QUADREX

EVALUATION OF THE EFFECTS OF POSTULATED ROCKING OF RACKS ON SPENT FUEL POOL STRUCTURES OF DRESDEN NUCLEAR STATION UNITS 2 & 3

Prepared for

COMMONWEALTH EDISON COMPANY
Chicago, Illinois

by

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1.0 INTRODUCTION

It has been proposed that each spent fuel pool for Dresden Units 2 & 3 would contain 33 high-density spent fuel racks. These racks would be of free-standing design with no anchor to the floor or lateral support from the walls. During a postulated seismic event, the racks can potentially slide and/or rock. The purpose of the present evaluation is to determine effects of such rocking of the racks on the pool floor and the walls.

The original design basis of the plant had an SSE of 0.2g at the ground level (Reference 1). It used a Housner type response spectrum. Further evaluation of the plant was performed using the time-history record from the El-Centro earthquake of May 18, 1940, scaled to 0.2g. Subsequently, a site-specific ground spectrum for the Dresden site was developed by USNRC. This had a zero-period SSE acceleration of 0.13g at the ground level. The response spectrum of the pool floor motion computed on the basis of this site-specific ground spectrum was used in an earlier analysis for evaluating the adequacy of the pool floor (Reference 2). Results of this analysis showed that, during a postulated SSE, the racks would rock with a maximum uplift of about 0.1 inch. The pool floor was evaluated to be structurally adequate to withstand the additional load resulting from the impact of the racks during such rocking. However, during discussion with USNRC staff, it was decided that additional analysis would be necessary from the following consideration:

> Reference 2 analysis used an approximate energy-balance method based on linear response-spectrum analysis of the rack. Since rocking/sliding phenomena is actually nonlinear, the uncertainty associated with this analysis cannot be quantified. Hence, a nonlinear analysis of the phenomenon would be preferable.

From the above consideration, it would have been adequate to perform a nonlinear response analysis of the rack, using the pool floor motion time-history calculated from site-specific ground response spectrum. However, USNRC staff reasoned that this may not be adequate, because the site-specific spectra is relatively narrow-banded, and so it may not be conservative enough to account for the possible sensitivity of the nonlinear response to the change in the input time-history and to the change in the friction coefficient between the rack and the pool floor. Considering the large computational cost associated with multiple time-history analyses using different coefficients of friction, it was decided that the adequacy of the pool structures would be evaluated using a single nonlinear time-history analysis, and the effects of the above-mentioned uncertainties would be considered by way of selecting a wide-banded ground response spectrum which would also have significantly higher amplification factors. The wide-banded ground response spectrum of USNRC Regulatory Guide 1.60 scaled to 0.2g (i.e., 54 percent higher than site-specific value of 0.13g) was judged to provide the basis for such an analysis and was used in the present evaluation.

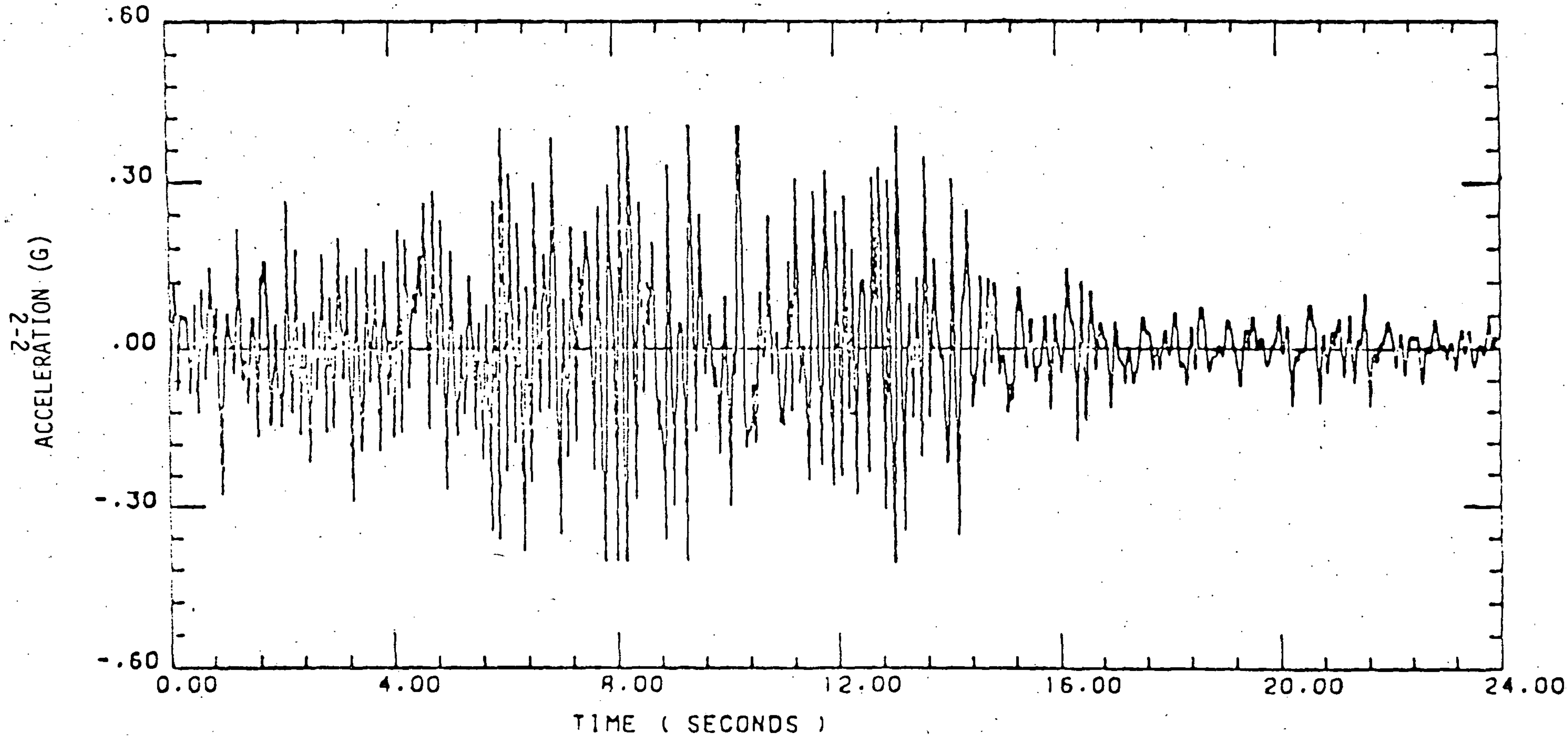
In Section 2.0, the procedural steps and assumptions used in the development of the pool floor motion time-history is presented. The mathematical model used in the analysis is described in Section 3.0. Section 4.0 presents analysis results and the evaluation of the rack, pool floor and the walls. A discussion of the evaluation results and the conclusions are presented in Section 5.0.

