Omaha Public Power District
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402/636-2000

May 14, 1993
LIC-93-0146

U. S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Mail Station P1-137
Washington, DC 20555

References:
1. Docket No. 50-285
2. Letter from OPPD (W. G. Gates) to NRC (Document Control Desk) dated December 7, 1992 (LIC-92-340A)
3. Letter from NRC (S. D. Bloom) to OPPD (T. L. Patterson) dated February 4, 1993
4. Letter from OPPD (W. G. Gates) to NRC (Document Control Desk) dated March 19, 1993 (LIC-93-0081)
5. Letter from NRC (S. D. Bloom) to OPPD (T. L. Patterson) dated April 8, 1993

Gentlemen:

SUBJECT: Request for Additional Information Concerning the Fort Calhoun Station Spent Fuel Pool Rerack (TAC No. M85116)

Attached are the Omaha Public Power District (OPPD) responses to the subject NRC request (Reference 5). This request concerned the Fort Calhoun Station (FCS) spent fuel storage rack modification proposed in the Reference 2 submittal. These questions were informally addressed during an NRC Audit conducted on April 20, 1993 at the U. S. Tool & Die facility in Pittsburgh, PA. The responses provided by this letter reflect the discussions held with the NRC Branch reviewers during the audit.

If you have any further questions, please contact me.

Sincerely,

W. G. Gates

W. G. Gates
Vice President

WGG/tcm

Attachment

c: LeBoeuf, Lamb, Leiby & MacRae
J. L. Milhoan, NRC Regional Administrator, Region IV
S. D. Bloom, NRC Project Manager
R. P. Mullikin, NRC Senior Resident Inspector

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OPPD Responses to April 8, 1993
NRC Questions on the FCS Spent Fuel Pool Rerack

Question 1

The licensee, Omaha Public Power District (OPPD) utilized compression gap elements in the dynamic analyses to provide for opening and closing of interfaces such as the pedestal-to-bearing pad interface. Provide the magnitude of the largest compression force and the location of the gap element and the time when that event occurred for the case of Run I.D. DRA1MHEO.RF8 of the submittal (Reference 1). *[Note: although the computer run was named "DRALMHEO.RF8" in the NRC written questions, the correct identification is "DRA1MHEO.RF8."]*

Response 1

Maximum force in gap element = 1.208×10^5 lbs.
Location of pedestal = SW corner of Rack A1
Time of occurrence = 12.83 sec.

Question 2

Provide the details of the findings of Run I.D. DRA1MHEO.RF8 for the fuel assembly impacts against the cell wall:

a) The magnitude of the largest impact force and location of the impact in the fuel assembly and cell wall.

b) The masses and accelerations of the fuel assembly and the cell wall at that critical point,

c) The effects of the fluid between the fuel assembly and cell wall during the interactions, and

d) Provide experimental study that verifies the numerical simulation of the fuel assembly and wall interactions.

Response 2

a) The run DRA1MHEO.RF8 pertains to module A1 simulated under the following conditions.

- All storage locations loaded with intact fuel.
- Rack pedestal-to-liner interface coefficient of friction = 0.8.
- MHE seismic excitation applied in all three orthogonal directions simultaneously.

The rattling effect of the fuel assemblies inside the storage cells is modeled by five lumped masses at five equally-spaced elevations. The maximum rattling forces at the five locations and the instants of time when the maxima are reached are provided in Table 1. Note that mass number 2 (Figure 1) shows the largest value of the impact force.

Response 2 (continued)

The magnitude of the lumped masses is also provided in Table 1. The total weight of a fuel assembly is 1380 lbs.

b) The acceleration time-history of the fuel assembly lumped masses was not saved in the computer program runs and therefore cannot be provided. The NRC reviewers at the NRC Audit on April 20, 1993 agreed but asked for an estimated value. However, there is no reasonable way to provide a meaningful estimate for this value.

c) The fluid coupling between the fuel assembly and the storage cell was modeled using the classical Lagrange's formulation. The fuel assembly was modeled as a square planform body vibrating in a square planform opening to calculate the hydrodynamic and coupling coefficients. The NRC reviewers at the NRC Audit on April 20, 1993 agreed but asked for an estimated value of the fluid effects. However, there is no reasonable way to provide a meaningful estimate for this value.

d) Experimental study of the fuel assembly to cell wall interaction, to the best of our knowledge, is not available in the literature. Therefore, the impact spring stiffness has been modeled in a conservative manner to ensure that the impact forces due to the rattling phenomena are computed as upper bounds on the values which will be obtained in nature.

Question 3

Provide the largest magnitude of the fluid pressure distribution along the height of the rack during the fluid and rack interactions for each case of the 3-D single and multi-rack analyses. Also, provide results of any existing experimental study that verifies the simulation of the fluid coupling utilized in the numerical analyses. [Note: OPPD is requested to provide results of experimental study in addition to the experimental study indicated in Reference Section of the submittal. The experimental study mentioned in Reference Section does not reflect realistic rack configuration, boundary conditions, rack and fluid interaction, and dynamic input loading conditions.]

Response 3

The hydrodynamic pressure due to the relative movements between the assemblage of racks and the pool wall is available from the Whole Pool Multi-Rack (WPMR) analyses. The pressure distribution between the pool wall and the facing rack walls is utilized in pool structural evaluation and is therefore saved in the output files. The dynamic pressure time-histories on the walls for the WPMR analysis runs reported in the Reference 2 submittal are presented in attached Figures 2 through 9. Figure 10 shows the pool layout. Please note that the change in appearance of each plot beyond 10 seconds is due to the program not saving the time step to enough significant figures. The magnitudes are correct, however.

Response 3 (continued)

The single rack analysis model, by definition, assumes an artificial confinement boundary; as such, the hydrodynamic pressures from such runs have little physical import and were not saved in the computer runs.

The fluid coupling among the rack arrays and the pool walls is derived from the Kelvin circulation theorem and fluid continuity, which are two fundamental postulates in irrotational fluid mechanics. However, four simplifying assumptions, i.e., potential flow, zero fluid viscosity, small interbody gaps and planar flow, are made to apply the theory for calculating the linear fluid coupling coefficients. These assumptions are generally held to be reasonable and appropriate for fuel rack structures which feature unperforated external surfaces. The veracity of the theory was further confirmed by QA validated experiments which have been previously submitted to the NRC for the D. C. Cook plant. To the best of our knowledge, no other experimental validation of multi-body coupling theory has been conducted.

Question 4

OPPD states that the four sets of artificial time-histories were used for dynamic analyses. From the review of the response spectra, it is found that not only the response spectra calculated for each individual time-history did not envelop the licensing basis design response spectra of the final safety analysis report (FSAR), but also the average calculated response spectra generated from these time-histories did not envelop the design response spectra of FSAR. Section 3.7.1 of the standard review plan (SRP) provides a guideline that the average calculated response spectra should envelop the licensing basis design response spectra for use of multiple time-histories. OPPD is requested to demonstrate adequacy of the artificial time-histories used in the analyses (e.g., the extent of conformance to a target power spectral density (PSD) function of the artificial time-histories) or to use new artificial multiple time-histories such that the average calculated response spectra of the time-histories should envelop the licensing basis design response spectra, and perform the structural analysis.

Response 4

Each of the four time histories developed was used to regenerate corresponding response spectra. The average of the four response spectra gives new response spectra called the "averaged spectra". Based upon OPPD's interpretation of Option 2 of SRP 3.7.1, the time-histories are acceptable if the "averaged spectra" satisfy the enveloping criteria when compared to the original target spectra. The time-histories generated for the FCS analyses do satisfy the requirement of SRP 3.7.1 with respect to the comparison of "averaged spectra" with target spectra. The time-histories were each applied to a typical single rack model to determine which of the four sets gave the largest stress factors and/or displacement. This set was designated as the "controlling set". Although not specifically stated in the Reference 2 document, the earthquake set chosen as the "controlling set" was then amplified in each direction by 1.25 to conservatively ensure a bounding earthquake.

Question 5

OPPD simply states that analyses were performed for checking the integrity of the pool liner. No detailed quantitative information were provided in the submittal. OPPD is requested to provide the following:

a) Analytical approaches or methodologies,
b) Loading conditions,
c) Failure (tear and rupture) criteria,
d) Material properties used including concrete bearing strength and friction between the pedestal and liner, and
e) Provide complete summary of the findings in a tabular form.

Response 5

A summary of procedures and findings is presented below.

