Tennessee Valley Authority, Post Office Box 2000, Spring City, Tennessee 37381-2000

John A. Scalice
Site Vice President, Watts Bar Nuclear Plant

MAY 2 8 1997

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555

Gentlemen:

In the Matter of	)	Docket No. 50-390
Tennessee Valley Authority	)	

WATTS BAR NUCLEAR PLANT (WBN) UNIT 1 - RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION REGARDING REQUEST FOR LICENSE AMENDMENT TO TECHNICAL SPECIFICATIONS (TS-WBN-96-010) - SPENT FUEL POOL STORAGE CAPACITY INCREASE (TAC NO. M96930)

The purpose of this letter is to provide TVA's response to the request for additional information dated May 5, 1997, from the NRC's structural engineering branch. The enclosure provides the response to the NRC's questions.

No new commitments are identified in this letter. If you should have any questions, please contact P. L. Pace at (423) 365-1824.

Sincerely,

J. A. Scalice

Enclosure
cc: See page 2

D030 1/1

030080

MAY 2 8 1997

cc (Enclosure):

NRC Resident Inspector
Watts Bar Nuclear Plant
1260 Nuclear Plant Road
Spring City, Tennessee 37381

Mr. Robert E. Martin, Senior Project Manager
U.S. Nuclear Regulatory Commission
One White Flint North
11555 Rockville Pike
Rockville, Maryland 20852

U.S. Nuclear Regulatory Commission
Region II
Atlanta Federal Center
61 Forsyth St., SW, Suite 23T85
Atlanta, Georgia 30303

ENCLOSURE

WATTS BAR NUCLEAR PLANT (WBN) UNIT 1
REQUEST FOR ADDITIONAL INFORMATION
SPENT FUEL STORAGE CAPACITY

QUESTION 1

The result of the maximum horizontal pedestal load is not given in Table 2.1 of the response to our request for additional information (RAI) question number 2. Provide it.

RESPONSE 1

The following are the maximum horizontal pedestal loads (lbf) for the single rack analysis:

Summary of limiting Results from Single Rack Analysis			
Item	OBE*	SSE	15 X 15 SSE*
Max. Horizontal Pedestal Loads (lbf)	41,600	62,000	149,000

* Evaluation used heavier (consolidated) fuel

QUESTION 2

During the telephone conference on April 24, 1997, you indicated that the results of the safe shutdown earthquake (SSE) analysis in Table 2.1 are for the rack with the dimensions 8 ft x 9 ft. Confirm that this is in fact the rack dimension.

RESPONSE 2

That is not the correct dimension. The full size racks being installed at WBN are two sizes, 7 cells wide by 8 cells long and 7 cells wide by 9 cells long. For convenience, the racks are referred to as 7 x 8 and 7 x 9, meaning the number of cells. The dimensions of the top castings are 72.625 x 83.0 inches for the 7 x 8 rack and 72.625 x 93.375 inches for the 7 x 9 rack. Tables 2.1.1 and 2.1.3 in the October 23, 1996, licensing submittal request reflect this same information. In the April 24, 1997, telephone conference it was misstated that an 8 x 9 rack was analyzed. As can be seen from Table 6.5.3 of the October 23, 1996, licensing submittal, analyses for the spent fuel pool were performed using the 7 x 9 rack. The 15 cell by 15 cell (15 x 15) rack in the cask pit area was also separately analyzed.

QUESTION 3

You indicated that the results of the SSE analysis in Table 2.1 for the rack (15 feet x 15 feet) are for the consolidated fuel load. The consolidated fuel is not related to the licensing amendment submitted. Submit appropriate analysis results with the intact fuel load for our review.

