The Light company
Houston Lighting & Power

P.O. Box 1700 Houston, Texas 77001 (713) 228-9211

August 30, 1988
ST-HL-AE-2764
File No.: G20.02.01, M20.1
10CFR50

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

South Texas Project Electric Generating Station
Units 1 and 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions Regarding the
Use of High Density Spent Fuel Racks

Reference (1): HL&P Letter to USNRC, G. E. Vaughn to Document Control Desk, ST-HL-AE-2417 dated March 8, 1988.

(2): HL&P Letter to USNRC, ST-HL-AE-2738; Summary of Meeting on July 11 & 12, 1988 to discuss High Density Spent Fuel Racks.

(3): HL&P Letter to USNRC, ST-HL-AE-2750; Summary of NRC Technical Audit of U. S. Tool & Die, Inc. on July 20 to 21, 1988.

On July 11 & 12, 1988, personnel from Houston Lighting & Power Company (HL&P), Bechtel Energy Corporation (BEC) and U. S. Tool & Die, Inc. (UST&D) met with members of the NRC Staff to discuss the proposed license amendment regarding expansion of the spent fuel pool storage capacity using high density spent fuel racks that was submitted to the NRC via letter dated March 8, 1988 (reference 1). Minutes of the meeting are documented in reference 2.

As a result of the meeting, HL&P committed to provide additional information for some of the questions discussed during the meeting. Attached (Attachment #1) are revised responses to questions 4, 5, 11, 12, 13 and 14. The forces induced in the spent fuel pool walls due to the reduced rack-to-wall distance have been addressed in Question Response 11 which was a confirmatory item in reference 3. Change bars have been used to indicate where the response has been revised. This information should close these items.

B021
1/1

L4/NRC/bx

Also attached (Attachments # 2 & 3) is information from UST&D regarding the additional forces on the rack walls due to the reduced rack-to-wall distance at the north end of the spent fuel pool (Attachment #2), and information on the 3-rack analysis (Attachment #3). Attachments 4, 5 and 6 provide annotated revisions to the High Density Spent Fuel Racks Safety Analysis Report, FSAR and Evaluation of No Significant Hazards Considerations as a result of the information in Attachment #1.

If you should have any questions regarding this matter, please contact Mr. A. W. Harrison at (512) 972-7298.

M A McBurnett
M. A. McBurnett
Manager,
Operations Support Licensing

MEP/hg

Attachments:
(1) Revised Responses to NRC Questions 4, 5, 11, 12, 13, and 14 on Spent Fuel Racks.
(2) Calculation of Forces on Rack Walls due to Reduced Rack-To-Wall Distance at North End of SFP
(3) Information on the 3-Rack Analysis.
(4) Annotated Revisions to the High Density Spent Fuel Racks Safety Analysis Report
(5) Annotated Revisions to Section 3.7 of the Final Safety Analysis Report.
(6) Annotated Revision to the Evaluation of No Significant Hazards Considerations contained in Reference 1.
(7) Figures 1 through 9 for question 5 (Attachment #1)

L4/NRC/bx

cc:

Regional Administrator, Region IV
Nuclear Regulatory Commission
611 Ryan Plaza Drive, Suite 1000
Arlington, TX 76011

*George Dick, Project Manager
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Jack E. Bess
Resident Inspector/Operations
c/o U.S. Nuclear Regulatory
Commission
P. O. Box 910
Bay City, TX 77414

Don L. Garrison
Resident Inspector/Construction
c/o U.S. Nuclear Regulatory
Commission
P. O. Box 910
Bay City, TX 77414

J. R. Newman, Esquire
Newman & Holtzinger, P.C.
1615 L Street, N.W.
Washington, DC 20036

R. L. Range/R. P. Verret
Central Power & Light Company
P. O. Box 2121
Corpus Christi, TX 78403

R. John Miner (2 copies)
Chief Operating Officer
City of Austin Electric Utility
721 Barton Springs Road
Austin, TX 78704

R. J. Costello/M. T. Hardt
City Public Service Board
P. O. Box 1771
San Antonio, TX 78296

Rufus S. Scott
Associate General Counsel
Houston Lighting & Power Company
P. O. Box 1700
Houston, TX 77001

INPO
Records Center
1100 Circle 75 Parkway
Atlanta, GA 30339-3064

Dr. Joseph M. Hendrie
50 Bellport Lane
Bellport, NY 11713

(*) Attachment 1 through 7. All others Attachment 1 only.

Revised 06/15/88

L4/NRC/bx

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #4:

Provide information on the effects of High Density Racks (HDR) on soil bearing capacity and liquefaction potential.

RESPONSE #4:

During the July 11 & 12, 1988 meeting with the NRC Bechtel stated that the Fuel Handling Building (FHB) soil bearing pressure under the spent fuel pool (SFP) is insignificantly increased by 1.8 kips per square foot (ksf) due to the additional weight of the high density spent fuel racks. The revised total soil bearing pressure of 13 ksf (DL) and 22 ksf (DL+SSE) are well within the respective allowable values of 21 ksf and 32 ksf. In addition, it was stated that the liquefaction potential is not affected by the slight increase in soil bearing pressure and that liquefaction is effectively controlled by assuring the denseness of the compacted backfill material rather than by limiting soil bearing pressures which are already within the allowable values.