Analyses

ANSYS finite element analysis was used with vertical pedestal load and horizontal friction loads applied in a manner to achieve maximum stress.

Results

1. Maximum stress in the liner or liner weld is 47,659 psi.

   The ultimate stress of the material is 66,200 psi. Therefore, the margin of safety (MS) is:

$$MS = \frac{S_u}{S_{calc}} = 1.39$$

2. Fatigue Analysis

   a. Analysis follows ASME Section III Code fatigue methodology.

   b. The number of stress cycles is estimated from the time-history of highest vertical force on any pedestal.

   c. Cycle counting is used to evaluate the number of stress cycles vs. alternating stress range. The result is 29 stress cycles in one MHE. Results are corrected for Young's Modulus and ASME curves used to calculate the cumulative damage factors (CDF).

1 MHE + 5 DE events considered.
CDF = .00018

$$MS = \frac{1}{.00018} = 5555$$

Response 5 (continued)

Conclusions

1. Conservative Analysis:

   a. All seismic events occur with consolidated fuel.

   b. Friction between the slab and liner is neglected.

   c. The largest normal and friction loads on a pedestal are combined without regard for time of occurrence.

2. No fatigue problem occurs under postulated event sequence based on the fatigue analysis.

3. No tear or rupture occurs based on a comparison of maximum vs. ultimate stress.

Question 6

With respect to the fuel pool structure analyses, Table 8.5.3 of Reference 1 shows the limiting safety margin of 1.03 for a wall. OPPD concluded that the pool structure maintains its structural integrity during a critical loading combination since the safety margin is larger than 1.0. However, the calculated factor of safety is very marginal, and it could be changed depending on analytical methodologies and other parameters such as material properties used in the analyses. OPPD is requested to submit the input and output of the pool structural analyses of a slab and five walls for all four different critical loading conditions including physical dimensions and locations of the reinforcement of the pool structure for further staff review. Any technical assumptions made during the analyses should be discussed in details.

Response 6

The pool structural analysis was completed with adequate conservatism. Some key assumptions which render the results highly conservative are:

a) Consolidated fuel (2:1 consolidation) is used, with mass per canister of 2,480 lbs (this assumption essentially doubles the rattling inertial mass).

b) The transfer canal is drained. This gives higher load on the intermediate wall.

c) Temperature loading corresponding to T_A is used in lieu of T_o (abnormal instead of normal loading).

d) A lower bound value of reinforced concrete compressive strength is used even though actual core test data indicated higher strength was available.

e) A lower bound value of rebar yield strength is used which is estimated to add approximately 10% margin of safety.

Response 6 (continued)

f) No credit is taken for shear redistribution or moment redistribution due to re-bar yielding.

The last assumption has the effect of producing localized peak stress resultants. The lower bound margin of 1.03 is applicable only over a small region. The lower bound value actually occurred at the top corner of the intermediate wall between the main pool and the transfer canal and is a margin on local shear. Allowing for load distribution increases this margin to 1.26. Also note that the original local shear calculation conservatively neglected any effect of the rebar material at this location. Provision of the input and output of the pool structural analyses of a slab was waived by the NRC reviewers during the April 20th Audit.

Question 7

With respect to the analyses of the piles supporting the pool structure, Table 8.5.4 of Reference 1 shows the safety margin of 1.07 for the piles. OPPD states that the pile analysis was done using the finite element analysis program of ANSYS. However, OPPD did not provide any technical details and summary results in the submittal. From the review, it appears that the pile analysis did not take into account effects of the soil-pile-structure interactions during the dynamic loadings of the operating basis earthquake (OBE) and safe shutdown earthquake (SSE). OPPD is requested to provide complete details of the analyses (i.e., loading conditions, soil and pile properties, effects of the soils, analysis methodologies, dimensions of pile, and supporting and adjacent structures, etc.) including summary of the results (i.e., bending moments, shear and axial forces, deformations, etc. versus their corresponding allowables).

Response 7

The loading in the piling was also evaluated with several highly conservative assumptions. For example,

a) All storage cells were assumed to be loaded with consolidated fuel. This implies that the total fuel loading at the end of pool storage life under the present license amendment request will still be 50% of the value assumed in the analysis.

b) The added load due to fuel was assumed to directly load the pilings underneath the pool slab. In reality, the thick concrete slab (12 feet thick under the pool) would distribute the load over a wide area in the Auxiliary Building.

c) The increase in building overturning moment due to a seismic event was assumed to be 2.0% even though the total weight increase due to added fuel is 0.8% for intact fuel, and this added load is confined to the bottom of the building.

d) No credit for soil bearing strength was taken.

Response 7 (continued)

By relaxing Assumption (b) above and taking partial credit for the rigidity of the slab, the margin of safety in the most heavily loaded piling increases to greater than 1.1. The request to provide complete details of the pile analysis was waived by the NRC reviewers during the April 20th Audit.

Question 8

OPPD states that all computer programs utilized in performing the rerack analysis were verified in accordance with Holtec International's nuclear Quality Program (QP). Indicate whether the QP was reviewed and approved by the NRC staff. Also, indicate whether or not the QP documentation is available for staff audit.

Response 8

Holtec International's QA program has been audited by a number of nuclear utilities and utility groups, including NUPIC. The NRC staff reviewers and consultants (such as the Franklin Research Center and the Brookhaven National Laboratory) have also made technical audits with particular focus on the dynamic analysis codes in the past. Overall, Holtec's QA program, including its computer code development and validation systems, has been audited over 40 times by different organizations with 10 CFR 50 Appendix B programs and 10 CFR Part 21 reporting responsibility. In particular, the NRC and its consultant (Brookhaven National Laboratories) did a review of the codes during the Diablo Canyon ASLB hearings in early 1987 (Docket Numbers 50-275 and 50-323). This response was dicussed during the April 20th Audit.

Question 9

Describe plan and procedure for the post operating basis earthquake (OBE) inspection of fuel rack gap configurations.

Response 9

OPPD will incorporate into Section III of Abnormal Operating Procedure AOP-1, "Acts of Nature," plans to measure the inter-rack and rack-to-wall gaps at pre-selected points (control locations) subsequent to an OBE event. These control location gaps will be compared with the as-installed gaps measured and archived after completion of the rack installation.

In the event that the gaps are found to have changed, OPPD will either analytically evaluate and demonstrate the continued acceptability of the altered gap sizes, or restore the gaps to their original as-built values (within ± 1/8 inch tolerance). If analytical evaluations are used, upon completion of these evaluations a safety evaluation will be performed in accordance with the provisions of 10 CFR 50.59.

Question 10

Discuss the basis for selecting storage locations for newly discharged spent fuel versus the spent fuels which have been stored for some periods (i.e., the fuel storage location plan and the associated load distribution patterns as well as their effect on pool stress distribution and permanent deformations due to the previous spent fuel loadings, etc.).

Response 10

The unburned (fresh) fuel is restricted to storage racks designated as Region 1 in the Reference 2 submittal. The Region 2 cells can store burned fuel with a burnup restriction as provided in Section 4 of the Reference 2 document.

Since the weights of the fresh and burned fuel are nearly identical, the arrangement of the designated Region 1 and 2 cell locations has minimal consequence in the rack seismic analyses.

Question 11

Discuss the key procedures and assumptions for generation of three dimensional multi-rack model used in the analysis and basis for considering bounding cases for analysis (e.g., are the so-called bounding cases truly bounding?).

Response 11

The key procedures noted in Section 6.4 of the Reference 2 submittal and used in the generation of a three-dimensional Whole Pool Multi-Rack model can be summarized as follows:

a) Model each rack in the pool individually as a free-standing body which can execute *all* rigid motions.

b) Each rack is represented by six degrees-of-freedom.

c) All fuel assemblies in each rack are modeled as one rattling mass.

d) The impact phenomena at all potential impact locations (pedestal to pool floor, fuel to cell wall) are modeled with conservative impact springs.

The key modeling parameters which affect rack response are:

a) Pedestal-liner interface friction coefficient.

b) Cell-to-fuel assembly gap.

c) Cell-to-fuel assembly gap spring stiffness.

d) Rack pedestal to pool floor interface stiffness.

e) Non-uniqueness of acceleration time-histories.

Response 11 (continued)

f) Mass of the fuel assembly.

g) Extent of fluid coupling between the cell wall and the fuel assembly.