RESPONSE 3

TVA investigated the possibility of disassembling the fuel assemblies and storing the fuel rods from two fuel assemblies in one storage cell. This is termed consolidated fuel. The consideration for using consolidated fuel was discussed in Response to Question 4 in TVA's letter dated January 31, 1997. Table 6.5.3 of the October 23, 1996, licensing submittal request also identifies that consolidated fuel was used in some analyses. It is emphasized that the use of consolidated fuel was only being considered as an option and is not being requested for approval by this request for licensing amendment. An analysis was not originally performed for the 15 cell x 15 cell rack with intact fuel because the analysis with heavier consolidated fuel was considered to be more severe. This is because the weight of consolidated fuel is taken as 3000 pounds and the weight of an intact fuel assembly is taken as 1700 pounds. At the NRC's request, an analysis has now been performed using intact fuel. The table below provides the results requested. The table shows results using both consolidated and intact fuel for comparison. The table shows that the loads, displacements, and stress factors are smaller with intact fuel than with consolidated fuel. This would also indicate that the analysis of a 7 x 9 single rack with consolidated fuel, would bound an analysis with intact fuel.

Summary of SSE Results for 15 x 15 Rack

Item	Intact Fuel	Item
Max. Vertical Pedestal Load, lbf	239,000	402,000
Max Horizontal Pedestal Load, lbf	134,000	149,000
Max. Displacement at Rack Top Corner, in	0.403	0.525
Max. Displacement at Baseplate, in	0.019	0.024
Max. Fuel-to-Cell Impact Load, lbf	427	729
Max. Impact load at Rack Top Corner, lbf	0.0	0.0
Max. Impact Load at Baseplate, lbf	0.0	0.0
Max. Stress Factor - Above Baseplate (R5)	0.115	0.193
Max. Stress Factor - Support Pedestal (R5)	0.207	0.257

QUESTION 4

Provide a complete deformation shape for the rack from the bottom to the top for the single rack SSE analysis when the maximum displacement at the rack top corner equals 0.704 inch.

RESPONSE 4

The dynamic model used for single rack analysis consists of 22 degrees of freedom (DOF). The rack structure accounts for 12 DOF, and the remaining 10 DOF represent the stored fuel. The DOF are divided equally between two nodes which are located on the rack vertical centerline (i.e. 6 DOF at the baseplate elevation node and 6

DOF at the rack top elevation node). Each node is assigned 3 translational DOF and 3 rotational DOF. A complete DOF listing is presented in Table 6.5.1 of the October 23, 1996, licensing submittal. A series of springs, which represent the stiffness properties of the fuel rack, are used to couple the nodes. A schematic of the dynamic model is shown in Figure 6.5.1 of the October 23, 1996, licensing submittal.

The maximum rack top corner displacement obtained from single rack analysis under SSE conditions is -0.704 inches. This displacement is in the local x-direction and occurs at 24.97 seconds. At any instant in time, the displacement of the rack is fully described by its 12 rack DOF. Inspection of the DOF values at the time of maximum top corner displacement reveals that the dominant rack motion is rigid body motion. A combination of rigid body translation in the x-direction and rigid body rotation about the y-axis accounts for 91.6% of the maximum rack top corner displacement. Figure 1 (attached) identifies the significant DOF at the time of maximum corner displacement. The values of these DOF are:

$$x_1 = -\ 2.93 \times 10^{-2} \text{ in}$$

$$x_2 = -\ 6.99 \times 10^{-1} \text{ in}$$

$$\Theta_1 = -3.74 \times 10^{-3} \text{ rad}$$

$$\Theta_2 = -\ 3.86 \times 10^{-3} \text{ rad}$$

The rigid body displacement at the top of the rack in the x-direction (x_{top}) is calculated as:

$$x_{top} = x_1 + h\Theta_1$$

where h equals the height of the cells above the baseplate (h = 164.5 in). Substituting the values for x_1 and Θ_1 above, x_{top} equals -0.645 inch. The remaining -0.059 inch of the maximum corner displacement is due to beam-type bending of the rack cells, torsion of the rack cells and rigid body rotation about the vertical z-axis. The sum total of these three components accounts for only 8.4% of the total displacement.

To confirm this rigid body motion, the calculated forces in the rack pedestal springs are checked at the time of maximum corner displacement. For identification, the pedestals are numbered 1 through 4 in Figure 1. The calculated forces in the pedestals are:

Pedestal Number	Force, lb
1	0
2	0
3	23,400
4	101,000

As expected, two pedestals lose contact with the pool liner as the rack tilts in the negative x-direction. This indicates the rack behaves as a rigid body.

