA TRANSMITTED

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Seismic Evaluation of K Basin Bridge Cranes (HOI-320 & HOI-418) and Supporting Structure

B. V. Winkel & S. K. Kanjilal
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U.S. Department of Energy Contract DE-AC06-87RL10930

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Abstract:

The Safety Class 1 100-K fuel storage basins are vulnerable to impact damage if a bridge crane were to fall during a seismic event. The purpose of this report is to address the adequacy of the K Basin bridge cranes to resist a seismic-induced fall. The approach used to demonstrate adequacy against falling, was to evaluate the crane structural components relative to requirements specified in ASME NOG-1, *Rules for Construction of Overhead and Gantry Cranes*. Additionally, wheel lift-off and the adequacy of the crane supporting structure, are addressed. Seismic adequacy of the mechanical hoist equipment is not addressed in this report.

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SEISMIC EVALUATION
OF THE
K BASIN BRIDGE CRANES (HOI-320 and HOI-418)
AND

SUPPORTING STRUCTURE

Issued by

ICF KAISER HANFORD COMPANY

JANUARY 1996

for the

U.S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
RICHLAND, WASHINGTON

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INDEPENDENT REVIEW CHECKLIST

Document Reviewed WHC-SD-SNF-SA-002, Seismic Evaluation of the K Basin Bridge Cranes (HOI-320 & HOI-418) and Supporting Structure (Trolley and Bridge Components)

Authors B. V. Winkel, S. A. Kanjilal

<u>Yes</u>	<u>No</u>	<u>N/A</u>	
☒	☐	☐	Problem completely defined.
☒	☐	☐	Necessary assumptions explicitly stated and supported.
☒	☐	☐	Are computer analysis applications documented in the report per EP 2.1? (code, version, hardware, input, output files)
☒	☐	☐	Is the software verification documentation either referenced or included in the report?
☒	☐	☐	Computer code error reports reviewed for impact if any.
☒	☐	☐	Data used in calculations explicitly stated in document.
☒	☐	☐	Data checked for consistency with original source information as applicable.
☒	☐	☐	Mathematical derivations checked, including dimensional consistency of results.
☒	☐	☐	Models appropriate and used within range of validity or use outside range of established validity justified.
☒	☐	☐	Hand calculations checked for errors.
☒	☐	☐	Computer code run streams correct and consistent with analysis documentation.
☒	☐	☐	Computer code output consistent with input and with results reported in analysis documentation.
☒	☐	☐	Acceptability limits on analytical results applicable and supported. Limits checked against sources.
☒	☐	☐	Safety margins consistent with good engineering practices.
☒	☐	☐	Conclusions consistent with analytical results and applicable limits.
☒	☐	☐	Results and conclusions address all points required in the problem statement.

D. A. Wallace
Reviewer: D. A. Wallace

2-6-96
Date

TYPICAL CHECKLIST FOR INDEPENDENT REVIEW

Document Reviewed WHC-SD-SNF-SA-002 App. C
Author S. Kanjilal (Supporting Structure)

Yes No N/A

☒	☐	☐	Problem completely defined.
☒	☐	☐	Necessary assumptions explicitly stated and supported.
☒	☐	☐	Computer codes and data files documented.
☒	☐	☐	Data used in calculations explicitly stated in document.
☒	☐	☐	Data checked for consistency with original source information as applicable.
☒	☐	☐	Mathematical derivations checked including dimensional consistency of results.
☒	☐	☐	Models appropriate and used within range of validity or use outside range of established validity justified.
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☒	☐	☐	Conclusions consistent with analytical results and applicable limits.
☒	☐	☐	Results and conclusions address all points required in the problem statement.

Kuen-Chuan Tu
Reviewer

2/6/96
Date

SOFTWARE APPLICATION REPORT

SOFTWARE APPLICATION

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WHC-SD-SQA-CSWD-30014, Rev. 0 (Design verification report)

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EDT 140087
WHC-SD-SNF-SA-002

DISCUSSION:

Element types used: MASS21, BEAM4, SHELL63, BEAM44, LINK8

CONTENTS

1.0	INTRODUCTION	1
1.1	PURPOSE/SCOPE OF THE EVALUATION	1
1.2	CRANE/SUPPORTING STRUCTURE DESCRIPTION	1
1.3	SEISMIC EVALUATION CRITERIA	2
2.0	SUMMARY AND CONCLUSIONS	3
3.0	CRANE EVALUATION	4
3.1	CRANE ANALYSIS APPROACH	4
3.2	CRANE ANALYSIS RESULTS	5
4.0	CRANE SUPPORTING STRUCTURE EVALUATION	11
4.1	CRANE SUPPORTING STRUCTURE ANALYSIS APPROACH	11
4.2	CRANE SUPPORTING STRUCTURE ANALYSIS RESULTS	11
5.0	REFERENCES	12