In all cases of the above-mentioned modeling parameters, assumptions are made wherever possible which result in exaggerating the computed response. Several assumptions used to achieve the simplified dynamic modeling contain large elements of conservatism. Examples are:

a) Randomly rattling fuel assemblies in a rack are replaced with a single composite fuel assembly, i.e., all fuel assemblies vibrate in unison.

b) No credit is taken for form drag due to fuel assembly motion in the cell or due to rack motion in the pool.

c) Fluid coupling terms are based on nominal gaps, not on variable gaps which have been shown to further reduce the response of the vibrating body.

d) Upper bound values are used for all gap spring constants, which is a proven method to amplify the overall structural response.

e) No internal structural damping in the rack is assumed.

Because of the multiple layers of assumptions imbedded in the Whole Pool Multi-Rack analysis model, the results are bounding in terms of displacements and stresses.

Question 12

Why did OPPD use the range of coefficient of friction of 0.2 to 0.8? Is this still the best range? In addition, OPPD is requested to provide the results of the dynamic analyses similar to the case of Run I.D. DRA1MHE0.RF8, but with the coefficients of friction of 0.01 and 0.99, respectively. Please summarize the two results in tabular forms similar to Table 6.7.3 of Reference 1.

Response 12

The experimentally measured values of interfacial coefficient of friction between two wet stainless steel surfaces are reported to exhibit Gaussian distribution with a mean of 0.503 and standard deviation σ of 0.125 (from Reference 6.4.1 in the OPPD Licensing Report [Reference 2]). In other words, 68.25% of all measured friction values are expected to lie between .378 and .628. Therefore, the bounding values of 0.2 and 0.8 (two standard deviations) provide a two-sided confidence limit of over 95%. Runs with values of 0.01 or 0.99 are well beyond the outliers of available friction data and, therefore, would not help to quantify the behavior of our rack structures. The request for running additional computations with friction coefficients of 0.01 and 0.99 was waived by the NRC reviewers during the April 20th Audit.

Question 13

Are the stress ratios provided for rack support assembly and the supporting concrete based on strictly linear assumption of structural behavior? Do these numbers truly reflect reality?

Response 13

The stress ratios for the rack structures are based on a linear structural behavior assumption. This is largely borne out by the calculated data which indicate that all primary stresses in the FCS racks will remain below the material yield strength even under the MHE condition. Local stresses will inevitably exceed yield strength; however, the overall rack response remains elastic.

In contrast to the racks, the concrete analysis utilizes the "ultimate strength" method which implicitly acknowledges plastic action.

Question 14

Describe the factors attributable to the large discrepancy between the Whole Pool Multi-Rack (WPMR) analyses of Chin Shan and the plant in New Jersey pertaining to the ratios of the maximum displacements between the WPMR and the single rack analyses.

Response 14

OPPD is not in a position to discuss the analysis of another plant. The NRC reviewers at the April 20th Audit agreed to this response.

Question 15

Given the statement that tracking the inter-rack gap showed that the presence of water has the effect of injecting a certain symmetry into the motion of adjacent racks, leading to a strong in-phase motion of component, try to explain the 60% increase of the maximum displacement of a WPMR analysis over that of a single rack analysis as shown in Table 1 of Reference 2.

Response 15

All single rack analyses reported in Table 1 of Reference 2 were for the assumption of out-of-phase motion of adjacent racks. This model is used to emphasize the potential for rack-to-rack impact. When all racks are considered, the symmetry in the motion tends to decrease the hydrodynamic effect on adjacent racks and, therefore, leads to an increase in displacement in this case.

Question 16

Referring to Figure 9 of Reference 2, would the statement that the computed gap amplitude of 0.1 to 0.2 inch, oscillating at approximately 2 Hz frequency, generally apply to most of the racks similar to the one analyzed? Discuss any safety significance attributable to these parameters.

Response 16

OPPD is not in a position to discuss the analysis of another plant. The NRC reviewers at the April 20th Audit agreed to this response.

Question 17

In a WPMR analysis, was a randomly selected friction coefficient based on the coefficient of friction of 0.5 with standard deviation of 0.125 at a given instant applied to all racks identically? If one were to select random coefficient for each rack at each instance, would the results be appreciably changed?

Response 17

In the whole pool analysis, each pedestal in the pool is assigned a randomly generated friction coefficient such that the friction coefficients on all the pedestals in the pool have a mean of 0.5 and lie in the band in excess of 2 standard deviations on each side ($0.2 \le COF \le 0.8$). These coefficients of friction *for each pedestal* are held constant throughout the run for reproducibility. If the pedestal coefficients were randomly changed at each instant, there would be some change in the results. Holtec's experience in running a few single rack test cases (not for any licensing application) shows that the changes are not appreciable.

Question 18

Was there any physical rack design change necessitated by the results of the WPMR analysis? If yes, describe the change(s).

Response 18

Yes, the results of the WPMR analysis reported in Section 6.7.1b of Reference 2 led to buttressing the corners of the FCS racks to harden them against inter-rack impacts. No other design changes were introduced by the WPMR analysis.

Question 19

In the WPMR analyses performed for Chin Shan and the New Jersey plant, discuss how the fluid rack interaction effects was accounted for as well as the assumptions made in the codification of the computer code used for the analyses.

Response 19

OPPD is not in a position to discuss the analysis of another plant. The NRC reviewers at the April 20th Audit agreed to this response.

Question 20

Discuss the error band associated with the use of central difference equations in the representation of the non-linear governing equation of motion.

Response 20

The successful use of central difference equations in non-linear analysis requires the choice of the time step to be below a certain critical value. Changing the time step to a smaller value allows the convergence of the solution to easily be assessed. Other schemes are assessed in the article "Solution Schemes for Problems of Non-Linear Structural Dynamics", J.F. McNamara, *J. Press. Vessel Technology*, Trans. ASME, 96, Ser J, 96-102, 1974. For a number of reasons McNamara concludes that the central difference operator can give better accuracy. The validation manual for DYNARACK was provided to the NRC during the course of the FitzPatrick (NYPA) reracking project (Docket Number 50-333). This manual contained many solutions using the central difference algorithm.

Question 21 **(Requested during April 20 audit)**

Provide the stress analysis of a bottom corner panel (unstiffened portion) of the rack with respect to plate buckling. Provide the values of the maximum computed panel stress and critical buckling stress.

Response 21

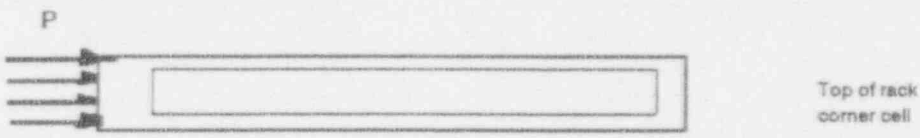
This response follows on the next page.

OPPD RESPONSE TO NRC QUESTION 21 ON LOCAL BUCKLING OF FUEL ASSEMBLY DUE TO SEISMIC LOADS

Df = \mcad\oppdnrc in ais computer 5/3/93

PROBLEM: During seismic event, corner cell is heavily loaded in compression. Demonstrate that during faulted condition, local buckling is not a problem.

RESPONSE TO NRC: The dynamic analysis gives maximum stress factors just above the baseplate level. These stress factors allow computation of actual direct stress in the cell as a function of the material yield strength. In Chapter 7 of the OPPD licensing report, a buckling calculation is carried out demonstrating structural adequacy under the assumption that only the lower 3-4" was subject to buckling. Here, to reinforce that conclusion, we present an alternate calculation which compares the calculated load in a cell wall, near the bottom, to the limit strength of the panel (based on carrying capacity after load redistribution). The figure below shows a cell wall of thickness t and width b which is subject to a known end pressure P which is derived from the baseplate pushing against the lower cell as the entire rack tends to rock.