QUESTION 5

The result of the maximum displacement at the baseplate is not given in Table 2.2 of the response to our RAI question number 2. Provide it.

RESPONSE 5

Table 2.1 of the TVA's response dated February 24, 1997, shows displacements at the baseplate for the single rack. Since these displacements are very small, it was felt that baseplate displacements would not be of interest for the final whole pool multi-rack configuration. However, per the NRC's request, the maximum displacements at baseplate for the whole pool multi-rack analysis are 0.0578 inch for OBE and 0.1652 inch for SSE.

QUESTION 6

Provide a complete deformation shape of the rack for the multi-rack SSE analysis when the maximum displacement at the rack top corner equals 1.394 inches.

RESPONSE 6

The determination of rack displacement for the multi-rack is similar to the determination for the single rack as discussed in Response to Question 4 above. The maximum rack top corner displacement obtained from multi-rack analysis under SSE conditions is 1.394 inches. This displacement is in the local y-direction for a baby rack and occurs at 27.98 seconds. Similar to the single rack, the dominant rack motion is rigid body motion. A combination of rigid body translation in the y-direction and rigid body rotation about the x-axis accounts for 98.4% of the maximum rack top corner displacement. Figure 2 (attached) identifies the significant DOF at the time of maximum corner displacement. The values of these DOF are:

$y_1 = 9.56 \times 10^{-2}$ in $\qquad \Theta_1 = -7.76 \times 10^{-3}$ rad

$y_2 = 1.39$ in $\qquad \Theta_2 = -7.80 \times 10^{-3}$ rad

The rigid body displacement at the top of the rack in the y-direction (y_{top}) is calculated as:

$$y_{top} = y_1 + h\Theta_1$$

where h equals the height of the cells above the baseplate (h = 164.5 in). Substituting the values for y_1 and Θ_1 above, y_{top} equals 1.372 inches. The remaining 0.022 inch of the maximum corner displacement is due to beam-type bending of the rack cells, torsion of the rack cells, and rigid body rotation about the vertical z-axis. The sum total of these three components accounts for only 1.6% of the total displacement.

To confirm this rigid body motion, the calculated forces in the rack pedestal springs are checked at the time of maximum corner displacement. For identification, the pedestals are numbered 1 through 4 in the attached Figure 2. The calculated forces in the pedestals are:

Pedestal Number	Force, lb
1	0
2	15,100
3	17,100
4	0

As expected, two pedestals lose contact with the pool liner as the rack tilts in the positive y-direction. This indicates the rack behaves as a rigid body.

QUESTION 7

Indicate whether the rack dimension of 8 ft x 9 ft and the assumption of consolidated fuel are used for the multi-rack analysis.

RESPONSE 7

As discussed in Response to Question 1 above, the rack sizes and dimensions are given in Table 2.1.3 of the October 23, 1996, licensing submittal request. Intact fuel was used for the multi-rack analysis. It is also confirmed that the hydrodynamic pressures in Table 2.3 of TVA's letter dated February 24, 1997, are based on intact fuel. Consolidated fuel was used only in the initial stages of TVA's investigations. After that early stage of analysis, TVA decided not to pursue consolidated fuel as a storage option at this time.

QUESTION 8

You indicated in Table 4.1 that the elastic modulus of 2.76E+06 psi was used for the stainless steel material in the rack analysis and design. It is a very small modulus for a stainless steel. Confirm that this is in fact the actual material property of the steel.

RESPONSE 8

The value 2.76E+06 is a typographical error. The value used in the rack analysis was 2.76E+07. This number is consistent with the material properties presented in Table 6.4.2 of the October 23, 1996, licensing submittal request.

QUESTION 9

You stated in the January 14, 1997, meeting at NRC that the WBN Unit 1 does not have plug welds on the spent fuel pool liner and that all liner plates were welded to the embedded channels in the concrete. However, you corrected this in the response to our first RAI by stating that WBN used plug welds on the wall liner plates. Provide analysis results that demonstrate that there is no local buckling on the liner plate or a liner plate separation from the weld plug under the maximum temperature loading condition during an accident.