APPENDICES:

APPENDIX A: NORPAC REVIEW OF UNLOADED CRANE EVALUATION	A-1
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LIST OF FIGURES

1	105 KE/105 KW Fuel Transfer Bay Bridge Cranes (HOI-320 & HOI-418) and Adjacent Structure Layout	2
2	Crane and Trolley Locations Considered in the Analysis	10

LIST OF TABLES

1	Envelope of Crane Seismic Accelerations (5000-year, 0.2g DBE)	4
2	Summary of Unloaded Crane DBE Evaluation (0.2g, 5000-year DBE)	5
3	Summary of Unloaded Crane DBE Evaluation (0.12g, 2500-year DBE)	6
4	Loaded Crane DBE Evaluation (5000-year, 0.2g DBE)	7
5	Crane and Trolley Location Table with Related Computer Files	11

**SEISMIC EVALUATION
OF THE
K BASIN BRIDGE CRANES (HOI-320 AND HOI-418)
AND
SUPPORTING STRUCTURE**

1.0 INTRODUCTION

1.1 PURPOSE/SCOPE OF THE EVALUATION

The K Basin bridge cranes are located in the fuel storage/transfer area of the 105 KE and 105 KW buildings. The K East and K West bridge cranes are structurally identical. The cranes interface with the fuel storage basins at the north and south loadout pits. Whenever a crane is positioned over a loadout pit, the Safety Class 1 fuel storage basin is vulnerable to impact damage if a crane should fall. Previous analyses (Kanjilal 1993) were limited to a seismic evaluation of the storage/transfer area structure with the mass of the unloaded crane applied at the southernmost position of the crane travel. The seismic adequacy of the crane components was not addressed.

The primary purpose of the evaluation described in this report is to address the potential for a K Basin bridge crane to fall during a seismic event. This evaluation was prompted in response to the Discovery Unreviewed Safety Questions (USQ) K-95-0210 and K-95-0214 (Siddoway 1995). The basic assumption of the evaluation is that a crane will not fall onto the basin, if the stress levels of the crane structural members, rail hardware, and crane supporting structure (beams and columns) remain within the appropriate code allowables during a Design Basis Earthquake (DBE). The evaluation also addressed the potential for crane wheel failure and wheel lift-off.

The specified scope of the current evaluation was limited to structural components. If a hoist component (cable, drum, gear reducer, brake, etc.) failed during a seismic event, the crane payload could drop and damage the basin. As discussed in Section 2.0, increased frequency of crane usage in the future may necessitate an evaluation of the hoist components.

1.2 CRANE/SUPPORTING STRUCTURE DESCRIPTION

The layout of the K Basin cranes is sketched in Figure 1-1. Note that the crane bridge spans between Column Lines 13 and 15. The bridge rails, which run in the north/south direction, are attached to supporting beams with bolted rail clamps. The bridge rail support beams are supported by the building columns in Column Lines 13 and 15. The trolley runs on rails which are connected to the east/west bridge beams with welded rail clamps. Both the

bridge and trolley wheels are double-rimmed, which helps prevent the wheels from sliding off the tracks during a seismic event.

The original K Basin bridge cranes were designed and constructed by Moffett Engineering in the early 1950's. In the mid 1980's, a new trolley was designed and fabricated by HECO Pacific Manufacturing, Inc. Reasonably detailed information on the trolley is available on HECO Pacific drawings located in the Certified Vendor Information files (CVI 4670). However, the available CVI file information on the bridge components is very limited. Bridge details (beam sizes, etc) were obtained by performing field measurements.

Figure 1. 105 KE/105 KW Fuel Transfer Bay Bridge Cranes (HOI-320 & HOI-418) and Adjacent Structure Layout.

1.3 SEISMIC EVALUATION CRITERIA

The basic criteria document specifying facility loads for natural phenomena at the Hanford Site is Hanford Plant Standard SDC-4.1 (DOE 1993). SDC-4.1 seismic requirements are specified as a function of the component safety class. Although the cranes are not considered to be Safety Class 1, the basins are within the "zone of influence" of the cranes. Therefore, it was necessary to evaluate the cranes as three-over-one (3/1) components using the Safety Class 1 DBE loading, given in Section 3.1.2 of SDC-4.1.