CORNER FUEL CELL JUST ABOVE BASEPLATE - APPLIED LOADING RESISTED BY INERTIA LOADS AND ADJACENT CELLS NOT SHOWN

Material Properties E (Young's Modulus), Sy (Yield stress)

$E := 27600000 \cdot psi$ $\qquad S_y := 25000 \cdot psi$

4. Calculation of Maximum force on rack column

The ultimate load that can be carried in the cell wall is based on the load carrying capacity of thin plate sections. The reference for this work is Timoshenko and Gere, Theory of Elastic Stability, McGraw Hill, 1961, p.425. The load carrying capacity is given as Plim = C*t*t*(E*Sy)**.5 where C is a factor which depends on the thickness to width ratio and on the ratio E/Sy. The cited reference contains a curve enabling the calculation of C. For OPPD, the width and thickness of a panel section is

$b := 8.46 \cdot in$ $\qquad t := .075 \cdot in$

$x := \frac{t}{b} \cdot \sqrt{\left(\frac{E}{S_y}\right)}$ $\qquad x = 0.29456$

For the above value of x, the value of C is

$C := 1.6$

Therefore, the load capacity of any impacted cell wall is

$P_{lim} := C \cdot t^2 \cdot \sqrt{(E \cdot S_y)}$

$P_{lim} = 7.47596 \cdot 10^3 \cdot lbf$

This corresponds to a stress in the wall section of amount

$\sigma := \frac{P_{lim}}{(b \cdot t)}$ $\qquad \sigma = 1.17824 \cdot 10^4 \cdot psi$

From Chapter 7 of the licensing report, the maximum stress factor R6 in the cellular region just above the baseplate, is

$R_6 := .321$

Therefore, the calculated panel wall stress is

$S_{calc} := R_6 \cdot (.6 \cdot S_y)$ $\qquad S_{calc} = 4.815 \cdot 10^3 \cdot psi$

The margin of safety is

$MS := \frac{\sigma}{S_{calc}}$ $\qquad MS = 2.44703$

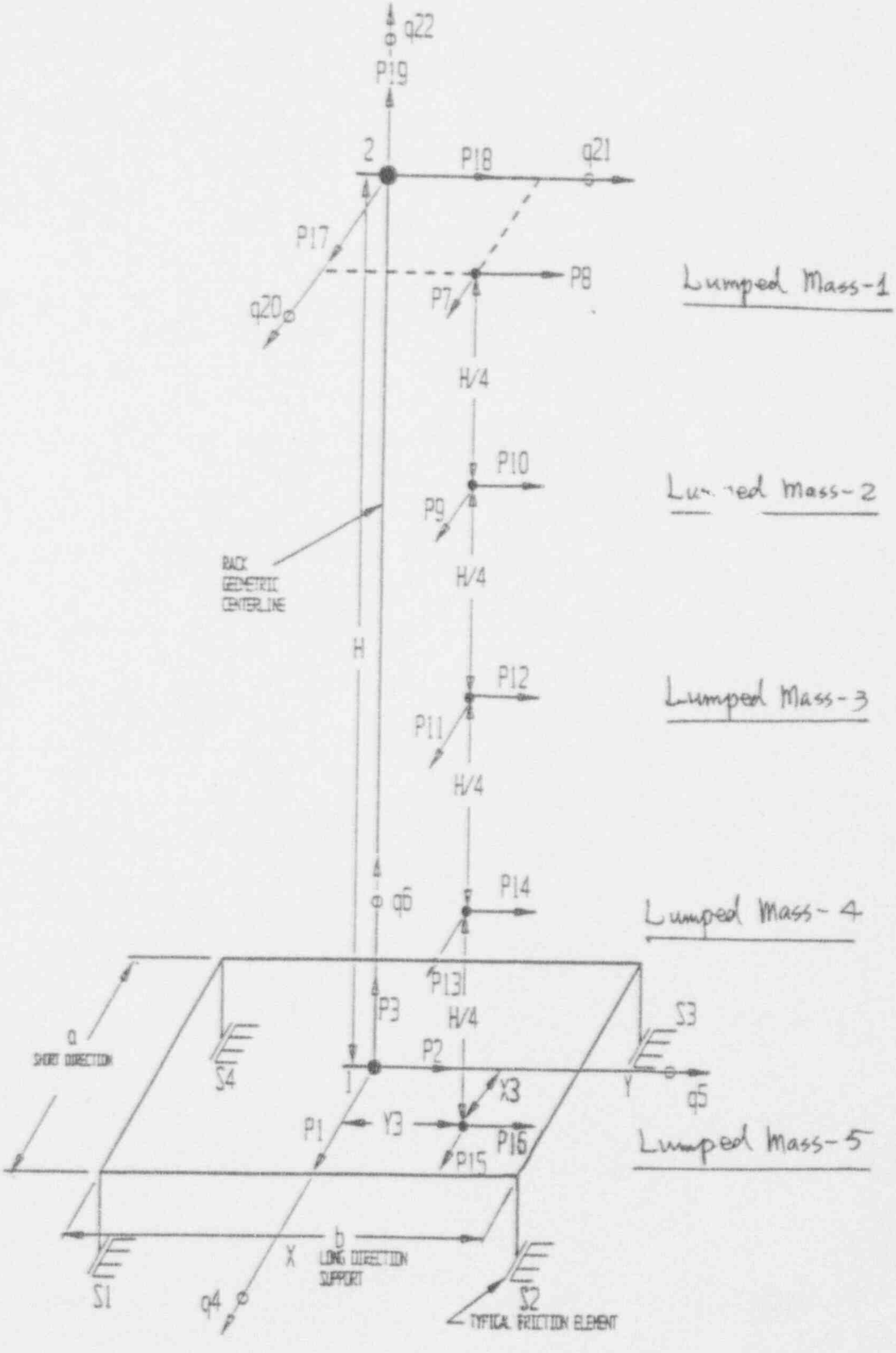
Note that for faulted conditions, it would be appropriate to use a stress higher than Sy in the calculation of Plim inrecognition of the fact that we are talking about a faulted dynamic condition here. In any event, we demonstrate that there should be no gross failure of the cell due to buckling during the faulted condition seismic event

Table 1

THE MAXIMUM IMPACT FORCES
BETWEEN FUEL ASSEMBLIES AND CELL WALLS
AT FIVE EQUI-SPACED ELEVATIONS

(from single rack analysis Run ID: DRA1MHEO.RF8)

GAP ELEMENT NO.	MAX.IMPACT FORCE (lbf.)	TIME (sec.)
Lumped Mass-1:		
5	2.899D+04	1.209D+01
6	2.867D+04	1.330D+01
7	2.196D+04	1.121D+01
8	2.260D+04	9.343D+00
Lumped Mass-2:		
9	0.000D+00	0.000D+00
10	0.000D+00	0.000D+00
11	4.059D+04	1.123D+01
12	3.959D+04	1.151D+01
Lumped Mass-3:		
13	0.000D+00	0.000D+00
14	0.000D+00	0.000D+00
15	1.273D+03	1.944D+01
16	0.000D+00	0.000D+00
Lumped Mass-4:		
17	0.000D+00	0.000D+00
18	0.000D+00	0.000D+00
19	8.669D+03	1.950D+01
20	7.333D+03	1.884D+01
Lumped Mass-5:		
21	1.056D+03	1.694D+01
22	1.796D+03	1.897D+01
23	5.357D+03	1.814D+01
24	5.964D+03	1.858D+01