RESPONSE 9

The maximum design water temperature for the spent fuel pool is approximately 159 degrees Fahrenheit (F), as described in the October 23, 1996, licensing submittal. This condition occurs with the pool at maximum capacity with one train of cooling operating. For the purpose of this response, an evaluation was performed based on an accident temperature of 212 degrees F resulting from loss of both trains of cooling and subsequent boiling in the pool. (The Watts Bar Final Safety Analysis Report (FSAR) states in Section 9.1.2 that a loss of pool cooling accident is not considered a credible accident because the pool cooling system is Seismic Category I and single failure proof.)

As stated in the FSAR, the purpose of the liner is leak tightness. Other than bearing loads transmitted through the floor to the 25-foot thick basemat, no significant loads are carried by the liner and no structural function is served. Individual plates comprising the liner are welded full length along each edge to leak chase channels which are part of the building structure. The wall plates also have an array of plug welds on approximately 24-inch centers.

Due to the symmetric arrangement of the plug welds, the loads resulting at a given weld due to restraint of free thermal expansion of the liner will be balanced (i.e., no net shearing force) as long as the plate segments surrounding the weld are either all unbuckled

or all buckled. In predicting the response of the liner at plug weld locations, buckling was assumed to occur once the local buckling limit had been exceeded. The worst case loading combination on the plug welds was found to occur due to the load imbalance created when buckling was assumed on one side of the plug weld with no buckling on the other side. The load from the non-buckled side was combined with a calculated pull-out load from the buckled side. Under this accident (faulted) condition, calculated stresses in the weld were found to remain below allowable stresses for the weld material. This approach is consistent with industry practice in evaluating faulted plant conditions and is based on consideration of the inherent ductility and fracture toughness of the stainless steel material.

These results indicate that the spent fuel pool liner will retain leak tight integrity under an accident condition which results in pool boiling.

QUESTION 10

In the response to our first RAI, you did not clearly describe how the floor liner plates were attached to the concrete base slab. Your response seems to indicate that the liner plates are not welded to the channels embedded in the concrete slab except the plates near to the concrete walls. If that is the case, you are requested to provide analysis results that demonstrate that there is no buckling problem for the floor liner plates under the maximum temperature loading condition during an accident.

RESPONSE 10

The response to Question 9 in TVA's letter dated February 24, 1997, states, "The liner plates on the floor of the spent fuel pool are not positively attached to the concrete." Vertical loads in the pool are transferred to the 25-foot thick concrete floor slab by bearings. Therefore, positive attachment to the concrete through plug welds, studs, embedded plates, or shear bars is not necessary. Since the liner does not serve as a load carrying element, any buckling which might occur would be self limiting and would not impair the ability of the liner to perform its intended function.

Also, at each edge of a liner plate, either floor plate or wall plate, there is a leakage channel. The drawing excerpt provided in the above referenced response and labeled as Figure 9.1 clearly shows how the liner plates are attached to the embedded channel. Since the figure did not delineate between floor and wall, it should be clear that the detail applies to both locations.

ENCLOSURE

WATTS BAR NUCLEAR PLANT (WBN) UNIT 1
REQUEST FOR ADDITIONAL INFORMATION
SPENT FUEL STORAGE CAPACITY

ADDITIONAL QUESTIONS

The following additional questions were discussed in a teleconference on May 23, 1997.

QUESTION PART a AND b

The result of the maximum vertical pedestal load is given in Figure 2.2 of the submittal as a response to our request for additional information question number 2. The figure looks to indicate that there are negative vertical pedestal loads during an SSE. Explain the following:

a) Indicate whether there has been a negative vertical pedestal load during an SSE in your analysis.

b) If there has been, does it indicate a separation (lift-off) of the leg from the floor?

RESPONSE

Figure 2.2 in the February 24, 1997, letter shows the pedestal force diminishing to zero which indicates lift-off. The pedestal force can never be tensile. Each rack pedestal is modeled as a compression-only spring with an equivalent stiffness constant. When the spring compresses, it behaves according to the linear equation,

$$F = -k\delta$$

where positive F is the compressive force in the pedestal and δ is the spring extension. When δ is positive, the restoring force in the pedestal spring is set equal to zero (i. e., tensile pedestal forces cannot occur). This spring definition mirrors the actual behavior of a freestanding rack pedestal, where the pedestal is capable of separating from the pool liner during a seismic event.