The Safety Class 1 DBE specified in SDC-4.1 has a peak horizontal ground acceleration of 0.2 g's, and a peak vertical ground acceleration equal to 2/3 of the horizontal value. This DBE definition corresponds to a 5000-year earthquake (2×10^{-4} probability). However, as discussed in Section 3.4 of SDC-4.1, it is permissible to double the hazard probability (probability of 4×10^{-4} , 2500-year event, peak ground acceleration = 0.12 g's) for existing facilities with a remaining service life of less than 20 years. The 0.12g acceleration is derived from the (Tallman 1989) seismic hazard curves, using a 2500-year return period. Since the 105 KE and 105 KW fuel storage basins remaining service life is less than 20 years, the crane seismic evaluations were performed for both the 0.2g and 0.12g seismic events.

Section 3.3 of SDC-4.1 specifies that load combinations and allowable stresses "shall be in accordance with UCRL-15910" (Kennedy et al 1990). Section 4.2.3 of UCRL-15910 describes the options available for structural facilities, but does not specifically address cranes. For nuclear facilities, the industry standard for crane design is ASME NOG-1 (ASME 1989). Therefore, ASME NOG-1 was selected for evaluating the seismic adequacy of the cranes, and UCRL-15910 was used for the structural members supporting the crane.

2.0 SUMMARY AND CONCLUSIONS

The seismic structural calculations for the trolley, bridge, and supporting structure are provided in Volume 2 of this SD report. A summary of the analysis approach and results is provided in Sections 3.0 and 4.0. A brief overall summary of the crane seismic evaluation follows below.

A finite element model of the crane structural components was developed and added to the existing basin superstructure model. Using this coupled model, several dynamic seismic analyses were performed with the crane, trolley, and hoist payload in various positions. An envelope of crane seismic accelerations was obtained from the series of runs, which were used as input to a more detailed crane/trolley model. The building/crane coupled model results were also used to generate seismic member loads for the crane supporting structure.

To evaluate the margin against falling, the crane and trolley structural member loads were compared with "extreme environmental" allowables specified in ASME NOG-1 (ASME 1989). In addition to the ASME NOG-1 structural requirements, "track hopping" was evaluated by demonstrating that the dead weight loading is sufficient to prevent all trolley and crane wheels from experiencing lift-off during a DBE. The structural adequacy of the beams and columns supporting the crane was evaluated by comparing the predicted seismic member loads with DBE limits specified in UCRL-15910 (Kennedy et al 1990).

As indicated in Sections 3 and 4, positive safety margins were found for the trolley, crane, and crane supporting structure. In order to demonstrate seismic adequacy for trolley wheel lift-off and bridge rail clamp failure, it was necessary to resort to the reduced-magnitude DBE. The long-span (east/west) crane bridge beams were also found to have a slightly negative safety margin for the 5000-year, 0.2g DBE. Although it was concluded that the crane structures are adequate for the reduced-magnitude DBE, replacing the existing bridge-rail bolted clamps with higher strength clamps and bolts would significantly increase the confidence in the seismic-fall resistance of the K Basin cranes.

By assuring that the bridge crane components and supporting structure are not at risk during a seismic event, the probability of loadout pit impact damage due to a crane hoist failure can be reduced to less than credible (probability $<10^{-6}$). This is currently achieved by limiting the time the hoist is loaded when positioned above the loadout pits. For other crane positions, load drops onto adjacent flooring was evaluated, and it was concluded that the resulting loads to the basin water retention boundary would not result in significant damage.

It is anticipated that the increased frequency of crane usage for the SNF Integrated Process Strategy will result in the probability of a DBE coinciding with a load suspended over a loadout pit exceeding 10^{-6} in the future. Since the scope of the present evaluation was limited to crane structural components, a seismic evaluation of the hoist hardware will

probably be necessary as part of the safety authorization basis for the fuel removal activities.

The structural adequacy of the hoist components is covered, in part, by the 125% capacity overload test. However, as discussed below, the crane payload vertical acceleration is predicted to exceed the 25% overload during a DBE. Therefore, the hoist overload test does not qualify the hoist components for the DBE loading, nor does it demonstrate that there is an adequate margin above the DBE loading.