SCHEMATIC MODEL FOR DYNARACK

Fig. 1

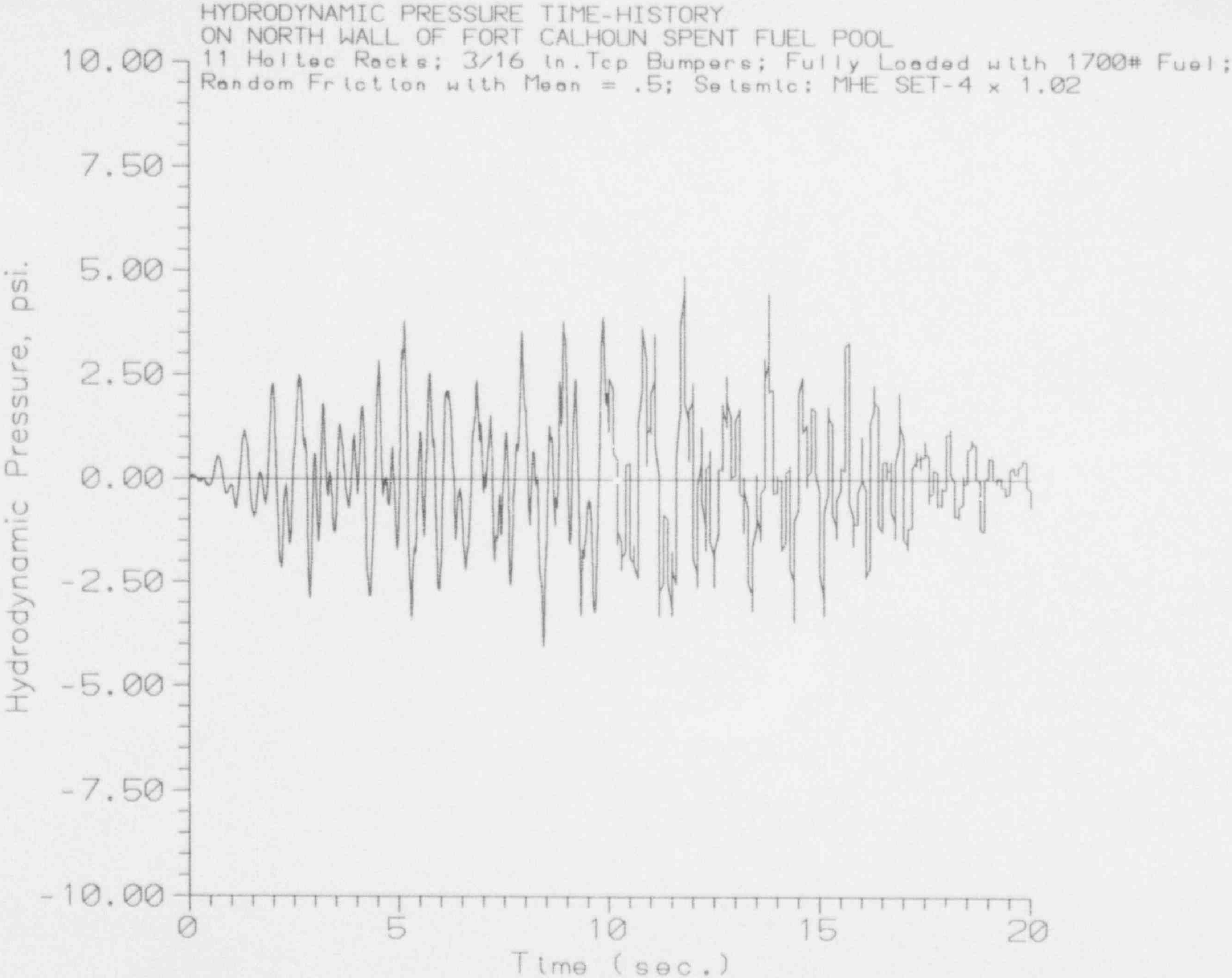


Fig. 2 Hydrodynamic Pressure on North Wall, MHE Seismic

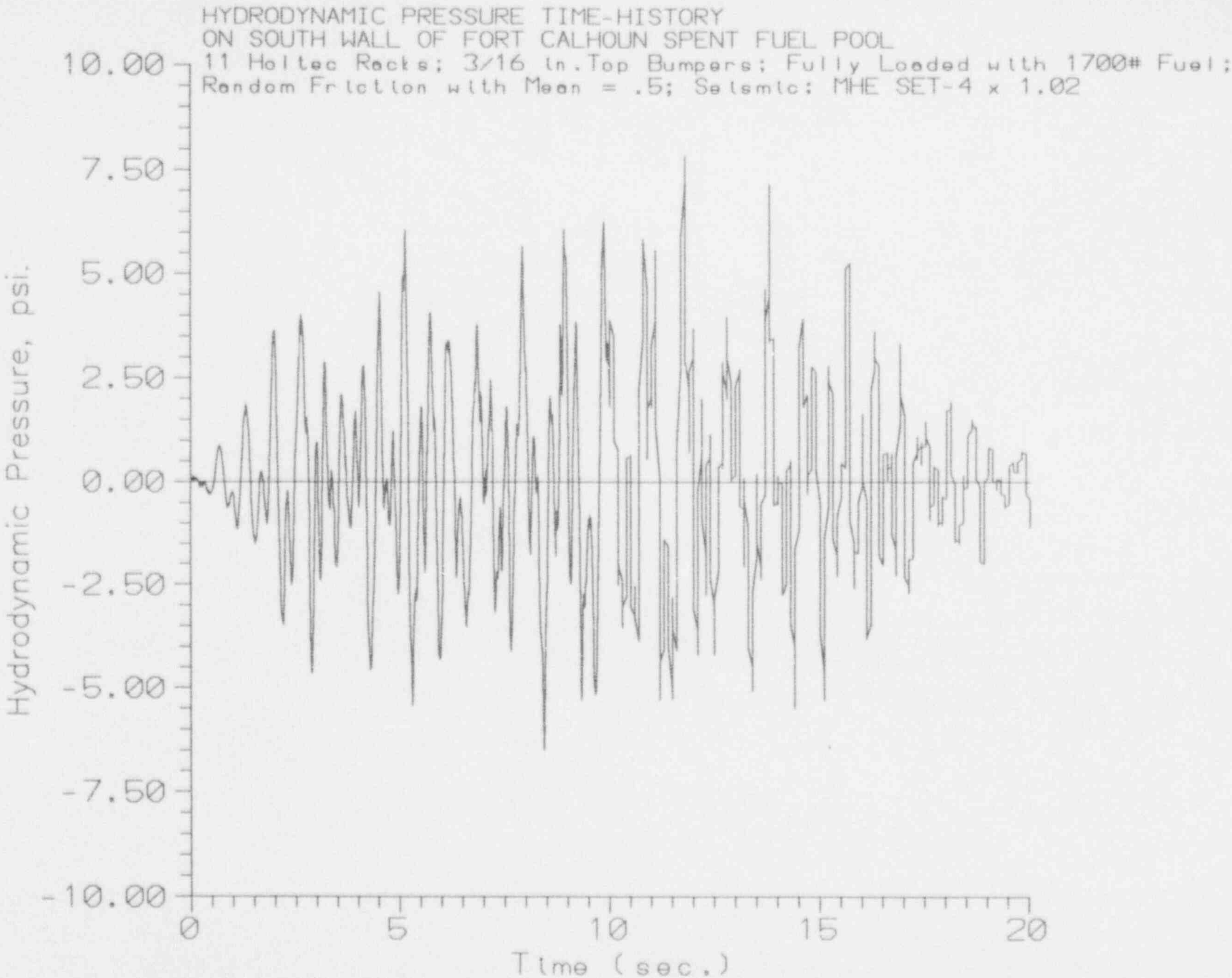


Fig. 3 Hydrodynamic Pressure on South Wall, MHE Seismic