2.0 DEVELOPMENT OF POOL FLOOR MOTION TIME-HISTORY

The development of the pool floor motion time-history used in this evaluation consisted of the following steps:

a. The building structural model developed by Lawrence Livermore Laboratory was modified for as-built condition.
b. USNRC Regulatory Guide 1.60 response spectrum scaled to 0.2g was selected as the basis for the input motion at the ground level. A synthetic time-history matching this response spectrum was developed.
c. A time-history response analysis was performed using the building model (Item 'a'above) and the synthetic time-history (Item 'b' above). A seven percent building structural damping was used per USNRC Regulatory Guide 1.61. For use with such a conservative ground time-history, which is likely to cause high stress in the building, the use of seven percent damping value is conservative, especially when it is compared to the "best-estimate" or average damping value of 7 to 10 percent recommended in References 1 and 3.
d. Response spectrum at the spent fuel pool floor level was developed using a 2 percent equipment damping. Again, the use of 2 percent damping is conservative since Regulatory Guide 1.61 recommends 4 percent damping, and Reference 3 recommends 5 to 7 percent damping.
e. The floor response spectrum developed in Item 'd' above was smoothened and peak-broadened by ± 15 percent to account for building modeling and response uncertainties.
f. A synthetic time-history was developed (Figure 2-1) matching the peak-broadened floor response spectrum (Item 'e' above). The comparison of this synthetic time-history with the actually computed motion is shown in Figure 2-2 in terms of their response spectra. The peak acceleration for the synthetic time-history is about 20 percent higher than the actually computed peak acceleration. This provided additional conservatism in the input motion.

DRESDEN 2 REACTOR-TURBINE BUILDING
NRC REG. GUIDE 1.60 RESPONSE T-H (N/S)



FLOOR TIME HISTORY, ELEV. 570 FT.
ITERATION = 5+5+5, DAMPING= 0.02

FIGURE 2-1

DRESDEN 2 REACTOR-TURBINE BUILDING
NRC REG. GUIDE 1.60 RESPONSE T-H (N/S)

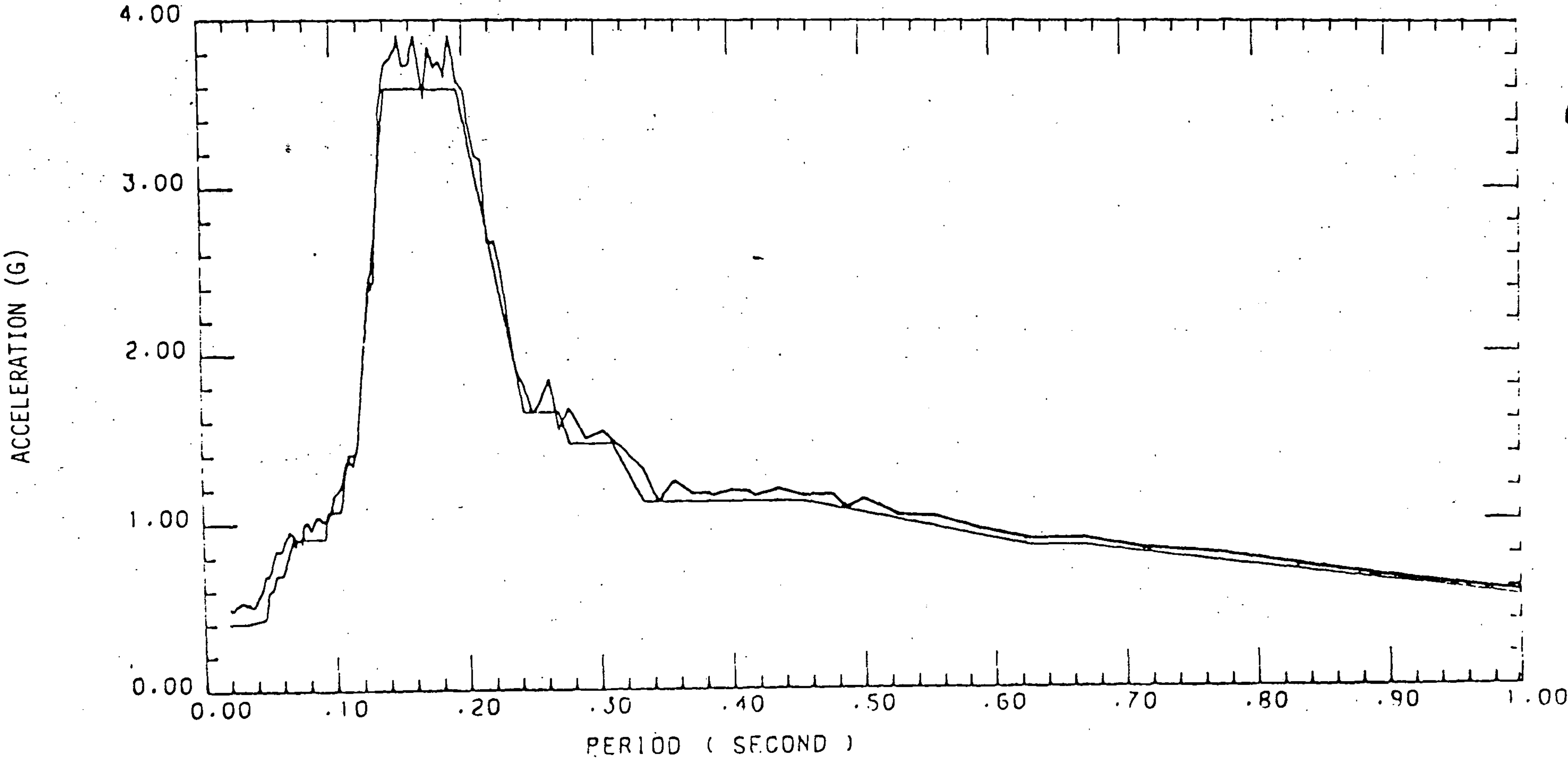


FIGURE 2-2 FLOOR RESPONSE SPECTRUM, ELEV. 570 FT.
ITERATION =5+5+5, DAMPING = 0.02

3.0 MATHEMATICAL MODEL

A schematic of the finite element mathematical model used in the evaluation is shown in Figure 3-1. It consists of a lumped mass stick model of a loaded rack and a simplified mass-spring-dashpot system representing the overall behavior of the floor as it interacts with the rocking of the rack. A nonlinear rocking/sliding response analysis of this rack-floor system was performed using the computer program ANSYS (Reference 4).