In response to the NRC request for a more quantitative treatment, the reply per Items 1 and 2 is presented. Item 1 describes the original evaluation of liquefaction for the STP, and points out that upon reviewing the parameters considered in that evaluation, it is evident that the slight increase in soil pressure will not reduce the established factors of safety against liquefaction. Item 2 describes the specific calculation of factors of safety against liquefaction performed to incorporate the changes in soil pressure resulting from the additional weight of the high density racks.

Item 1

The liquefaction potential of the cohesionless soil layers at the plant site was evaluated during the geotechnical investigation for the project and is described in Section 2.5.4.8 of the South Texas Project FSAR. The evaluation was performed using the following two soil profiles as representative of the subsurface conditions:

Case 1. The in-situ soil profile in the plant area.

Case 2. The plant area soil profile with the top 40 feet of existing soil replaced by structural sand backfill.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #4 (cont'd):

The Case 1 soil profile is representative of the area immediately outside the area of construction. The Case 2 soil profile with the structural sand backfill approximates the soil and loading conditions in the area immediately surrounding the plant structures. The evaluation performed using the Case 2 profile resulted in slightly higher factors of safety due to the higher vertical effective stress produced by the backfill. The results of the liquefaction evaluation are shown on Table 2.5.4-35 in the South Texas Project FSAR.

The analyses did not consider the higher vertical effective stress resulting from the structural loading of the plant buildings as an additional factor contributing to the safety against liquefaction of the plant area soils. Therefore, since no credit was taken for the additional benefit of the buildings' soil bearing pressures to mitigate liquefaction potential, moderate changes in bearing pressures due to variations in building loads will not affect the calculated factors of safety against liquefaction.

The FHB, approximately 88 feet by 190 feet, is a reinforced concrete structure with foundations at different levels. Within the FHB, the nominal foundation level in the fuel pool area is at approximately 64 feet below grade and rests on the natural cohesionless sands of soil profile layer E, as shown on Figure 2.5.4-54 of the South Texas Project FSAR. The addition of high density racks in the spent fuel pool increases the bearing pressure for the portion of the mat foundation underneath the pool by approximately 1.8 ksf. The vertical effective stress applied to soil layer E from the total foundation load from the spent fuel portion of the FHB is significantly higher than the effective vertical stress of approximately 4.5 ksf applied to layer E by the surrounding soils. This higher vertical stress would be expected to produce at least as high a factor of safety against liquefaction beneath the building as in the soil outside the building footprint. Similarly, the 1.8 ksf increase in soil pressure attributed to high density racks may augment, but will not reduce the calculated factor of safety against liquefaction. Therefore it is determined that liquefaction is not a potential occurrence in the plant area during the postulated SSE, and the original determination remains valid and unaffected by the change in soil bearing pressure under consideration.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #4 (cont'd):

Item 2

A specific calculation of factors of safety against liquefaction was performed to account for the changes in soil pressure resulting from the additional weight of the high density fuel storage racks.

First the increase in soil pressure was determined by an upper bound analysis considering bearing only on the portion of the basemat under the spent fuel pool, without relying on load distribution to the rest of the basemat. Next the potential reduction in soil pressure due to the eccentricity of the added weight was investigated. It was determined that the added eccentric weight was insignificant in creating uplift at the far edge of the basemat away from the pool and that the minimum factor of safety against liquefaction was not reduced. Therefore, the additional analysis considers only the effect of higher vertical stress due to building load variation on the factor of safety against liquefaction. The analyses included the Reference (1) recommendations to correct the cyclic stress ratio when high values of vertical stress occur. The analyses showed no change in the minimum factor of safety due to the loading increase of the high density fuel racks. The factors of safety for the various cases considered are summarized in the attached Table Q4-1.

Reference (1) Seed, H. Bolton: "Earthquake-Resistant Design of Earth Dams", Seismic Design of Embankments and Caverns, ASCE, 1983.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #4 (cont'd):

Table Q4-1

Summary of Liquefaction Factors of Safety for In-Situ Soil

CASE	LOCATION	FACTOR OF SAFETY AGAINST LIQUEFACTION
Generic for Site Original Evaluation		
Case 1	at depth of 65ft. below grade	1.4
Case 2	at depth of 65ft. below grade with the top 40ft. of soil replaced by backfill.	1.5
Specific analyses for FHB		
Original loading without High Density Racks	at location of maximum soil pressure:	
	o at 1.5ft. below basemat	1.6
	o at 12.5ft. below basemat	1.4
Increased loading due to High Density Racks	at location of maximum soil pressure:	
	o at 1.5ft. below basemat	1.5
	o at 12.5ft. below basemat	1.4

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #5:

If one (1) synthetic time history is used in analysis of racks, indicate whether the time history had a sufficient energy content at the natural frequency range of the rack.