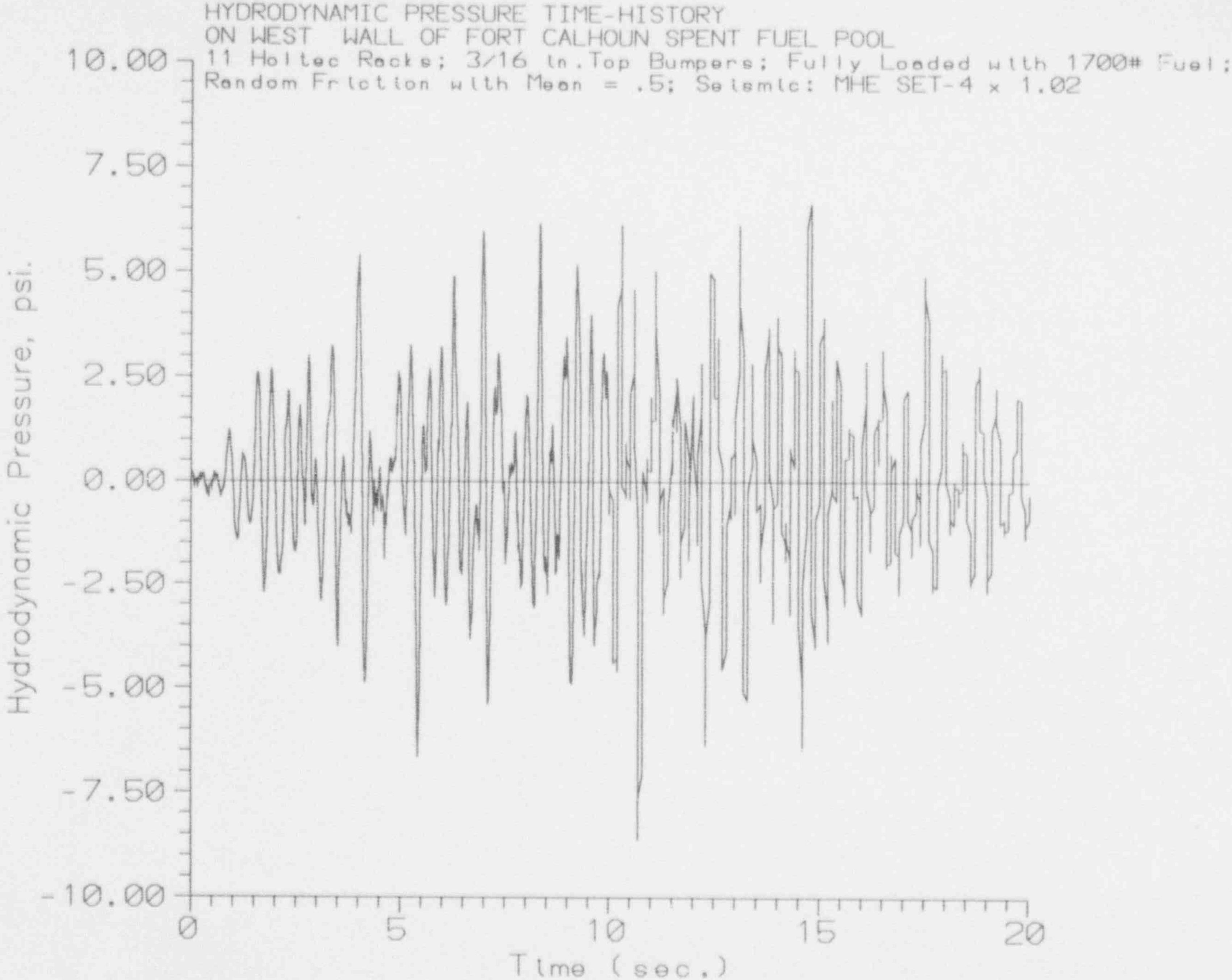


Fig. 4 Hydrodynamic Pressure on West Wall, MHE Seismic

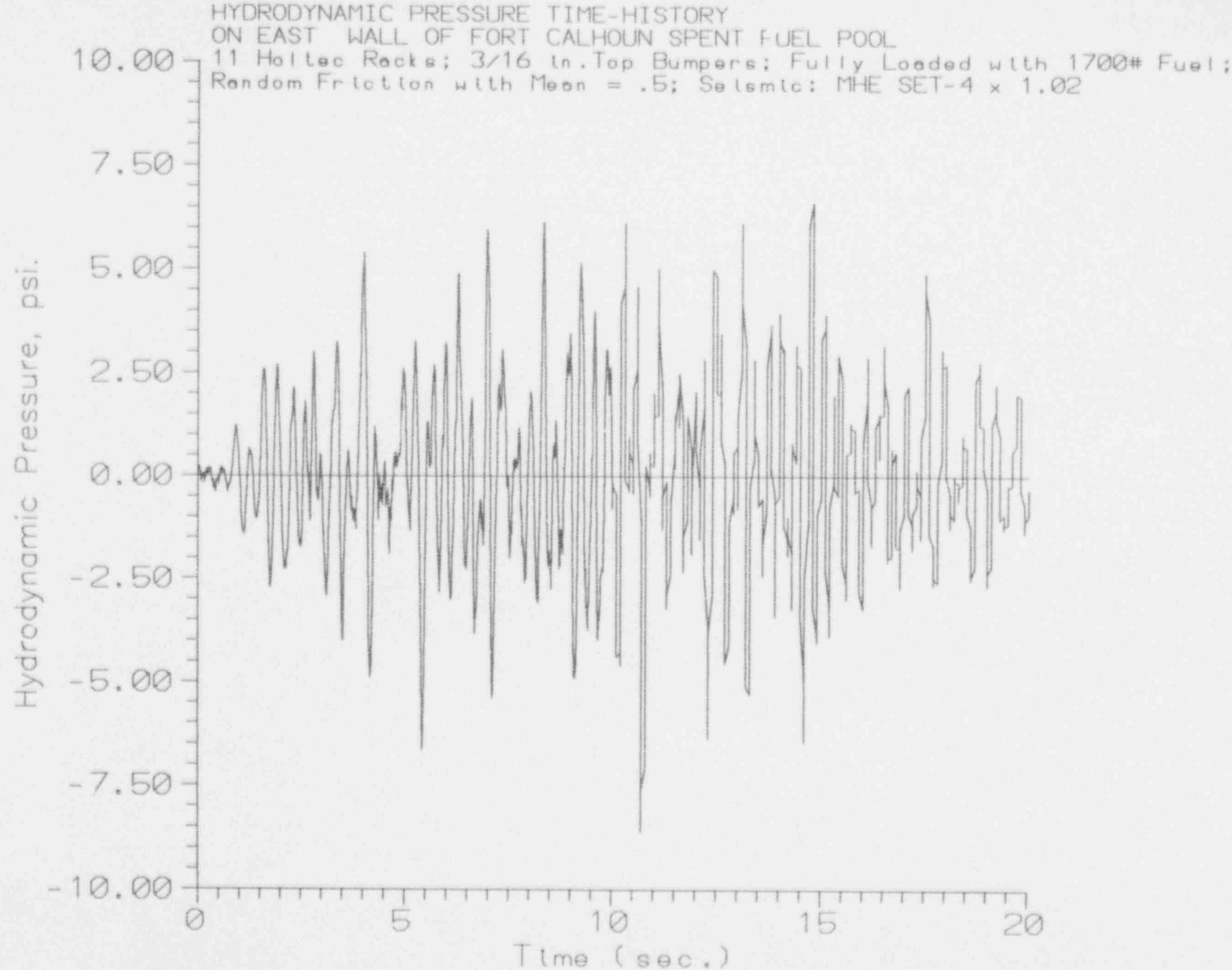


Fig. 5 Hydrodynamic Pressure on East Wall, MHE Seismic.