In Figure 2.2, the spring force is plotted versus time for a typical rack pedestal. At time zero, the compressive force in the pedestal is equal to one quarter of the total buoyant weight of the rack and the stored fuel. As time elapses, the rack responds to the seismic accelerations, and the force in the pedestal begins to oscillate. When the compressive force in the pedestal decreases to zero, this indicates that the pedestal has momentarily separated from the pool liner. During this loss of contact, the force in the pedestal remains zero. When contact is regained, a compressive load again develops in the pedestal. This sequence of pedestal lift-off and liner impact may be repeated many times during the seismic event.

QUESTION PART c

c) What is the maximum magnitude of the lift-off?

RESPONSE

The maximum vertical lift-off of any rack pedestal is 0.461 inches for the SSE condition. For the OBE case, the maximum lift-off is 0.213 inches. These two results correspond to rack numbers 20 and 32, respectively, in Figure 6.5.6 of the October 23, 1996, licensing submittal.

QUESTION PART d

d) Describe how your program, DYNARACK, handles the leg separation (lift-off) and calculates the leg impact forces.

RESPONSE

Each rack in the pool is modeled as a mass/spring system with 16 degrees of freedom (see Figure 3 attached). As discussed in section 6.5.7 of the October 23, 1996, licensing submittal, the computer code DYNARACK uses a central difference scheme to solve the equations of motion for each degree of freedom. This numerical method, as it pertains to the rack pedestals, can be described as follows. The displacement of each degree of freedom is calculated for a small time step (e. g., 0.0001 sec). The extension, δ, of any pedestal can be expressed in terms of the current values of P3, Q4, and Q5 in Figure 3. Based on these incremental displacements, the force on each pedestal spring is determined from the equation $F = -k\delta$. If the force, F, is tensile, then F is automatically set equal to zero. These pedestal forces are used in the next time step to compute new displacements. This calculation scheme is repeated until the final time is reached.

A pedestal is separated from the liner at all time steps where the spring is extended (i. e., δ is positive). Pedestals are in contact at time steps where the extension of the spring, δ, has a negative value. The compression force at any time during contact is proportional to the magnitude of δ.

QUESTION PART e

e) Did you experience any numerical instability and/or convergency problems during the analysis? If you did, how did you handle the problem?

RESPONSE

No instability or convergence problems occur. The central difference algorithm requires a certain minimum time step for stability; below

that, the convergence of the solution is dependent upon the size of the time step. The numerical solution converges as the time step approaches zero. Based on our previous experience with the computer code DYNARACK, a time step of 0.0001 seconds has been used in the present analysis.

QUESTION PART f

f) Does Figure 2.2 (Run #1) show the worst case of the vertical pedestal load?

RESPONSE

No. Figure 2.2 does not show the largest vertical pedestal load.

QUESTION PART g

g) Provide a figure that shows complete and negative vertical pedestal loads for the worst case.

RESPONSE

Figure 4 attached shows the complete time history plot of vertical pedestal load for the worst case.