3.0 CRANE EVALUATION

The initial seismic analysis of the crane was limited to an unloaded crane because for the particular operating mode of concern (PUREX fuel transfer), the probability of a seismic event coinciding with a loaded crane was considered to be incredible ($< 10^{-6}$ probability). The set of calculations for the unloaded crane are contained in Volume 2 of this report. The unloaded crane calculations were reviewed both internally and by an outside crane consultant. The crane consultant review report is attached (Volume 1, Appendix A).

Since total crane usage (over the loadout pits and all other locations) increases the probability of a loaded crane coinciding with a seismic event to the credible category, a seismic evaluation of a loaded crane was also performed. The loaded crane calculations are also provided in Volume 2. Summaries of both the unloaded and loaded crane calculations follow below. As emphasized in Sections 1.0 and 2.0, the scope of the calculations was limited to crane structural components. Hoist components (cables, drums, brakes, etc.) are not addressed. Therefore, seismically-induced load drops, due to a failure of a mechanical hoist component, is not covered by this report.

3.1 CRANE ANALYSIS APPROACH

The crane supporting structure is integral with the 105 KE and 105 KW fuel storage and transfer bay building structures. Thus, the crane seismic loading is dependent on the entire building seismic response. A seismic analysis of the fuel storage/transfer bay structures had been previously performed (Kanjilal 1993). A building/crane coupled model was developed by adding crane structural elements to the existing building model.

The effect of the crane and trolley location on the coupled seismic response was addressed by making several seismic runs with the crane and trolley at various locations, as discussed in Section 4.0. For the loaded crane analysis, the suspended load was also modeled and both up and down load positions were considered. Using the predicted forces in members attached to crane model mass elements (Volume 2 supporting structure results, trolley and payload masses), an envelope of crane DBE accelerations was developed as input to the detailed crane structural evaluation.

The envelope of crane DBE accelerations, predicted by the coupled model, is provided in Table 1. Note that the maximum horizontal accelerations occur in the east/west direction, perpendicular to the crane bridge rails.

Table 1. Envelope of Crane Seismic Accelerations (5000-year, 0.2g DBE)

Crane Hoist Loading Status	a_x , g's (N/S)	a_y , g's (Vertical)	a_z , g's (E/W)	Load a_y , g's (Vertical)
Unloaded	0.551	0.281	0.905	N/A
Loaded	0.582	0.478	0.805	0.364

The Table 1 accelerations, derived from the coupled dynamic analysis, were applied to the more detailed ANSYS (Swanson 1994) finite element models of the crane/trolley structure described and developed in the Volume 2 calculations. The DBE accelerations were combined with dead weight loading. Both the plus and minus directions of the DBE accelerations were considered, including all possible directional combinations.

The focus of the crane evaluation was on failure modes which could result in the crane falling from its position (3/1 fall onto the Safety Class 1 basin). Specific failure modes considered included: (1) wheel lift-off, (2) wheel failure, (3) track and track support hardware failure, and (4) crane primary structural member failure.

3.2 CRANE ANALYSIS RESULTS

Since the crane capacity loading of 60,000 lb is almost double the crane/bridge total dead weight of 38,000 lb, the critical load case for the crane structural members is obviously the loaded condition. For wheel lift-off concerns, the unloaded crane condition is critical. For the wheels, rails, and rail clamps, the peak horizontal accelerations enveloping both the loaded and unloaded crane applications were used.

The details of the crane analysis results are provided in the Volume 2 crane calculations, including comparisons to the "extreme environmental" limits prescribed in ASME NOG-1, Section NOG-4000. Comparisons are shown for both the 5000-year, 0.2g DBE and the 2500-year, 0.12g DBE. A summary of the results is provided in Tables 2 through 4. As indicated in the previous paragraph, the focus of the unloaded crane evaluation (Tables 2 and 3) was on the transverse loads on the wheels and tracks.