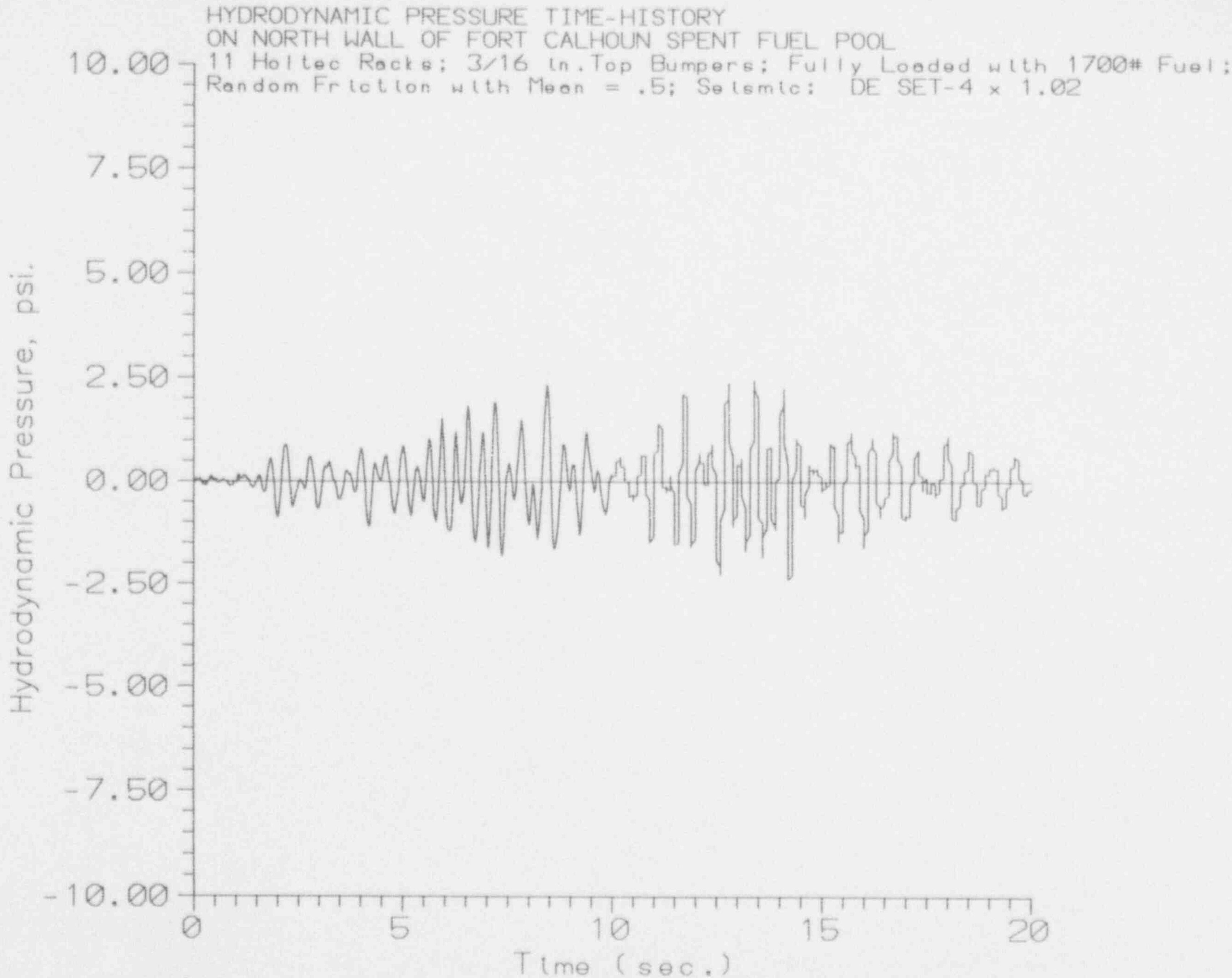


Fig. 6 Hydrodynamic Pressure on North Wall, DE Seismic

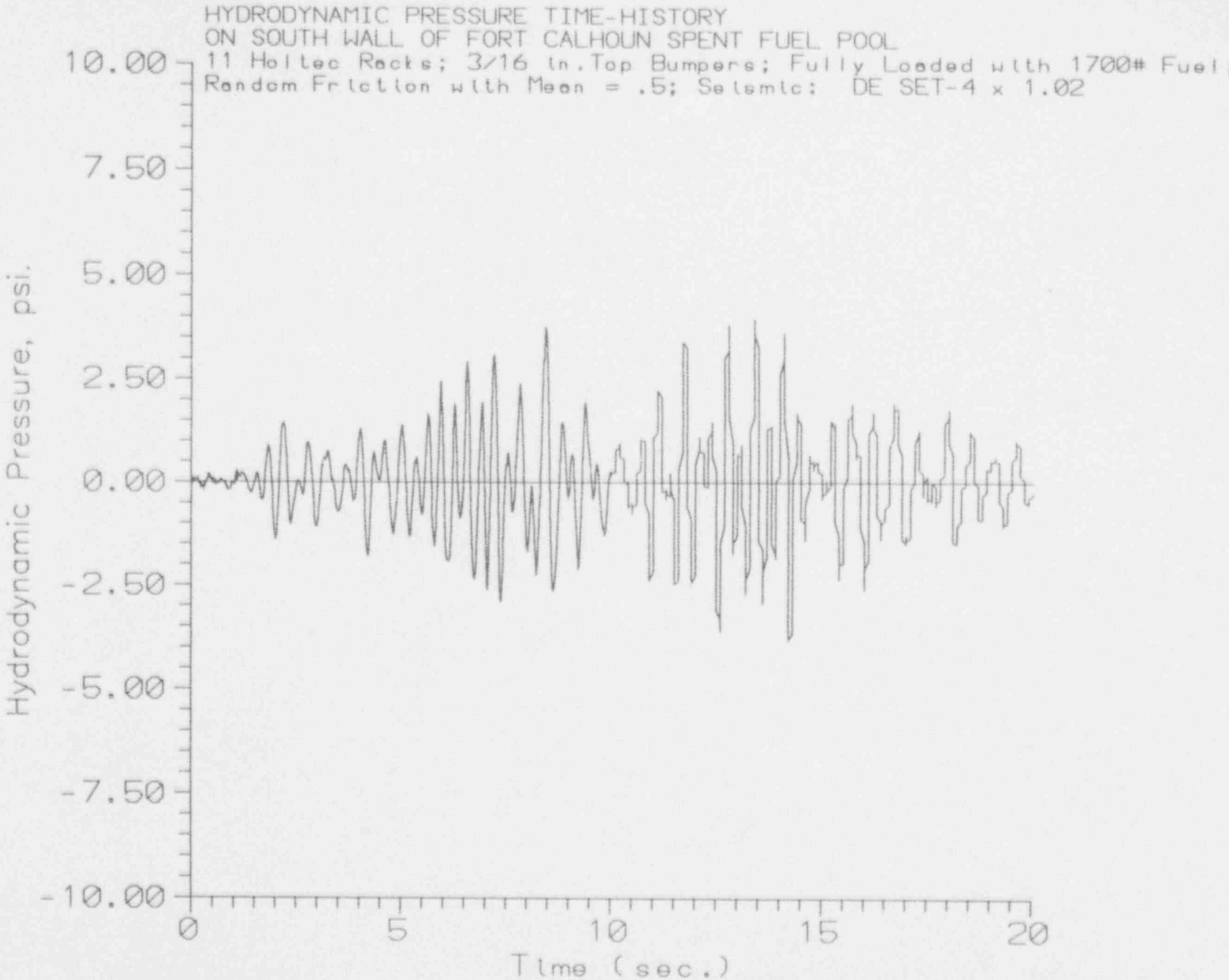


Fig. 7 Hydrodynamic Pressure on South Wall, DE Seismic.