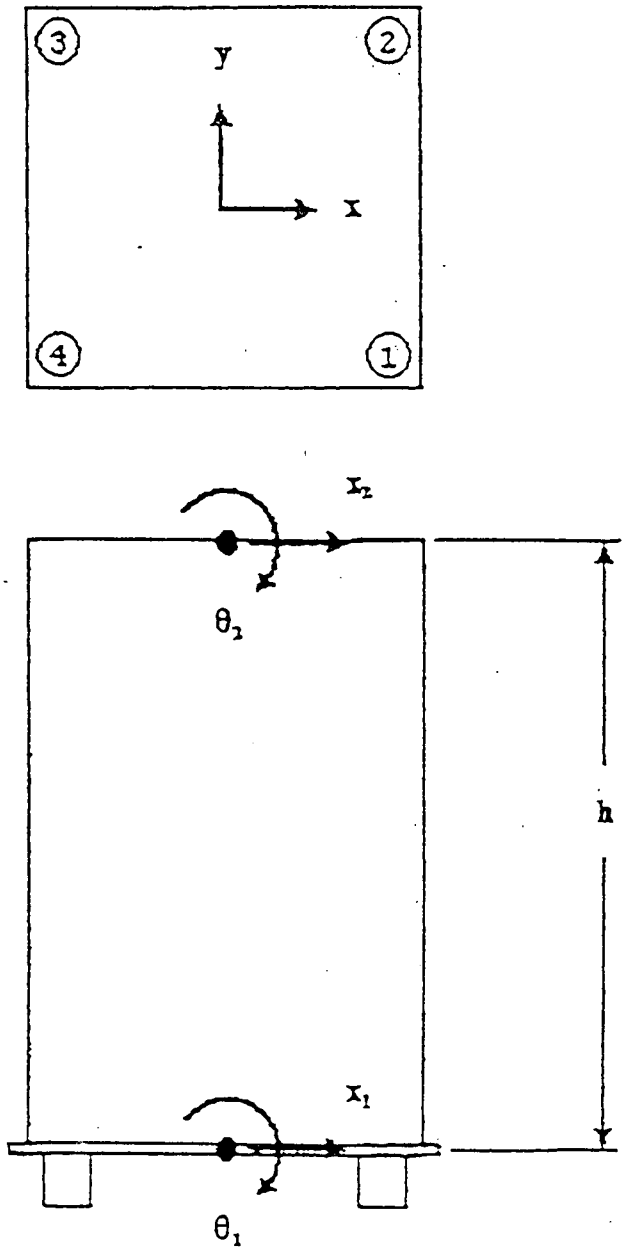

FIGURE 1

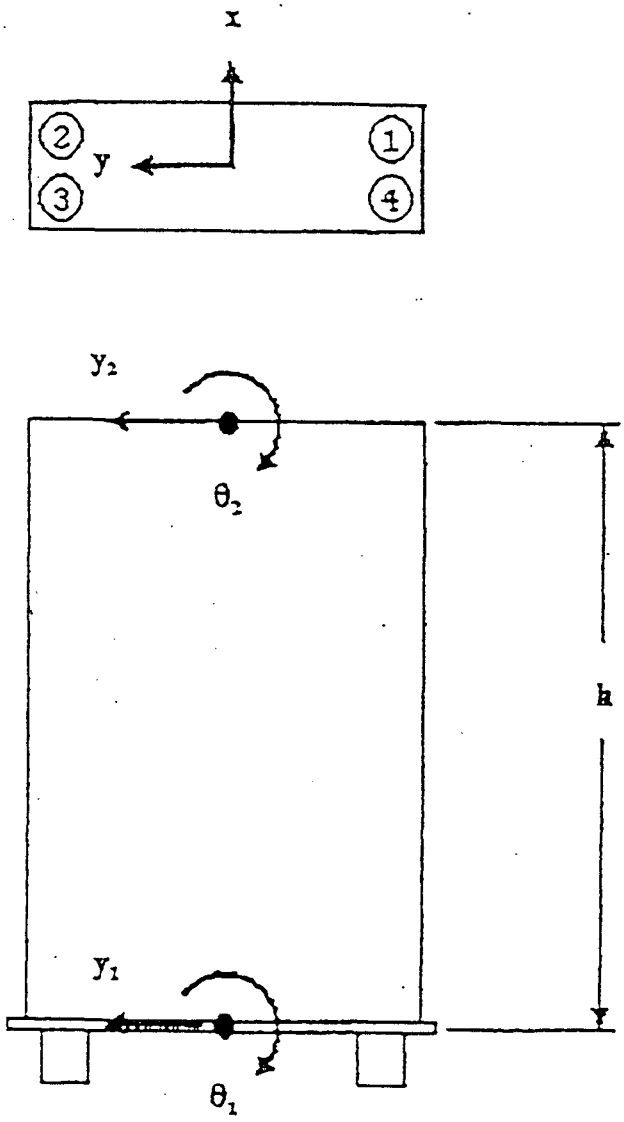

FIGURE 2

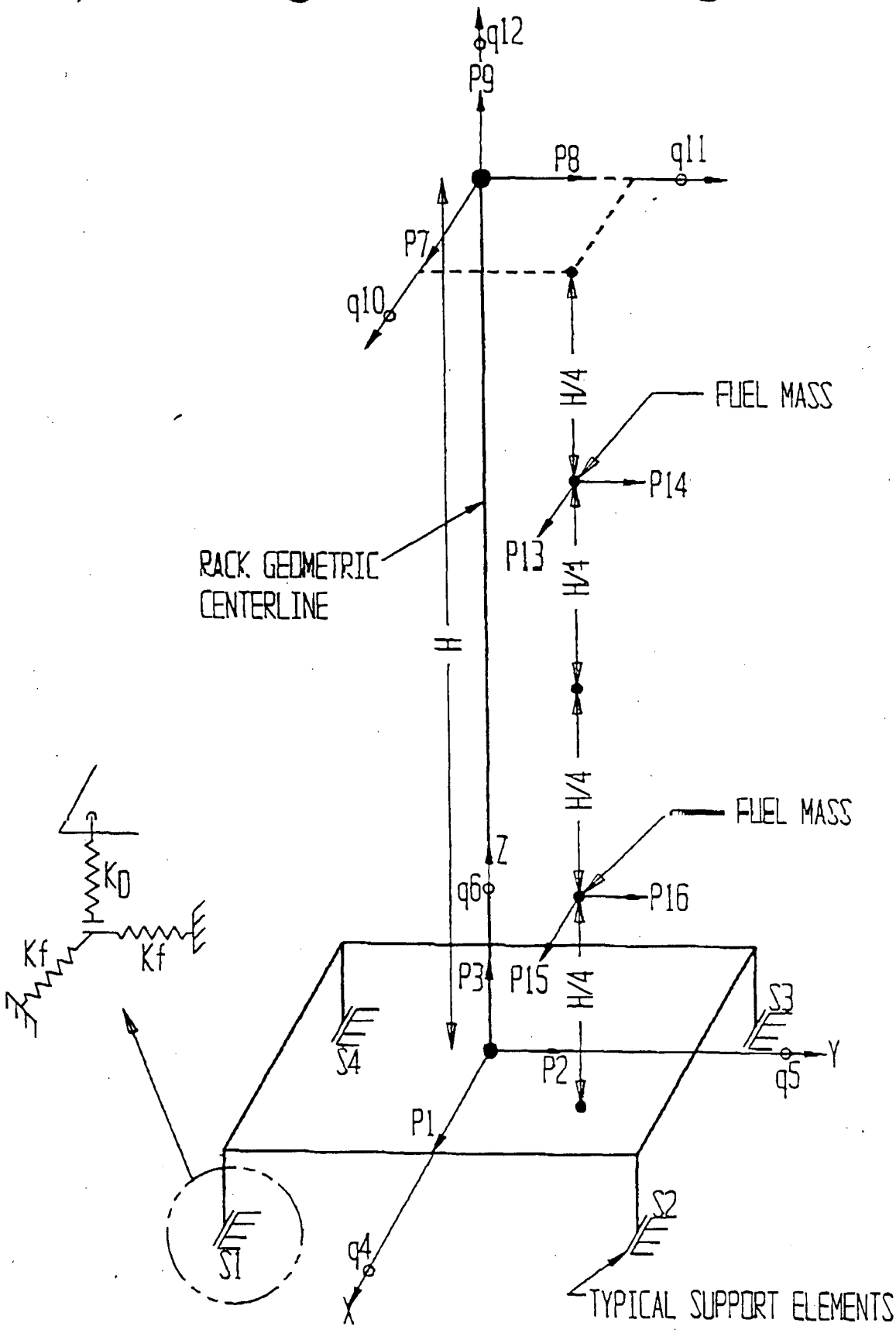
q12
P9
P8
q11
P7
q10
H/4
FUEL MASS
P14
P13
RACK GEOMETRIC
CENTERLINE
H
H/4
H/4
FUEL MASS
Z
q6
P16
KD
Kf
Kf
P3
P15
H/4
S3
S4
Y
P2
q5
P1
S2
S1
q4
X
TYPICAL SUPPORT ELEMENTS