The significant features of the mathematical model used in the nonlinear time-history analysis are briefly outlined in the following paragraphs:

3.1 The Rack Structure

The rack structure is idealized as a planar frame consisting of a beam cantilevering from the base plate. The leg beams connect the base plate to the floor. Elements 1, 2, and 3 represent the rack body consisting of an array of tubes which are welded to each other and to the base plate. Elements 6 and 7 represent the rack legs. Elements 4 and 5 represent the diaphragm behavior of the base plate in the horizontal direction, which is essentially rigid. Elements 8 and 9 represent the stiffness characteristics of the rack in the vertical direction. The stiffness properties of Elements 1 through 7 were obtained by matching the horizontal cantilever frequency of this simplified model with that of the detailed finite-element static model of the actual rack. The stiffness properties of Elements 8 and 9 were computed similarly by matching the vertical stiffness of this simplified model with that of the detailed model.

3.2 The Fuel Assemblies

A nominal gap of about 0.28-inch exists between a fuel assembly and the storage tube which forms the rack body. The fuel assemblies can potentially pivot inside the tubes and impact on the tube walls during a seismic event. To account for this nonlinear behavior, the fuel assemblies were represented by beam elements 10, 11, and 12, and gap-spring elements 13 through 16. The stick representing the fuel assemblies (i.e., elements 10, 11 and 12) was assumed to be pinned at the bottom node (i.e., Node 8). The stiffness properties of the beam elements (i.e., elements 10, 11, and 12) were calculated assuming conservatively that all the fuel assemblies are channeled, thus providing maximum stiffness to maximize impact effects. The stiffness of the gap-spring was based on local elasto-plastic deformation characteristics of the tube walls impacted by fuel assembly.

A big source of conservatism has been introduced in the above modeling of the fuel assemblies: it assumed that all of the fuel assemblies inside a rack will "rattle" in-phase impacting on the rack structure simultaneously. Impacts are more likely to be at random, and the impact of the adjacent fuel assemblies on the common cell wall may, to some extent, cancel each other, thereby significantly reducing the response.

3.3 The Water Mass

The horizontal hydrodynamic effects of the water mass surrounding the racks were incorporated by considering the external water mass along with the body mass of the rack structure. The virtual mass was computed in accordance with Reference 5. The water mass inside the annular space between the fuel channels and the storage tube cells was represented as coupled mass between the stick representing the rack structure (i.e., elements 1, 2, and 3) and the stick representing the fuel assembly (i.e., elements 10, 11, and 12). The water trapped inside the fuel assemblies was considered along with the body mass of the fuel assemblies.

The hydrodynamic mass effects associated with the motion in the vertical direction were not considered because the rack is open in the vertical direction and, hence, the effect is likely to be insignificant. Also, the inclusion of hydrodynamic mass effect in the vertical direction would have increased the effective mass in the vertical direction and would, thereby, have reduced the uplift.

3.4 Rack-Floor Interface

Elements 17 and 18 represent the sliding/rocking interface between the rack and the pool floor. The interface consists of two plane stainless steel surfaces which may maintain or break physical contact in the vertical direction, and may also slide horizontally relative to each other.

In the vertical direction, the stiffness properties of these two elements are such that no tensile force can exist in the interface. When the rack legs impact on the floor, the compressive stiffness of the element is represented by the local load-deformation characteristics of the pool floor under the rack leg.

In the horizontal direction, the stiffness properties of these two elements are based on a median coefficient-of-friction value of 0.5 (Reference 6). Since this evaluation considers the response of a large number of racks (33 racks) involving even a larger number of leg-floor interfaces (6 times 33 equals 198), the use of median coefficient of friction can be considered as the "best-estimate" value.

3.5 Damping for Rack Structure & Fuel Assemblies

Raleigh damping (proportional to mass and stiffness) was used for the rack structure and fuel assemblies. The proportionality constants were determined using an equivalent modal damping value not to exceed 2 percent within the frequency range of 1.5 cps (arbitrarily selected low-end frequency) to 11.5 cps (fixed-base rack fundamental frequency).

3.6 Pool Floor Model for Determining Impact Load

In order to be able to determine directly the vertical response of pool floor resulting from the rocking of the racks, it was modeled as single degree-of-freedom mass-spring-dashpot system. The equivalent floor mass corresponding to a single rack is represented by the mass at Node 14. The stiffness and damping properties are represented by the spring-dashpot element No. 30. The stiffness of this spring was determined using the overall load-deformation characteristics of the pool floor based on cracked concrete section properties. The dashpot damping value was selected such that the combined effect of the Raleigh damping (corresponding to floor vertical frequency of 23 cps) and the viscous dashpot damping do not exceed a conservative equivalent modal damping value of 5 percent.