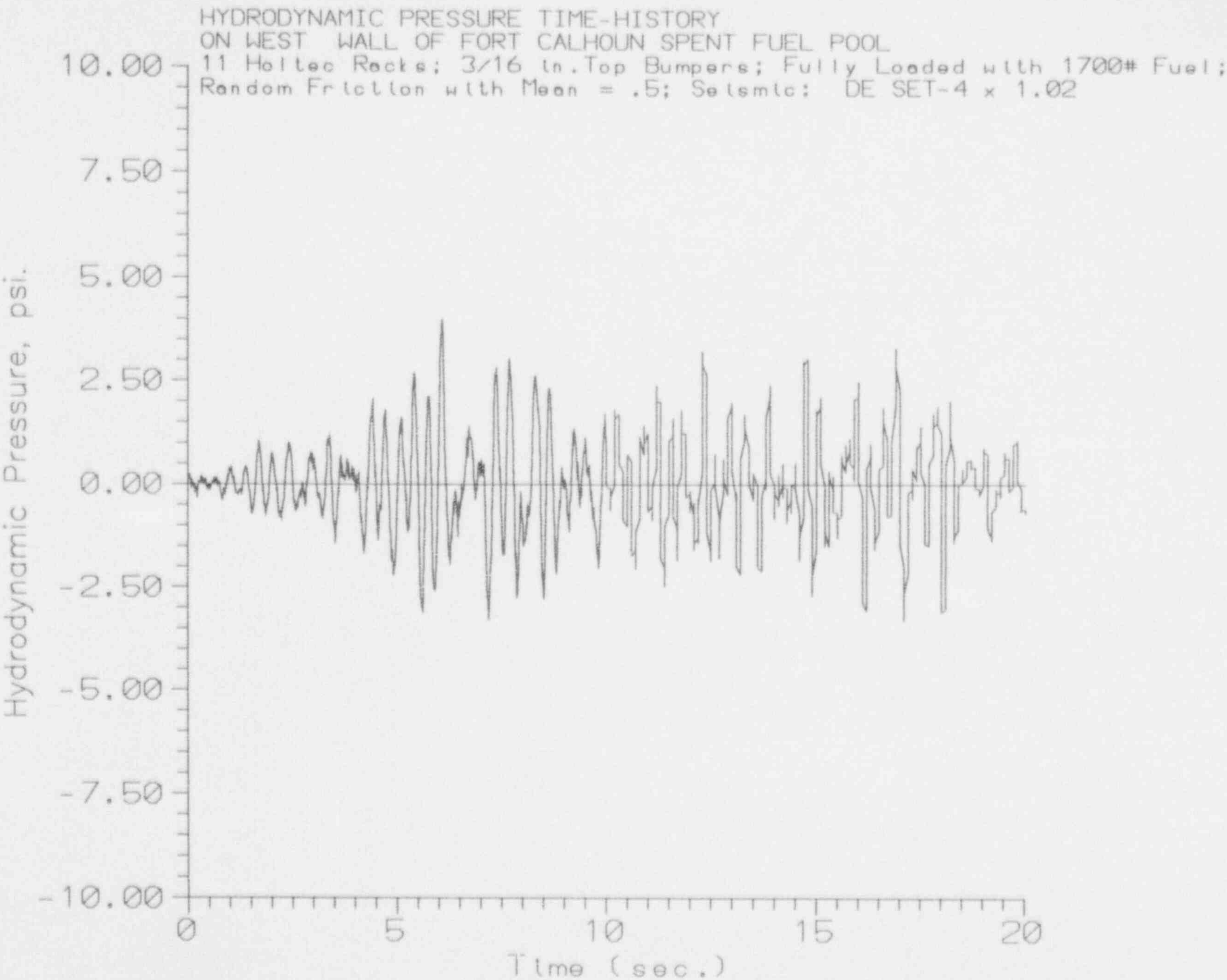


Fig. 8 Hydrodynamic Pressure on West Wall, DE Seismic

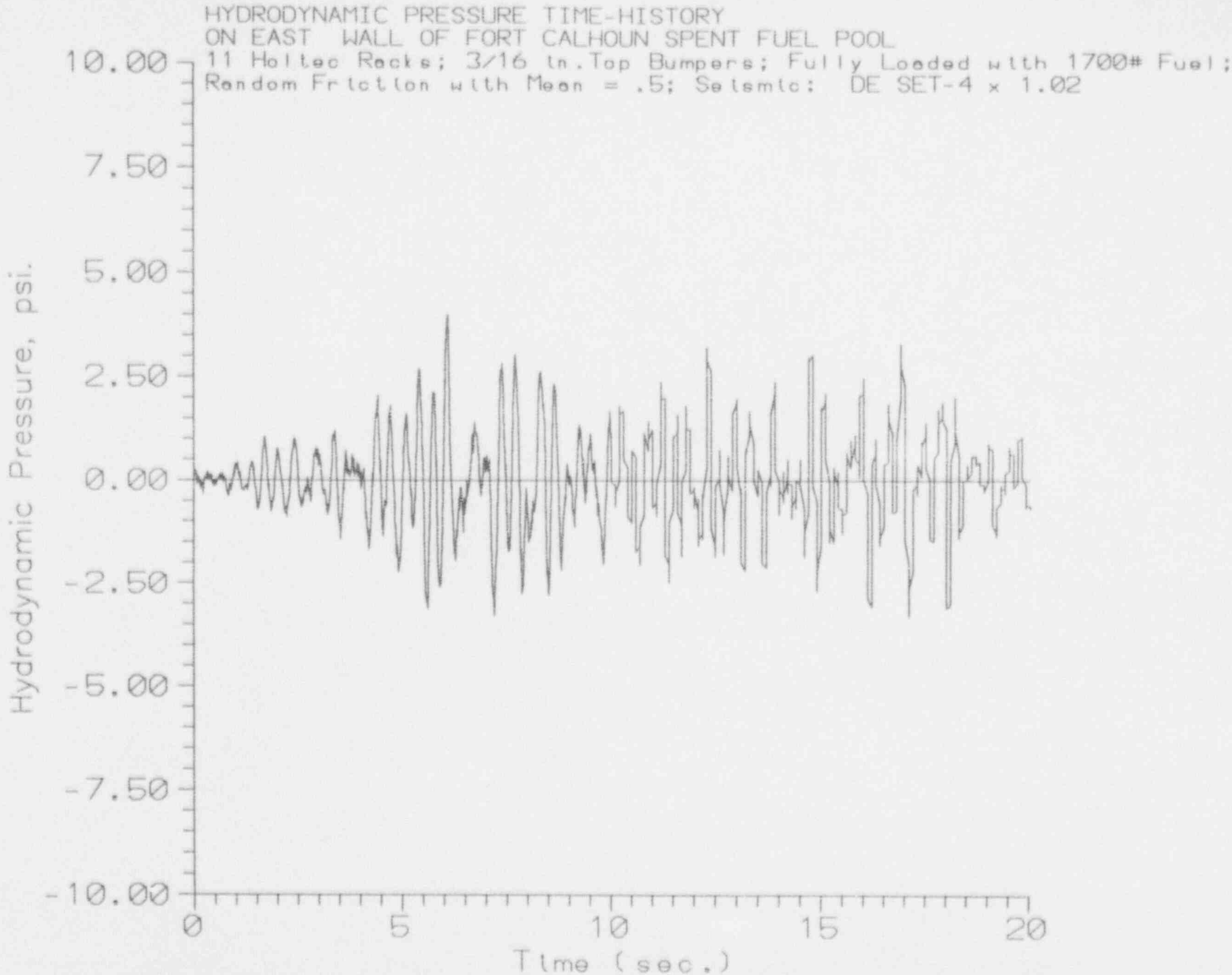


Fig. 9 Hydrodynamic Pressure on East Wall, DE Seismic.

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Attachment
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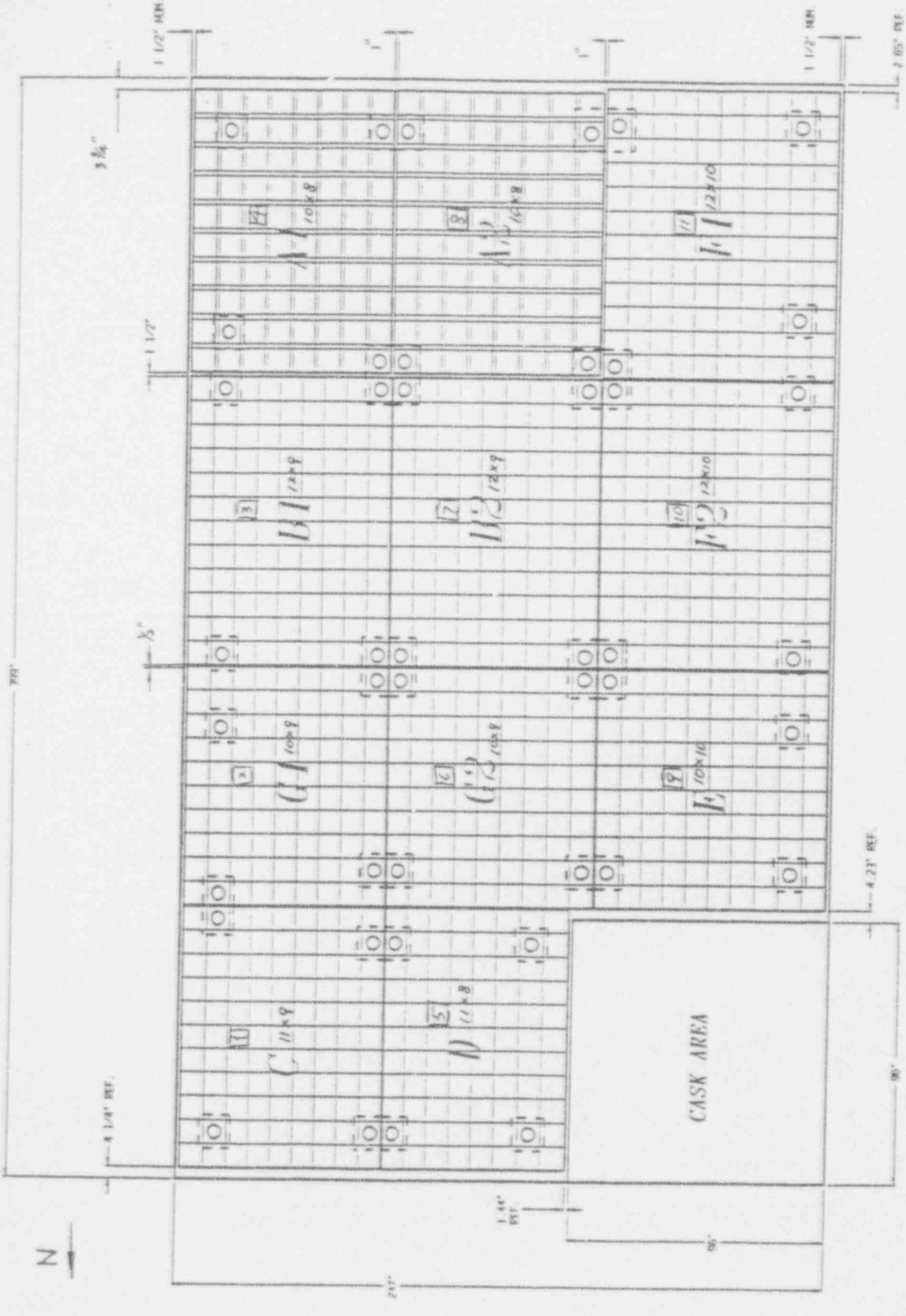

Pool Layout, FORT CALHOUN, OPPD.

Fig. 10