Figure 3

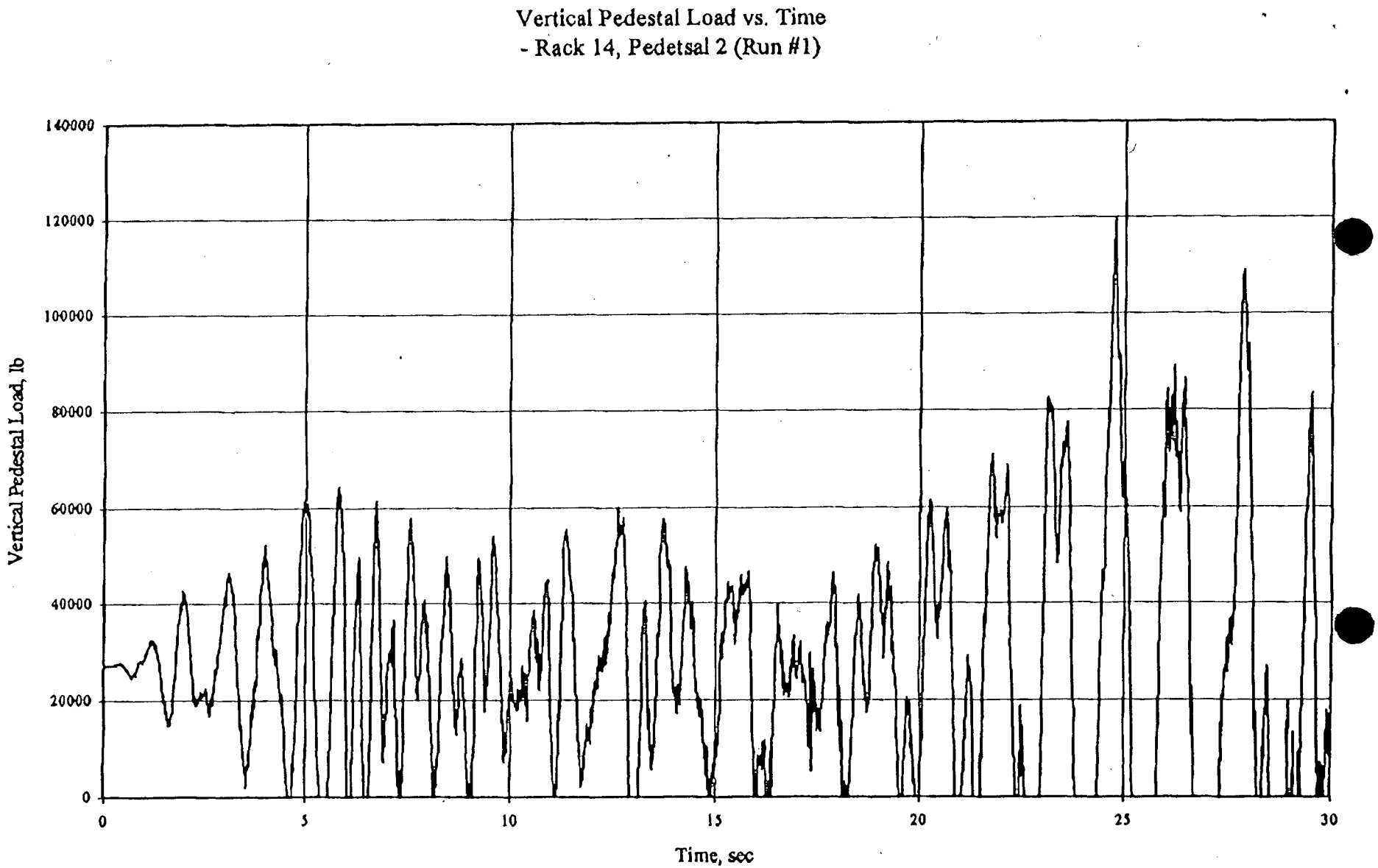
Vertical Pedestal Load vs. Time
- Rack 14, Pedetsal 2 (Run #1)
Vertical Pedestal Load, lb
140000
120000
100000
80000
60000
40000
20000
0
0
5
10
15
20
25
30
Time, sec

Figure 4