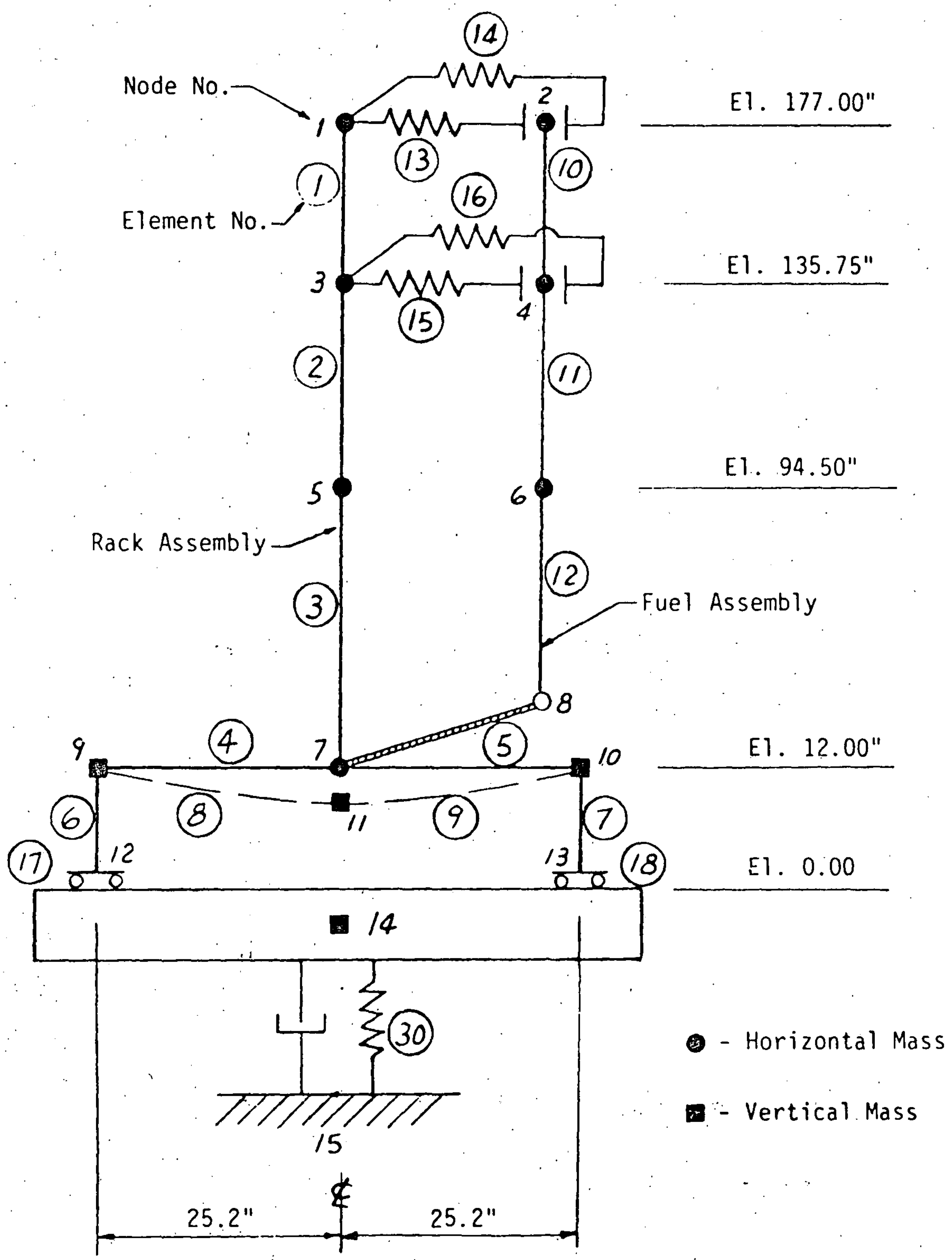


FIGURE 3-1 Non-linear Analysis Model of Dresden Rack

4.0 NONLINEAR RESPONSE ANALYSIS RESULTS AND EVALUATION

4.1 Response Analysis Results

A nonlinear time-history analysis of the rack-floor system was performed using the mathematical model and the input floor motion time-history described in Sections 3 and 2, respectively. An integration time-step of 0.0025 sec. was used. Since no strong motion occurs after the first 14 seconds of the 24-second time-history, the response was computed for the first 16 seconds. Results show that after the first 15 seconds, there are no significant peaks of the responses. Other pertinent results are briefly described in the following paragraphs:

a. The maximum sliding distance was about 0.03-inch, occurring at about t=7.4 and t=10.6 seconds. Maximum sliding velocity was about 1.55 in/sec and occurs at about t=6.8 seconds. A portion of the sliding time-history showing the maximum sliding is presented in Figure 4-1.

b. The maximum uplift was about 0.92-inch occurring at about t=7.05 seconds. A portion of the uplift time-history showing the maximum uplift is presented in Figure 4-2. The maximum corresponding rack impact on the pool floor interface (including dead load) is about 3450 lbs. per fuel assembly occurring at about t=7.3 seconds. A portion of the rack leg force time-history showing this peak leg load/fuel assembly (3450 lb) is presented in Figure 4-3.

c. The maximum rack impact load, occurring at about t=14.8 seconds, is about 3458 lb per fuel assembly. Maximum rack leg reactions, listed in Table 4-1, are based on this value.

d. The maximum floor reaction including dead load, occurring at about t=7.3 seconds, is about 3350 lbs per fuel assembly. A portion of the pool floor reaction time history showing this is presented in Figure 4-4. The design basis pool reaction load was based on this peak value.

4.2 EVALUATION

Table 4-1 shows a comparison of the rack leg forces resulting from the rocking effect with those calculated earlier assuming that the racks do not slide or rock. The stresses in different components of the racks are shown in Table 4-2. Comparison of these stresses with the allowable show that the rack components will not be overstressed.

Pool floor and walls were evaluated for the total load, including the effects from the rocking of the racks. The total equivalent uniformly distributed load, including the impact load obtained from nonlinear response analysis of the rack-floor system, was computed to be 11.07 kips per sq. ft. (See Table 4-3), assuming that all of the 33 racks in each floor will impact on the floor at the same instant. This assumption of simultaneous impact of all racks on the pool floor is very conservative; a more realistic assumption would be to use SRSS combination of impact loads from individual racks. With such realistic assumption, the total equivalent uniform load on the pool floor was computed to be 7.02 kips per square foot. The capacity of the pool floor slab based on diagonal shear is 13.04 k/ft^2; flexure and shear friction capacities are much higher. Thus, the floor slab capacity is adequate to withstand the additional loads resulting from the proposed storage of high-density racks.

The south and the east walls were evaluated for the loads equal to the shear capacity of the pool slab. These two walls are more critical than the other two walls. Evaluation results, presented in Table 4-4, show that the shear capacity of the walls are much higher than the predicted loads. Flexural capacity is still higher. Hence, the pool walls are structurally adequate to support the total loads resulting from the proposed storage of high-density racks.