Table 2. Summary of Unloaded Crane DBE Evaluation (0.2g, 5000-year DBE)

Crane Component	Max. Transverse Wheel Load (lbs)	Failure Mode	Capacity (lbs)	Margin of Safety ¹
Trolley	6217	Wheel Keeper Plate Shear	8100	0.30
		Keeper Plate Bolt Tension	12,400	0.99
		Axle Nut Thread Shear	37,100	4.96
		Wheel Rim Shear	134,000	20.60
		Rail Clip Weld Shear	8700	0.41
		Wheel Lift Off	One Wheel Lifts Off	Adequacy Uncertain
Bridge	18,469	Bridge Rail	29,800	0.61
		Wheel Rim Shear	59,000	2.19
		Rail Clamp Bolts Tension	27,100	0.47
		Rail Clamp Bolts Shear	14,300	-0.23
		Rail Clamp Bending	18,300	-0.01

¹ Margin of Safety = Allowable/Predicted - 1 (Negative margin means allowable is exceeded)

Table 3. Summary of Unloaded Crane DBE Evaluation (0.12g, 2500-year DBE)

Component	Max. Transverse Wheel Load (lbs)	Failure Mode	Capacity (lbs)	Margin of Safety ¹
Trolley	3170	Wheel Keeper Plate Shear	8100	1.55
		Keeper Plate Bolt Tension	12,400	2.90
		Axle Nut Thread Shear	37,100	10.7
		Wheel Rim Shear	134,000	41.3
		Rail Clip Weld Shear	8700	1.76
		Wheel Lift Off	No Lift Off	OK
Bridge	9420	Bridge Rail	29,800	2.15
		Wheel Rim Shear	59,000	5.27
		Rail Clamp Bolts Tension	27,100	1.88
		Rail Clamp Bolts Shear	14,300	0.52
		Rail Clamp Bending	18,300	0.94

¹ Margin of Safety = Allowable/Predicted - 1 (Negative margin means allowable is exceeded)

As discussed in the Volume 2, Appendix B calculations, the wheel/rail interface loads for the loaded and unloaded crane are about the same. For the bridge wheel/rail loads, the unloaded crane loads are higher. Therefore, the "bridge" portion of the results shown in Tables 2 and 3 conservatively cover the loaded crane application. For the trolley wheels, the loaded crane loads were slightly higher. Therefore, for the critical wheel/rail component (the keeper plate), a re-evaluation was performed. The loaded crane keeper plate analysis results are included in the loaded crane results summarized in Table 4. Note that the keeper plate safety margin remains positive for the loaded crane application.

As indicated by the Margin of Safety column in Table 4, positive safety margins were found for all listed components, with the exception of the east/west bridge beams, which are slightly overstressed for the 0.2g, 5000-year DBE. This negative safety margin is obviously resolved by the 0.12g, 2500-year DBE. As discussed in the previous paragraph, the bridge wheel/rail safety margins conservatively apply to the loaded crane, resulting in negative margins for the rail clamps and bolts for the 0.2g, 5000-year DBE (Table 2). Following the pattern set by the unloaded crane in Table 3, these negative margins are resolved by appealing to the reduced magnitude, 2500-year DBE.

Table 4. Loaded Crane DBE Evaluation (5000-year, 0.2g DBE).

Component	Stress Category	Stress (psi)	Allowable (psi)	Margin of Safety ¹
E/W Bridge Beam	Direct + Bending	1,479 (ratio) ²	1,333 (ratio)	-0.10
"	Shear	7,675	16,500	+1.15
N/S Bridge Beam	Direct + Bending	9,208	26,400	+1.87
"	Shear	4,870	16,500	+2.39
E/W Trolley Beam	Direct + Bending	23,430	26,386	+0.13
"	Shear	12,180	16,500	+0.35
N/S Trolley Beam	Direct + Bending	11,944	26,400	+1.21
"	Shear	42,750	16,500	+2.86
Trolley Wheel Keeper Plate	Shear	14,610	18,000	+0.23

¹ Margin of Safety = Allowable/Predicted - 1 (Negative margin means allowable is exceeded)

² Combined (Direct Stress)/(Allow. Dir. Stress) + (Bending Stress/Allow. Bend. Stress) < 1.333, i.e. $(f_a/F_a + f_{bx}/F_{bx} + f_{by}/F_{by}) < 1.333$.

Included in the unloaded crane calculations is a qualitative assessment of a crane or load fall onto a floor slab adjacent to the basin water retention boundary. Since the floor slabs are not structurally coupled to the basin, the drop impact loading would be transmitted through the soil. It was concluded that it was highly unlikely that the dynamic soil loading would be sufficient to damage the basin walls.

4.0 CRANE SUPPORTING STRUCTURE

Although the crane loading on the basin superstructure has some effect on the entire structure, the focus of the current evaluation was on the structural members within and adjacent to the crane support column lines (Column Lines 13 and 15). Structural members further from the crane are less likely to be affected by the seismic loading associated with the crane.