RESPONSE #5:

HL&P committed to perform a confirmatory calculation, developing the power spectral density (PSD) function as outlined in the proposed draft revision to Standard Review Plan (SRP) 3.7.1, pertaining to free-field ground motion. The objective will be to demonstrate that the single time history has sufficient energy content and is adequate based on acceptance criteria applicable to in-structure motion rather than ground motion as specifically addressed in the proposed SRP revision.

Power Spectral Density (PSD) Function

The NRC introduced in proposed Revision 2 to the Standard Review Plan (SRP) an additional requirement for the justification of artificial time history records used as input motion in dynamic analyses. The justification is based on the assessment of the energy content of the input motion by means of the power spectral density (PSD) function computed for each time history. For the specific case of time histories developed to represent design response spectra corresponding to the RG 1.60 ground motion spectra, a target PSD is defined as a baseline for the assessment of the computed PSDs. Accordingly, the target PSD as provided in the proposed SRP, is directly applicable to the assessment of time histories representing free-field ground motion. Nevertheless, the ground motion specific target PSD (unmodified) may be extended in use to judge the energy content from PSDs computed for time histories synthesized to represent in-structure seismic response spectra. In those cases, when PSDs of in-structure motion are compared against the ground motion target PSD, the computed PSDs may be below the target within the frequency range above 15 cps which is not of particular importance for rigid structures founded on soil.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #5 (cont'd):

In the South Texas Project (STP), for the nonlinear seismic analyses of the high-density spent fuel storage racks, four synthetic time history records representing the seismic horizontal response of the spent fuel pool floor were used: North-South and East-West directions for the Operating Basis Earthquake (OBE) and the Safe Shutdown Earthquake (SSE). In the vertical direction the seismic analysis of the racks was performed by the equivalent static method. Accordingly, no vertical time histories are involved, and in the following review of the STP ground motion and pool-floor time histories, only the time histories for horizontal input motion are addressed. Also, for simplicity, only the SSE motion is discussed since the results are the same for the OBE.

For the STP the RG 1.60 spectra are used as the ground motion seismic design spectra. The corresponding ground motion time histories are developed for the soil-structure dynamic analyses used to obtain in-structure response, and their computed PSDs are thus directly comparable to the target PSD. Figures 1 and 2 show the computed PSDs for the horizontal SSE ground motion plotted using 3-point averaging and 11-point averaging which is often used to rectify oscillatory functions. The computed PSD curves are above the target within the frequency range of importance for the excitation of the structure through soil-structure interaction and for the floor response within the structure. The isolated minima below the target (at f=14 cps and 20 cps) are not significant, and in fact vanish when the smoothened plot of Figure 2 is considered. The smoothened plot based on 11-point averaging permits better appreciation or the PSD features and indicates that the computed PSD does not fall below the target until after the unimportant high frequency range above 25 cps is reached. Therefore, it is concluded that the computed PSDs for the STP ground motion time histories comply with the target PSD as defined in the proposed SRP for ground motion based on RG 1.60.

The PSDs computed for the floor input motions used in the time history analyses of the spent fuel racks are shown in Figures 3 and 4 based on 3-point averaging and Figures 5 and 6 based on 11-point averaging. In general, and particularly in the low frequency range of importance (2 to 4 cps), the computed PSDs exceed the target PSD as evidenced from both sets of curves. At frequencies higher than about 12 cps, the PSD curves based on 3-point averaging are characteristically below the target PSD. However, that is not a significant indication of low energy content since (1) as stated earlier, this condition is not unexpected when PSDs for in-structure motion are compared to the target PSD for ground motion, and

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #5 (cont'd):

(2) the condition does not affect the 2.5 to 3.0 cps frequency range that corresponds to the dominant and governing lateral response of the racks. Moreover, that apparent underrepresentation of energy content becomes less prominent and its initial frequency of occurrence is shifted upwards (from 12 to 16 cps) when the smoothened 11-point average curves are considered. Another observed characteristic of the PSDs is an apparent low energy content in a narrow frequency band centered at 6.3 cps. This behavior is attributed to the "valley" and "abrupt drop" exhibited by the design-basis spectra at the characteristic frequency of 6 cps (Figures 7, 8 and 9), and it is not related to the synthesization of the artificial time histories. The condition is restricted to the narrow frequency band of 5 to 6.5 cps, thus it is not a significant concern because the rack modes in that frequency range have very low participation factors of 0.002 and 0.003 compared to the higher participation factors of 0.77 and 0.75 for the 2.5 to 3.0 cps modes that dominate the rack lateral seismic response. In summary: (1) the "dips" in the PSD curves at 6.3 cps are traceable to the design-basis spectra and are not originated by a deficiency in the synthetic time histories, (2) the amplitudes of the design-basis spectra have been demonstrated to be conservative by comparison to more rigorous solutions, and (3) the time histories with low PSD values in a narrow range at 6.3 cps are used exclusively in the analyses of the racks which do not have dominant modes in that frequency range. Therefore, it is concluded that the time histories used for the rack analyses are adequate notwithstanding the PSD indications at 6.3 cps.