TABLE 4-1

Maximum Rack Leg Forces Due to Rack Impact, Vertical Seismic (SSE) and Buoyant Weight of Rack

Consideration	Maximum Force (kips)	
	Corner Leg	Middle Leg
Considering rack impact by nonlinear analysis	131.3	164.1
Original Fixed-base analysis (1)	179.8	208.9

NOTE:

1. For the purpose of comparison only.

TABLE 4-2

Stresses in Rack Components Including
The Rocking Effect of Rack

Rack Component	Load Combination	Critical Stress Type	Allowable(1) Stress (ksi)	Computed(2) Stress (ksi)
Tube Wall	D+B+E'	Membrane	33.5	16.37
Fuel Support Plate	D+B+E'	Membrane	33.5	13.04
Filler Plate	D+B+E'	Membrane	33.5	13.17
Base Grid	D+B+E'	Membrane	33.5	2.41
Rack Leg	D+B+E'	Membrane	33.5	12.14
Interface	D+B+E'	Bearing	4.76	1.24

NOTES:

1. Using a dynamic increase factor of 1.2
2. Obtained by multiplying the dead load stresses (from original finite element analysis) with a scale factor, equal to the ratio of rack leg reaction considering the rack impact on pool slab over the dead load reaction of rack leg.

TABLE 4-3

Evaluation of Pool Floor Slab

(a)	Capacity of pool slab based on shear failure due to diagonal tension	13.04 k/sf
(b)	Total uniform load with impact forces to 33 racks added by absolute sum method	11.07 k/sf
(c)	Total uniform load with impact force due to 33 racks added by SRSS method	7.02 k/sf

TABLE 4-4

Evaluation of Critical Pool Walls[1]

Wall	Load Combination	Vertical Shear (kips)	
		Allowable	Computed
North and South Wall	D+L+H+E'+Impact	15724[2]	<5404[4]
East Wall	D+L+H+E'+Impact	3280[3]	<3104[4]

NOTES:

1. Bending, which is less critical than shear, was not tabulated.

2. Considering the effect of vertical reinforcement in resisting diagonal tension resulting from shear.

3. Considering shear capacity due to concrete only. Value will be much higher if effect of vertical reinforcement is included.