4.1 CRANE SUPPORTING STRUCTURE ANALYSIS APPROACH

One of the objectives of the crane supporting structure evaluation was to address the effects of all potential bridge, trolley, and hoist load positions. This complex issue of the effects of crane hardware position is further complicated by the lack of bridge support symmetry. That is, the crane support structure in Column Line 13 is different than in Column Line 15. Therefore, a fair degree of engineering judgement was required in order to balance the needs for a comprehensive analysis and the limited time allotted to the evaluation.

Previous seismic analyses (Kanjilal 1993) of the basin superstructure were made using lumped masses representing the unloaded crane with the crane bridge located at the south end of its travel (Column Line F) and the trolley located near Column Line 13. The present crane-related analysis considered the crane bridge at four locations with the trolley positioned at three different locations for each bridge location as shown in Figure 2. In addition, the 30-ton capacity load location was addressed by considering the hook in both the full-up position and near the floor. Horizontal seismic loads associated with the hook load were considered to be negligible in accordance with ASME NOG-1 (ASME 1989), Table NOG-4153.7-1. The negligible effect of horizontal loading from the hook load is due to the fact that the pendulum response frequency is well below the imposed seismic frequencies.

A significant difference between the current and previous seismic analyses of the superstructure is the recent reduction in the roof mass produced by re-roofing (seven lb/ft² reduction). This reduction in roof mass was conservatively ignored in the current evaluation, but in fact helps compensate for the addition of a loaded crane, at all positions, on the crane support members of the superstructure.

4.2 CRANE SUPPORT STRUCTURE ANALYSIS RESULTS

Using the basin superstructure/crane coupled model, a total of 13 response spectrum analyses were performed with the ANSYS (Swanson 1994) computer code. The crane hardware positions for each computer run are listed in Table 5.

The critical crane hardware position depends upon the specific crane support member being evaluated. For example, the slender columns above the crane support beam level, and the beams and roof trusses at roof level exhibit the highest stresses for the following crane hardware positions: (1) crane

bridge: between Column Lines A and Bg, (2) trolley: bridge midspan, (3) Hook: top. For the bridge rail support beams, the critical crane hardware positions were found to be: (1) bridge: between Column Lines A and Bg or Dg and F, (2) trolley: either west or east end, (3) Hook: top.

Figure 2. Crane and Trolley locations considered in the analysis.

TABLE 5. Crane and Trolley Location Table with Related Computer Files.

Crane Location ¹	Trolley Location	Lifted Load	Computer Input File	Computer Output File
1	A (west)	Top	FDWT1.NEW	**50.out
1	B (center)	Top	FDCT1.NEW	**51.out
1	C (east)	Top	FDET1.NEW	**52.out
1	A (west)	NO LOAD	FDWT2.NEW	**53.out
2	SLP center	top	SLPW.NEW	**42.out
2	"	bottom	SLPWB.NEW	**43.out
3	A (west)	Top	WBDT1.NEW	**40.out
3	B (center)	Top	CMBDB2.NEW	**41.out
3	B (center)	Bottom	COMBD3.NEW	**44.out
4	A (west)	Top	WBAT1.NEW	**30.out
4	A (west)	NO LIFT LOAD	WBAT2.NEW	**31.out
4	B (center)	Top	WBATC1.NEW	**32.out
4	C (east)	Top	WBATE1.NEW	**33.out

¹ See Figure 2 for crane locations.

In order to minimize the analysis effort, structural members were grouped by type (ANSYS real constant), and a conservative enveloping evaluation was performed. Individual members were considered only when necessary. This bounding approach was carried out only to the extent necessary to demonstrate seismic adequacy of each type of crane support structural member. No negative safety margins resulted from the evaluation.

5.0 REFERENCES

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APPENDIX A

NORPAC ENGINEERING REVIEW OF UNLOADED CRANE EVALUATION

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REPORT ON

**SEISMIC ASSESSMENT OF
30 TON TRANSFER CRANE, HOI-418
105KW TRANSFER BAY
P.O. #MRY-SWV-334807**

**FOR
WESTINGHOUSE HANFORD COMPANY**

**PREPARED BY
BRUCE ERIKSON**

JANUARY 8, 1996

NORPAC JOB 3325

RECEIVED

JAN 12 1996

W. A. FRIER

EXPIRES 2/22/97

WHC-SD-SNF-SA-002, Rev. 0
Volume 1NORPAC Engineering, Inc.
SEATTLE, WASHINGTON

CONCLUSIONS

Based on a review of the ICF Kaiser Hanford Engineering analysis, additional independent supporting analysis, and an onsite inspection, we find the existing crane configuration acceptable to handle the Design Basis Earthquake (2500 year), when unloaded with the trolley parked over the North or South load out pits. The bridge and trolley would remain on the travel rails along with crane stresses remaining at an acceptable level. It should be noted that analysis was not performed for other crane positions, a loaded crane, or building strength.