Design-Basis vs. EHS/Single-step FEM Floor Response Spectra

The underrepresentation of the N-S OBE and SSE spectral response within the narrow frequency range of 5.0 to 6.5 cps is a recognized condition as shown in Figures 8 and 9. The E-W and the vertical directions are not affected by this condition.

The total seismic response of the racks due to three-component earthquake is dominated by the E-W direction, which produces substantially higher responses than the N-S contribution. Therefore, an increase in the N-S contribution will be proportionately lower when compared to the total 3-component response, particularly since the square-root of the sum of the squares (SRSS) combination used for the total response tends to diminish the relative importance of the non-dominant components. Moreover, increases in the N-S response are not anticipated because the

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #5 (cont'd):

slightly higher N-S input corresponding to the rigorous (confirmatory) spectra is restricted to the 5.0 to 6.5 cps frequency range which excludes the 2.5-3.0 cps frequencies of the dominant lateral modes of the racks.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #11:

Identify critical items in the pool structure due to the new rack loads and discuss the margins available.

RESPONSE #11:

During the July 11 & 12, 1988 meeting with the NRC, Bechtel stated that the seismic dynamic response of the Fuel Handling Building (FHB) is dominated by the soil-structure interaction corresponding to an essentially rigid-body building supported by the relatively soft soil. Therefore, it is appropriate to assess the added weight effect on the seismic response by simply considering the incremental mass and additional eccentricity with respect to the building as a whole. The additional weight of the fuel storage racks amounts to a small percentage of the building total weight and produces nodal point shifts of about 5% of the building horizontal dimensions. Therefore, these resultant changes were considered insignificant.

The NRC stated that a specific modification of the dynamic analysis model is necessary, and that the flexibility of the pool floor slab must be included. Bechtel stated that these concerns were going to be resolved as follows:

- A revised dynamic analysis (elastic half-space soil-structure interaction) will be performed incorporating the flexibility of the spent fuel pool slab. For the purposes of the analysis, the fuel pool and its supporting slab will be treated as a subsystem for vertical response.

- The analysis will include torsional response of the building related to the mass increment and nodal point eccentricities due to the heavier fuel storage.

Upon establishing the adequacy of the original seismic response spectra, addressing the above concerns as described in detail in the response to Question 13, the existing calculation for the structural verification of the spent fuel pool was reviewed. The calculation is based on the original response spectra and justifies the adequacy of the existing pool structure.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION # 11 RESPONSE (cont'd):

The calculation considers the heavier weight and higher seismic response of the racks, combined with the previously defined thermal loading, gravity loading, hydrodynamic loading and seismic loading applicable to the FHB. The flexural and shear loads in the governing elements of the reinforced concrete structure are demonstrated to be within allowable limits with adequate design margins as summarized in the attached Table Q11-1. The design margins for the transverse shear in the spent fuel pool floor slab are itemized for consolidated and unconsolidated fuel storage. The shear check for consolidated fuel storage utilizes the confirmatory-basis response spectra described in the South Texas Project FSAR, Section 3.7.2.4. Accordingly, FSAR Table 3.7-8 (Attachment 5) is annotated to identify the use of the confirmatory-basis spectra. It is concluded that the existing structure is adequate to accommodate the heavier high-density fuel storage racks.

The loading induced on the walls of the spent fuel pool as a result of the reduced rack-to-wall distance has been addressed. The north wall is checked since the minimum rack-to-wall distance is at that location. Table Q11-1 includes the design margin for this case.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION # 11 RESPONSE (cont'd):

TABLE Q11-1

CRITICAL ELEMENT DESCRIPTION/LOCATION NOTE 2	TYPE OF LOADING	DESIGN MARGIN NOTE 1	REMARKS
Floor Slab of Pool			
Midspan, along E-W direction (element #46)	Flexure M(+) primary loads. M(+) primary + thermal loads.	 2.28 1.54	Primary & thermal are additive
Midspan, along N-S direction (element #44)	Flexure M(+) primary loads. M(+) primary + thermal loads.	 2.37 1.89	Primary & thermal are additive
Over the interior support wall, along E-W direction (element #42)	Flexure M(-) primary loads. M(+) primary + thermal loads.	 1.49 6.76	Thermal M(+) dominates and eliminates primary M(-)
At the support edge	Transverse shear.	For unconsolidated fuel storage:	
		1.03	SSE loading combination, based on 90-day tested concrete strength: $f'_c = 6.1$ ksi.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION # 11 RESPONSE (cont'd):

TABLE Q11-1

CRITICAL ELEMENT DESCRIPTION/LOCATION	TYPE OF LOADING	DESIGN MARGIN	REMARKS
		For consolidated fuel storage: (see Note 3)	
		1.03	SSE Loading combination, based on 90-day tested concrete strength: $f'_c = 6.1$ ksi
East Wall of Pool			
Base of wall at floor slab (element #267)	Vertical flexure M(-) primary loads. M(+) primary + thermal loads.	 3.00 3.54	Thermal M(+) dominates and eliminates primary M(-)
	Transverse shear due to primary loads (governs).	1.71	Thermal loads do not result in significant shear. Lateral loads from racks are not imparted to East or West walls.
North Wall of Pool			
Base of wall at floor slab	Vertical flexure M(-) primary loads. M(+) primary + thermal loads.	 8.40 2.90	Lateral seismic load imparted hydrodynamically by the racks on the wall is included. Thermal M(+) dominates and eliminates primary M(-).