4. These values are based on a maximum uniform load of 13.04 k/sf, which the pool slab can resist without shear failure.

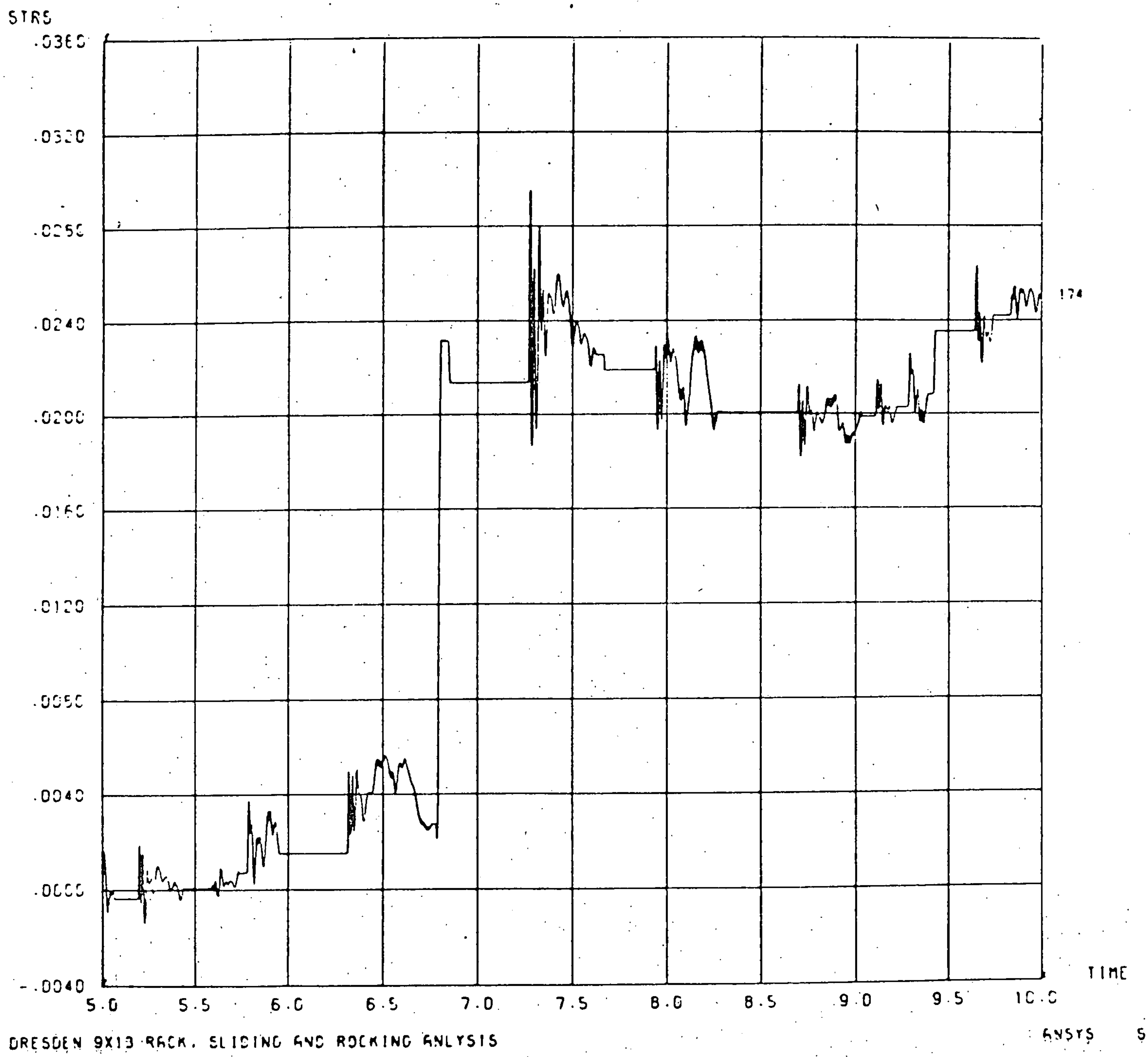


FIGURE 4-1a Sliding (In Inches) of Dresden Rack

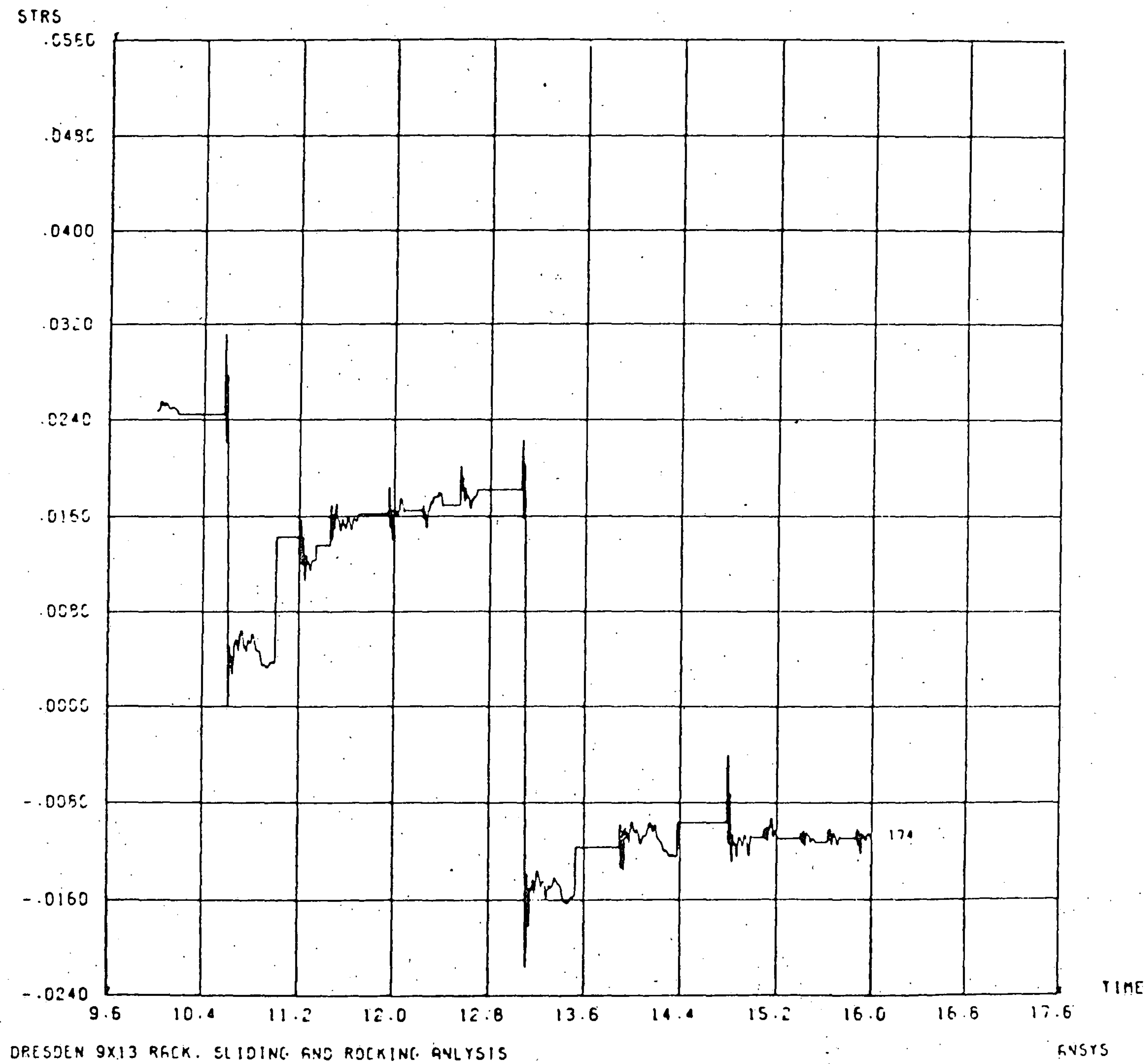


FIGURE 4-1b Sliding (In Inches) of Dresden Rack

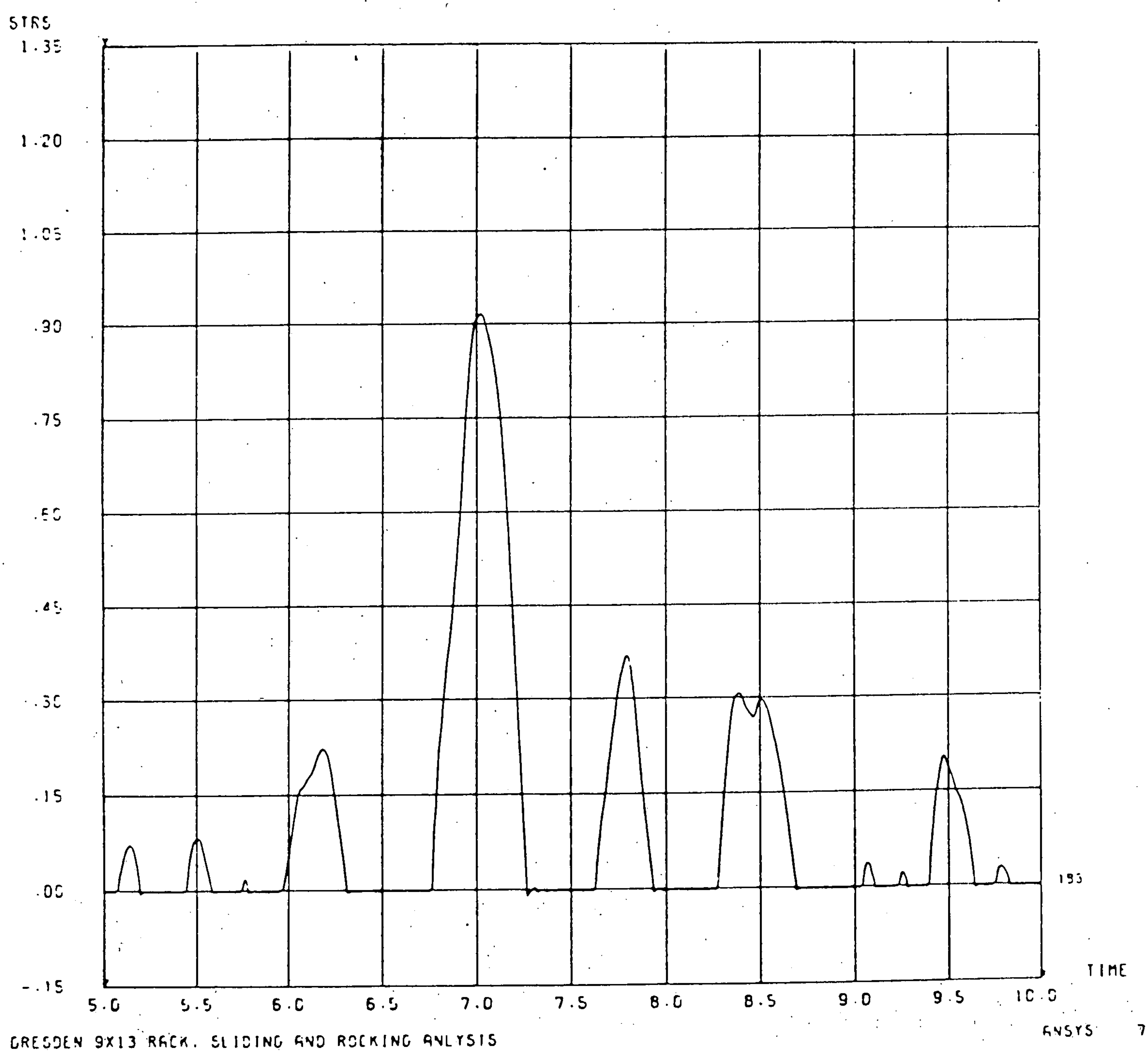


FIGURE 4-2. Uplift (In Inches) of Dresden Rack

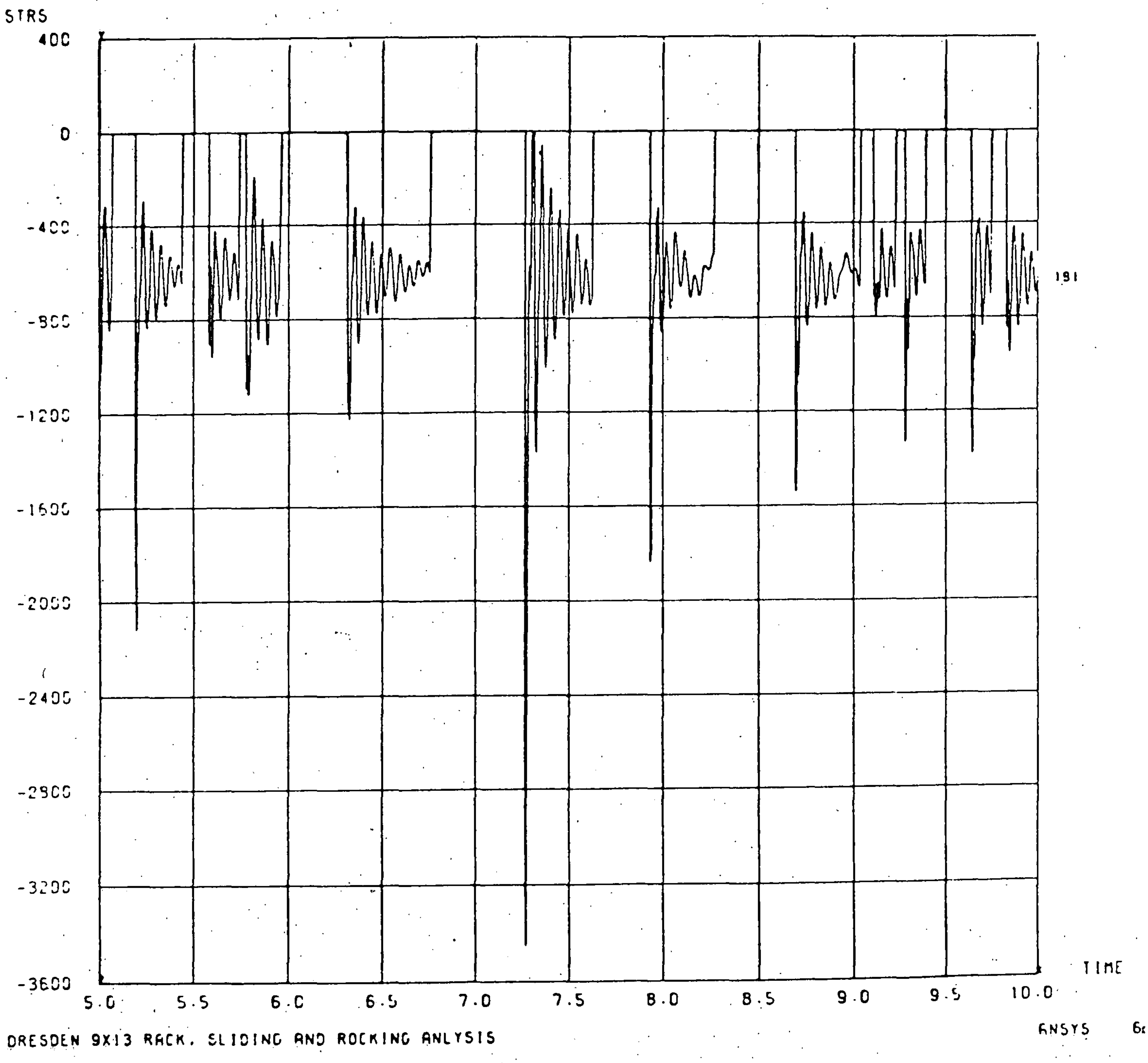


FIGURE 4-3 Rack Leg Impact Force (In Pounds) of Dresden Rack

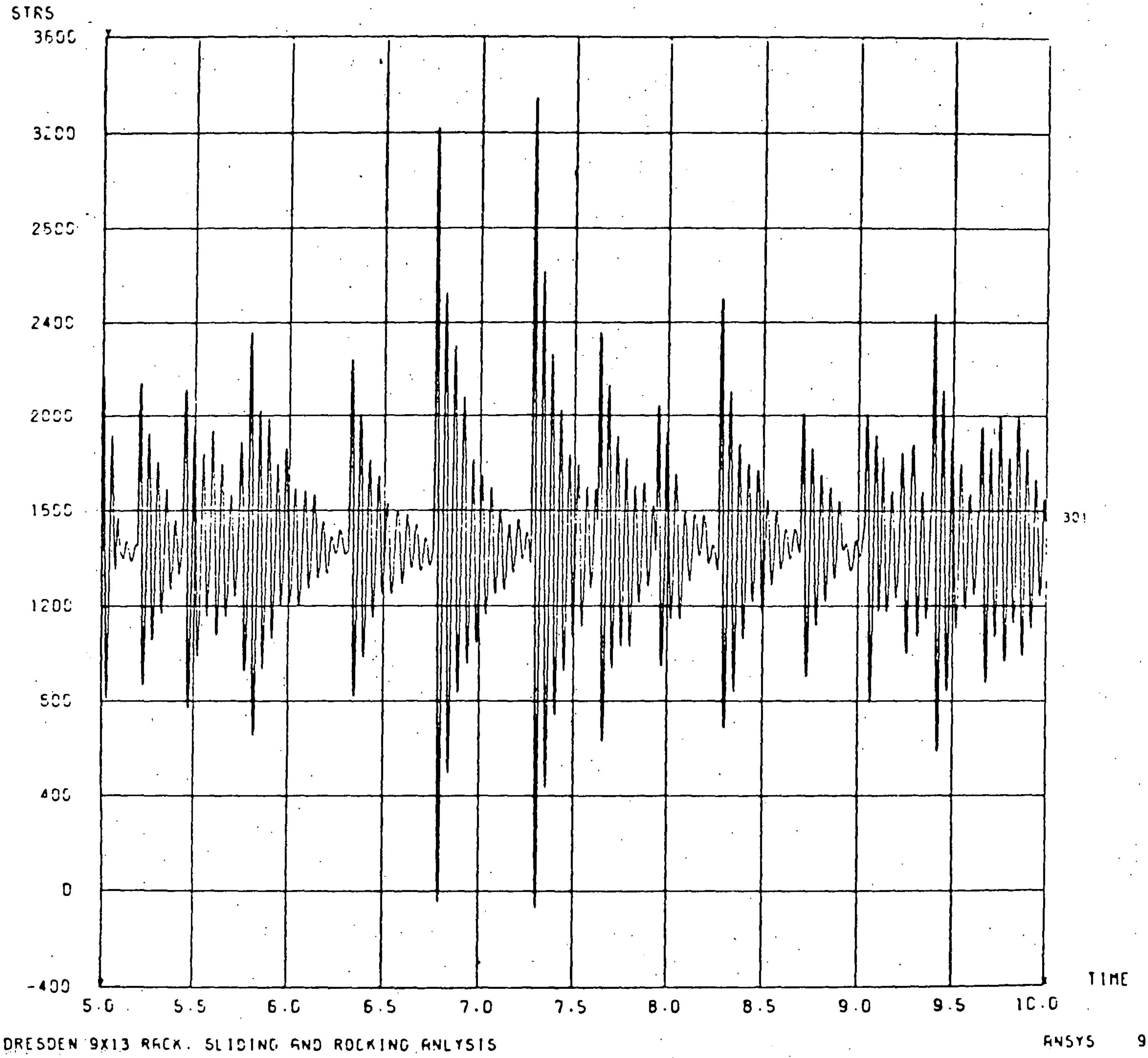


FIGURE 4-4 Pool Slab Force (In Pounds)
Due to Rack Impact

5.0 CONCLUSION

Evaluation of the effects of rocking of the racks showed that the racks, the pool floor and the pool walls are structurally adequate to withstand the additional loads that might result from racks impacting on the pool floor.

The evaluation was performed very conservatively to account for the possible sensitivity of the nonlinear response to the change in the input time-history. The sources of conservatisms are summarized and discussed below:

1. The realistic ground response spectrum applicable for Dresden site is the SEP site response spectrum. The design-basis response spectrum used in the present evaluation is the USNRC Reg. Guide 1.60 response spectrum The time-history compatible with this wide-band response spectrum is much more conservative frequency-content wise when compared to the site specific spectrum. Also, the response amplification factors used in the present analysis were increased by 54 percent by way of using 0.2g peak ground acceleration instead of the site specific value of 0.13g.