ONSITE INSPECTION

The onsite crane inspection was performed by the undersigned on December 7 and 8 1995, accompanied by Bill Frier of WHC, as accessible from the walkways below the crane. An additional inspection was performed on December 15, 1995, accompanied by Bob Winkel of ICF Kaiser. This supplemental inspection allowed access on the crane as the undersigned had completed the RAD Worker II training.

The crane is a 30 ton Moffett bridge crane manufactured in 1955. The trolley was replaced by a 30 ton HECO - Pacific trolley manufactured in 1984.

The basic bridge girder section is a WF 30x108 with a C15x33.9 channel cap spanning 36'-3". The trolley runs on 40# rails on the girders. The rail is held down by 3/8" x 1" welded clips with an 18" staggered spacing. The end trucks are double C15x33.9 channels at 10" back to back spacing. Sixteen inch tread diameter wheels are on 10' centers with the bridge girders on 6' - 10" centers. The wheels have an 18" flange diameter with a 7/8" flange thickness at the flange OD. The wheel is retained between the truck channels, so keeper failure would not cause the wheel to become dislodged. The crane is retained on the rails strictly by the wheel flanges, no seismic restraints have been provided. No detail drawings of the bridge were found, but a general arrangement drawing was provided (Moffett drawing # 616-1).

The bridge runs on 80# rails on wide flange runway beams. The East runway (running North and South) is a continuous WF 18x64 spanning 6 columns at 14' +/- spacing. The West runway is a continuous WF 18x96 spanning 4 columns at 23'-4" +/- spacing. The East columns are founded to concrete structure and tied to roof structures at approximately 1/3 height and at the top. The West columns are also founded in the concrete substructure but tied to roof structure only at the top. The whole structure is braced to the reactor building at the South. The rails are attached to the runway by standard rail clips on 2' centers with 3/4" diameter bolts through the top flange of the runway. General Electric drawings #H-1-21007, 21072, and 21102 were provided showing structural configuration of the building.

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The trolley is a 6'-10" span 8'-0" wheel base structure. It has four 12" tread diameter 13 3/4" flange diameter (3/4" thick) wheels riding on the 40# rails. Wheel flanges maintain the trolley on the rails. The wheel shaft is retained by a keeper to the trolley structure. A nut retains the wheel bearings to the wheel shaft. No seismic restraints are provided. HECO-Pacific drawings #C-2037-1 and D-2037-2 through 5 were provided to determine trolley structure and weight.

All components appeared to be in satisfactory condition with respect to taking seismic stresses. We would recommend, however, that all rail holddown bolts be checked for tightness, if this has not been performed recently as a part of normal maintenance procedure.

REVIEW OF ICF KAISER ANALYSIS

NORPAC performed a review of the ICF Kaiser Hanford analysis as faxed on the 38 pages dated 12/18/95 by B.V. Winkel and additional 6 pages faxed 12/27/95 by B. V. Winkel to clarify illegible copy and give ANSYS model input. Additional references used in the review were Hanford Plant Standards, Standard Design Criteria (SDC) 4.1 Rev. 12 and the 105KE/105KW Irradiated Fuel Storage Basin Seismic Qualification Report (WHC-SD-N031-SA-002).

In summary, the analysis was based on the internally reviewed and approved (ICF Kaiser) analysis for the building based on an ANSYS computer model. The bridge and trolley were then modeled and added to the previous analysis to give the coupled model loads. The input seismic response was based on a Safety Class 1; 0.2G 5000-year interval Design Basis Earthquake (DBE) assuming 7% damping on the structure. Per SDC4.1 a 0.12G 2500 year DBE is permitted for existing structures with a useful life less than 20 years. Additionally 10% damping is also permitted. This gave a load reduction factor of 0.51 [$0.12/0.2 \times 0.85$ (damping)] from loads output from ANSYS.

Possible failure modes to be investigated were as follows:

1. wheel lift off
2. rail or rail clamp failure
3. wheel failure
4. bridge structural member failure

Allowable stresses used were 1.6 times AISC allowables, except shear allowable which was 1.4 times AISC allowables.