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION # 11 RESPONSE (cont'd):

TABLE Q11-1

CRITICAL ELEMENT DESCRIPTION/LOCATION	TYPE OF LOADING	DESIGN MARGIN	REMARKS
	Transverse shear due to primary loads governs.	5.14	Thermal loads do not result in significant shear. Lateral loads from racks are included.

Notes:

1. The design margin is defined as the ratio of allowable load to calculated design load. For flexure the interaction with axial load is included, and for shear the reduction in concrete shear capacity due to axial tension is included.

2. The concrete element loads are determined from finite element analyses performed originally for the case of unconsolidated fuel storage and subsequently adjusted to account for increase in primary loads due to the heavier consolidated fuel stor .

3. The transverse shear check in the spent fuel pool slab for consolidated fuel storage required the use of the confirmatory (rigorous) response spectra in the generation of seismic loading.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #12

Describe methods of shear load transfer between the pool liner and the concrete. Indicate shear stress/margin on liner and liner anchorage.

RESPONSE #12

During the July 11 & 12, 1988 meeting with the NRC, Bechtel stated that the shear load transfer was considered to be effected by the friction of the liner plate against the concrete.

The NRC requested a more rigorous analysis, disregarding the friction consideration.

A supplementary calculation was performed in response to the request for a more rigorous analysis of the shear load transfer between the pool liner and the concrete. The calculation addresses the load transfer incorporating reliance on the liner plate anchorage instead of exclusively relying on friction between the plate and the concrete.

The calculation is based on a finite element analysis of the liner plate and its anchorage at the embedded plates and leak chases. The horizontal shear load delivered to the liner plate by the rack pedestals (considering the maximum loads obtained from non-sliding case) is transferred through membrane action of the liner plate into its anchorages. The membrane-action load transfer is achieved by relying only on tension in plane stresses since the thin liner plate does not have any in-plane compression load capacity.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION # 12 RESPONSE (cont'd):

The calculation considers the primary load transfer to be developed through the attachment of the liner plate to the embeds in conjunction with a nominal contribution through friction based on a conservatively low friction coefficient of 0.15 (compared to acceptable range of 0.4 to 0.7). On that basis, the stresses, loads and deformations of the liner plate and its anchorages were determined from the finite element analysis. The tension and shear stresses in the stainless steel plate, the stresses in the weldments of the liner plate to the embedded plates and the leak chases, and the loads in the anchor studs and welded bolt of the embedded plates were verified to be within the allowable values for the governing SSE loading combination. In addition, the out-of-plane deformation that may develop in the liner plate in order to accommodate the in-plane compression imposed in the liner plate is verified to be within the allowable ductility ratio. The compressive strain may result when the in-plane displacement, which results from the membrane tension loading of the plate, forces the plate against the embedments ahead of the rack pedestals where the horizontal load is delivered. The design margins for the various elements and types of loading dercribed above are summarized in the attached Table Q12-1.

Therefore, from the above verification of stresses, loads and deformations, it is established that the liner plate maintains its structural integrity and water tightness under the horizontal seismic load delivered by the racks.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION # 12 RESPONSE (cont'd)

Table Q12-1

STRUCTURAL ITEM	TYPE OF LOADING	DESIGN MARGIN NOTE 1	REMARKS
Liner plate	o in-plane tension stress	1.15	Loads and displacements due to membrane action developed to deliver horizontal shear load to embeds.
	o in-plane shear stress	1.19	
	o flexure due to out-of-plane deformation imposed by membrane compression strain	2.70	
Welds liner plate to embedded plates	o stress in fillet welds	1.02	(same as noted above) 2.2* (see Note 2)
liner plate to leakchase	o stress in backside of penetration weld over leakchase embedded beam	1.05	1.7* (see Note 2)
Embedded plates anchor studs	o shear load	1.01	(same as noted above) 1.14* (see Note 2)
anchor bolt	o shear load (fillet weld of bolt to plate)	1.27	
Liner plate/ concrete	o nominal contribution by friction (μ=.15) to resist horizontal shear load	2.70	

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION # 12 RESPONSE (cont'd):

Table Q12-1 (cont'd)

Notes:

1. Design margin is defined as the ratio of allowable load or stress to calculated design load or stress corresponding to the governing SSE loading combination.

2. Alternative design margins indicated by an asterisk (*) in the remarks column are with respect to 0.9 times ultimate load instead of allowable load (presented for reference when the design margin based on allowable load is less than 1.1).

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #13:

Provide quantitative changes to the FHB seismic model (stick model) due to the increased mass at the pool floor elevation.

RESPONSE #13:

The seismic analysis verification of the Fuel Handling Building (FHB) was performed incorporating the additional weight of the high-density spent fuel storage racks (with consolidated fuel) to address the following items:

1. The shift in center of mass at the level of the spent fuel pool floor due to the additional eccentrically positioned weight, and its potential effect on the torsional response and the related horizontal translational response at the location of the pool.