2. All the fuel assemblies inside a rack were assumed to move in-phase during a seismic event, impacting on the rack storage cells simultaneously. In actuality, it is more likely that these assemblies would move at random, in which case the reaction load on the rack structure resulting from the motion of one group of assemblies may be reduced or neutralized by the reaction loads from another group of assemblies inside the same rack but moving in the opposite direction. The factor of conservatism introduced due to this assumption is very difficult to estimate; however, if the response was linear, and if the motion of the assemblies could be assumed random, the factor of conservatism could be as high as 9.9.

3. Thirty-three high-density racks are proposed for each pool. Impact loads resulting from the rocking of these 33 racks are likely to be somewhat at random because of the following:

a) Differences in the inertia and stiffness characteristics of the 9x11 and 9x13 racks. Even though the short sides of these two sizes of racks are oriented in the same direction, their inertia and stiffness characteristics are different because of hydrodynamic mass effect and width-to-length ratio.

b) The probability that the friction coefficient between the pool floor and different racks would vary is extremely high. This would affect the time-phasing of the response.

c) During a postulated seismic event, each rack impacts on the pool floor several times. The resulting time history of the reaction load on the pool floor shows a large number of load cycles (in the order of hundreds). Of these load cycles, only a few (less than 5) have peaks which are more than 80 percent of the maximum floor reaction load. Hence, it is highly improbable that these infrequent maximum floor loads from different racks would occur simultaneously.

Items 'a', 'b', and 'c' above provide justifications for using SRSS combination of the peak floor loads resulting from the impact of each of the 33 racks. Hence, the equivalent distributed floor load of 7.02 k/ft^2, computed on the basis of SRSS combination, is more realistic than the value of 11.07 k/ft^2, which is based on the assumption that the peak impact forces from the 33 racks would occur simultaneously. Quantitatively, the latter assumption of simultaneous impact increases the floor impact load by approximately 5.7 times that obtained by SRSS method.

Considering the sources of conservatism discussed above, it is concluded that the computed floor load of 11.07 k/ft^2 has sufficient conservatism to account for the sensitivity of the response due to the change in the input motion time-history, since it is based on a very conservative (both frequency-content wise as well as magnitude wise) input response spectrum. Also, it assumes in-phase motion of all fuel assemblies inside a rack and is based on the very conservative assumption that peak impact loads from the 33 racks would occur simultaneously. Thus, this load approximates the upper bound load. Since even this load is significantly less than the floor shear capacity of 13.04 k/ft^2, it is concluded that the floor slab is structurally adequate to withstand the loads resulting from the storage of high-density spent fuel racks.

6.0 REFERENCES

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