Weight calculations were based on manufacturer's drawings, as available, and estimates. Calculated weights were rounded up to achieve the manufacturer's estimated weights of 16,000 lbs. and 22,000 lbs. for the trolley and bridge respectively.

WHC-SD-SNF-SA-002, Rev. 0
Volume 1

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The ICF Kaiser analysis showed no uplift at bridge or trolley rails, thus no uplift restraints required. Lateral loadings were given as 24,000# and 7000# for the bridge and trolley wheels respectively for the 0.2 DBE loadings.

These lateral loadings were then applied to the runway rail and clamps, bridge wheels, trolley wheels, trolley axle keeper plate and nut, and trolley rail clips. A positive margin of safety was found for all components (allowable stresses greater than predicted stresses).

The calculations have been reviewed with respect to method and a *rough* numerical check given and are approved subject to the following:

1. We understand the calculations were numerically checked internally at ICF Kaiser Hanford. The checked by signature blank needs to be signed.
2. No calculations were included for allowable stresses in bridge structural members. We understand these have been performed and stresses found to be acceptable. (Our calculations found them to be acceptable.)
3. We have not reviewed the complete computer runs for the building/crane from which the loads were generated, due to the size of the model involved. We understand the original building analysis had been internally reviewed and approved. Due to this gap in our review process, we have created a computer model of the crane and runway area for our own loads generation. See next section.

ADDITIONAL ANALYSIS BY NORPAC

To verify the loadings used in the analysis of the particular crane and rail components, NORPAC performed an simplified independent analysis to determine loadings. The analysis was performed using the finite element program IMAGES-3D. The results of the analysis (input and output) are in Appendix 1.

The transfer building portion of the total 105KW structure was modeled based on data from the above referenced drawings. (Sketches from which model was created follow this report). The 10 runway support columns were modeled with fixed bases. Storage basin attachment to the columns was modeled by the use of springs to ground as was the reactor building/rod room attachment at the South. Lateral bracing was simplified with fewer members. Spring stiffnesses were varied to find bounding values for seismic response.

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The bridge crane was modeled with rigid link offsets above the runway and appropriate members for the bridge girders and end trucks with increased densities to achieve a 22,000 pound bridge weight. Bridge locations over both pits were used. The trolley was located over the load out pits and modeled with rigid links to a node located at the trolley center of gravity, where a lumped mass of 16,000 lbs. was located. Member releases at the bridge and trolley wheels were per the requirements of ASME NOG-1.

The 0.2g DBE foundation response spectra per the Fuel Basin Seismic Qualification Report were used both horizontally and vertically. Directional combination was by square root of the sum of the squares method. Modal combination was per the requirements of NUREG 1.91 10% method for closely spaced modes.

The resulting wheel loads were all less than used in the ICF Kaiser analysis. Additionally, seismic vertical loads were all less than static loads, thus no wheel uplift occurs.

QUALITY ASSURANCE

Attached is NORPAC's quality assurance manual for the use of the finite element program IMAGES -3D along with the IMAGES-3D certificate of conformance.

Also attached is a resume for the undersigned.

SUMMARY

The crane as it exists meets the sight design requirements for a 2500 year seismic event for an unloaded condition over the North and South load out pits. The building, other than it's effect on crane loadings, other crane positions, and the loaded crane have not been analyzed.

If you have any questions please contact me.

Sincerely,

Bruce Erikson
Bruce Erikson

3325rep.doc

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\$H\$

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CALCULATED BY B. ERICKSON DATE 1/9/96

CHECKED BY _____ DATE _____

SCALE _____ 15

BUILDING MATH MODEL

57 NODES
74 BEAMS
24 SPRING EQUATIONS

NORPAC Engineering Inc.

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CALCULATED BY B. ERIKSON DATE 1/9/96

CHECKED BY _____ DATE _____

SCALE _____

TRUSS - ESTIMATED WEIGHT = 14,000# → 16K W/ BLOCKS
(CONSERVATIVE)

$$\begin{aligned}\bar{X} &= 12.2" \\ \bar{Y} &= 13.7" \\ \bar{Z} &= -16.6"\end{aligned}$$

SOUTH
GIRDER

NORTH
GIRDER

ATTACH TO NODES 42, 43, 51, 53 (SOUTH)
45, 47, 54, 56 (NORTH)

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