2. The vertical flexibility of the floor slab of the spent fuel pool, and its potential effect on the in-structure vertical response spectra used for the analysis of the racks and the verification of the spent fuel pool structure within the FHB.

The seismic verification is based on dynamic analyses performed through lumped parameter models with soil-structure interaction considered by the elastic half-space (EHS) method. The three-dimensional lumped parameter models (1) permit adequate accounting of the torsional response obtained under horizontal input due to the eccentricities between centers of mass and centers of resistance within the structure, and (2) account for translational responses that include coupled contributions from the other orthogonal directions of input and from the rotational (rocking and torsional) responses. The adequacy and validity of the EHS analysis is demonstrated by the comparable response spectra obtained from single-step finite element soil-structure interaction analyses. The soil impedance functions for the EHS analysis are based on the South Texas Project (STP) Geotechnical Report (by Woodward Clyde) as described in Reference 3.7.2-13 of the STP FSAR. The ground input motions are applied at the foundation base without introducing any reduction due to the embedment of the structure.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION # 13 RESPONSE (cont'd):

The verification is based on a comparative study of floor response spectra obtained from the original model and the modified model that includes the additional mass of the racks. For the comparison, the response spectra computed for the operating basis earthquake (OBE) at 2% damping are used because any differences between response spectra would be better appreciated from the OBE spectra which, due to their lower damping, exhibit higher amplification than the SSE. However, if desired, the SSE response is obtained by simply doubling the OBE response. This is a conservative approach since the higher damping associated with the SSE normally results in an SSE response that is lower than 2 times OBE.

The comparisons between original and modified models include the following:

1. Configuration of mathematical models

   o Changes in nodal point masses and locations

   o Incorporation of subsystem nodal point flexibly connected (in vertical degree of freedom) to floor master nodal point.

2. Modal frequencies and system damping.

3. Acceleration response spectra at the spent fuel pool floor (El. 21.92 ft.).

Review of the calculation indicates that there are no significant differences in modal frequencies and system damping values between the original and the modified analyses. The governing comparison of response spectra also clearly indicates no significant difference in seismic acceleration response. Therefore, it is established that the original design-basis spectra and corresponding time-histories used for the seismic analysis of the racks are adequate and need not be modified to account for the building mass variation introduced by the heavier racks.

Upon review of the High Density Spent Fuel Racks Safety Analysis Report (reference #1 in the cover letter) during the preparation for response to this question, HL&P determined that changes are appropriate for the second paragraph of Section 6.2 and the Evaluation of No Significant Hazards Considerations to clarify the use of FHB floor response spectra. See attached annotated revision to page 6-2 (Attachment # 4) and the Evaluation of No Significant Hazards Considerations (Attachment #6).

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #14:

How is the integrity of the liner at the floor slab accounted for at point of impact of the rack support?

RESPONSE #14:

During the July 11 & 12, 1988 meeting with the NRC, Bechtel stated that the through-thickness stress induced in the liner plate by the vertical impact loads is only 4.0 ksi, which is a very low value and indicates no compromise of liner plate integrity.

As a result of the subject meeting, the NRC requested a more comprehensive analysis.

A supplementary calculation was performed in response to the request for a more comprehensive analysis. The calculation addresses the following items:

1. The total vertical load from a rack pedestal, which is the maximum value including the impactive component obtained from the non-linear seismic analysis that involves slight uplifting, is applied to check the through-thickness compression stresses in the plate and the bearing stresses on the concrete.

2. The effects of possible irregularities on the surface of the concrete, postulated as a depression at the location where the rack pedestals bear on the liner plate and force it against the concrete, were considered. The resultant out-of-plane deformation imposed on the ductile stainless steel liner plate is demonstrated to be within the allowable ductility ratio.

3. The in-plane strains derived from the curvature and from the extreme-fiber flexural strain of the pool floor slab are imposed on the liner plate. As expected, the curvature of the 6 ft. thick slab is inconsequential on the 1/4 in. thick plate. The in-plane deformation derived from the flexural strain is considered as constrained between the embedded anchorage points of the liner plate, and is demonstrated to be accommodated by out-of-plane deformations within the allowable ductility ratio.

The design margins for the various elements and types of loading described above are summarized in the attached Table Q14-1.

SOUTH TEXAS PROJECT
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Revised Responses to NRC Questions
on Spent Fuel Racks

QUESTION #14 RESPONSE (cont'd):

Table Q14-1

STRUCTURAL ITEM	TYPE OF LOADING	DESIGN MARGIN NOTE 1	REMARKS
Liner plate	o through-thickness compression stress	6.70	
	o flexure due to the out-of-plane deformation imposed at concrete surface depression	4.40	
	o flexure due to the out-of-plane deformation required to accommodate the flexural strain from floor concrete slab	7.10	
	o stress imposed by curvature of floor slab	>10	
Concrete slab	o bearing stress at rack pedestals	1.21	based on concrete tested strength at 90 days

Notes:

1. Design margin is defined as the ratio of allowable load or stress to calculated design load or stress corresponding to the governing SSE loading combination.

ATTACHMENT # 2

CALCULATION OF FORCES ON RACK
WALLS DUE TO REDUCED
RACK-TO-WALL DISTANCE
AT THE NORTH END OF THE SFP

L4/NRC/bx

BASED ON NRC TECHNICAL AUDIT OF JULY 21, 88, ANALYSIS WERE PERFORMED TO CALCULATE THE ADDITIONAL FLUID FORCES ON THE RACK WALLS DUE TO THE REDUCED RACK-TO-WALL DISTANCE AT THE NORTH END OF THE SPENT FUEL POOL.

THE ADDITIONAL LOAD ON THE RACK WALL = 200.4 KIPS TOTAL. (SSE)

PERFORM STRESS ANALYSIS ON 3-CELL SECTION (C-C) DUE TO ADDITIONAL FLUID EFFECTS. EDGE "C" PEDESTAL TOP PLATE IS LOADED BY 3-CELL SECTION FOR CRITICAL WELD STRESSES.

REFERENCE: MECHANICAL REPORT (STRESS ANALYSIS): 8709-00-0120, REV. 4, 03-08-88. SECTION 4, PAGES 37 THRU 41 OF 69.

COMBINED STRESSES (σ_{NS}) IN NORTH-SOUTH DIRECTION ARE CALCULATED ON PG. 37 FOR 3-CELL SECTION (PLANE C-C, SSE)

$$\sigma_{NS} = \frac{M}{S} \pm \frac{P}{A_W}$$

WHERE
M : 12713685 IN·LB.
S : 1378.86 IN^3
P : 0.75F : 129923 LB
A_W : $(\underbrace{8}_{\text{LEVELS}} \times \underbrace{4.32}_{\text{WELD AREA/LEVEL}})$ IN^2

ADDITIONAL LOAD ON PLANE C-C, N-S

P_{RF} = 0.75 (200400) LB. = 150300 LB.

∵ N-S DIRECTION = 12 CELLS. LOAD ON 3 CELL SECTION = 9 ÷ 12 (P_{RF})

P_{TOTAL} = 129923 + 150300 = 280223 LB.

$$\sigma_{NS} = \frac{12713685}{1378.86} \pm \left\{\frac{(129923 + 150300)}{(8 \times 4.32)}\right\}$$

$$= \frac{17329}{1112}$$ PSI (TEN. TOP)
PSI T BOT. Neglect.

REF. Pg. 41, MECHANICAL REPORT

MAXIMUM STRESSES, PLANE C-C

$$\tau_S = \tau_{SWT} + \sqrt{(\tau_{NS})^2 + (\tau_{EW})^2 + (\tau_{VERT})^2}$$

$$\tau_S = 3833 + \sqrt{(9920)^2 + (4754)^2 + (5232)^2} = 16014 \text{ PSI}$$

$$\sigma_{AXIAL} = \sigma_{SWT} + \sqrt{(\sigma_{EW})^2 + (\sigma_{NS})^2 + (\sigma_{VERT})^2}$$

$$\sigma_{AXIAL} = 1778 + \sqrt{0 + (17329)^2 + (2427)^2} = 19276 \text{ PSI}$$

$$\tau_{MAX} = \sqrt{\left(\frac{\sigma_{AXIAL}}{2}\right)^2 + (\tau_S)^2}$$

$$= \sqrt{\left(\frac{19276}{2}\right)^2 + (16014)^2} = 18691 \text{ PSI}$$

τ_{MAX} = 18691 PSI $<$ 0.42 Su ALLOW. = 27250 PSI @ 225°C

F.S. = 1.46

MAXIMUMUM NORMAL STRESS

$$\sigma_{MAX} = \frac{\sigma_{AXIAL}}{2} + \tau_{MAX} = \frac{19276}{2} + 16014 = 25652 \text{ PSI}$$

CONCLUSION

1. PLANE C-C IS MORE CRITICAL THAN PLANE B-B (6-CELL SECTION). PLANE B-B O.K BY COMPARISION TO PLANE C-C.
2. FACTOR OF SAFETY FOR PLANE C-C (3-CELL SECTION) CHANGED FROM 1.54 (Pg. 41 of 69) TO 1.46.
3. OVERALL STRESSES O.K. E-W (SSE) LOADS ARE MORE LIMITING THAN NS-SSE LOADS

RACK #2, 110-CELL, HALF-FULL

WEIGHT OF NODAL MASSES			LBS.
RACK NODE	15		4059
	16		3891
	17		3891
	18		3891
	19		3942
	20		3263
FUEL	21	(47090 × .50)	23545
	22	(84971 × .50)	42486
	23	(84971 × .50)	42486
	24	(84971 × .50)	42485
	25	(63690 × .50)	31845
	26	(21243 × .50)	10621
ROTARY INERTIA	27	$\frac{1}{12}(236597)(100.65^2)$	EW 1.9974E8
		91.50	NS 1.6507E8
	28	$\left(\frac{\overset{DWT}{49873} + \overset{ENT. H_2O}{63321}}{2}\right)$	236597

VERTICAL STATIC DEFLECTION (RACK #2)

$$= -\frac{187548}{2(18.674E6)} = -0.005022$$

SWT = 187548 LB.

ENTRAINED WATER

$$= (8.90)^2(201.3125)(.0361)(\overset{CELLS}{110}) = 63321 \text{ LB.}$$

DWT : 236597 LB. (INCLUDES ENTRAINED H_2O)

DWT RACK : 22937 LB. BWT RACK : 3437 LB. SWT_{RACK} : 19500 LB

FUEL WT : 193468 LB. BWT FUEL : 25420 LB. SWT_{FUEL} : 168048 LB

SWT (RACK + FUEL) = (19500 + 168048) = 187548 LB.

(Pg 2, CONSOLIDATED FUEL, 110-CELL RACK)

RACK #3, 110-CELL EMPTY RACK

CONSIDER RACK + ENTRAINED WATER ONLY.

WEIGHT OF NODAL MASSES (LBS)

NODE 29	:	4059 + .1016(63321)	=	10492
30	:	3891 + .1833(63321)	=	15498
31	:	3891 + .1833(63321)	=	15498
32	:	3891 + .1833(63321)	=	15498
33	:	3942 + .1833(63321)	=	15549
34	:	3263 + .1652(63321)	=	13724
35 ROTARY INERTIA @ RACK BASE		$\frac{1}{12}(86259)(100.65^2)$	EW =	7.282E7
		$\frac{1}{12}(86259)(91.50^2)$	NS =	6.018E7
36		Σ M_{29} THRU M_{34}	=	86259

VERTICAL STATIC DEFLECTION (RACK #3)

$$= -\frac{19500}{2(18.674E6)} = -0.000522$$

SWT = 19500 LB

DWT : 86259 LB (INCLUDING ENTRAINED H_2O)

DWT (RACK) = 22937 LB (INCLUDES COMPONENTS)
BWT (RACK + COMPONENTS) = 3437 LB.
(REFER pg. 2 OF 110-CELL, CONSOLIDATED FUEL)

SWT = 22937 - 3437 = 19500 LBS

U.S. TOOL & DIE, INC.

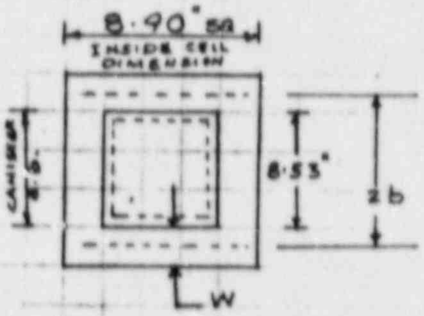
BY Q.t DATE 08/02/88 SUBJECT 110 - CELL RACK SECT. NRC Q SHEET A5 OF A16
CHKD. BY [illegible] DATE 8/3/88 WALL COUPLING FORCES PROJ. NO. 8709-30

WALL COUPLING FORCES, CONSOLIDATED FUEL. (RACK TO FUEL)

$b = \frac{1}{2}\left(\frac{8.90 + 8.53}{2}\right) = 4.36"$

$W = \left(\frac{8.90 - 8.53}{2}\right) = 0.185"$

$h = 176"$ (CANISTER)



CONSOLIDATED FUEL CELL-SECTION

$M_H^{TF} = \frac{16}{3} \rho \frac{b^3 h}{W}$ (PER CELL)

$M_H^{TF} = \frac{16}{3} \frac{(.0361)(4.36)^3 (176)(\text{NO. CELLS})}{0.185}$

$= (1581.21)(110\text{ - CELLS}) = \underline{166933}$ LB

M_1^F = WATER DISPLACED BY FUEL (CANISTER)

$= (8.53)^2 (176)(.0361)(110 \text{ cells}) = \underline{50852}$ LB.

$M_1^T = L \cdot W \cdot H \cdot \rho_w = (201.3125)(9.15)(9.15)(.0361)(110) = \underline{66929}$ LB.

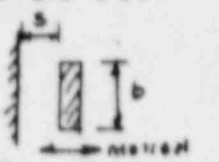
M_2^T = WATER INSIDE THE CELL IN THE ABSENCE OF THE FUEL

$= (8.90)^2 (.0361)(201.3125)(110 \text{ cells}) = 63321$ LB

RACK TO WALL COUPLING FORCES.

$M_H^{TW} = \frac{\rho h^3 \cdot b}{12 (S)} = \frac{(.0361)(201.3125)^3}{12} \left(\frac{b}{S}\right)$ PER CELL

$= 24544 \left(\frac{b}{S}\right)$ PER CELL



$b : (10 \times 9.15) = 91.50"$ RACK WIDTH E-W

$b : 11 \times 9.15) : 100.65"$ RACK WIDTH N-S

PHASE 3 INSTALLATION

S: MAX. DISTANCE BETWEEN POOL WALL & EDGE OF RACK

EAST	S :	16.58"
WEST	S :	32.14"
NORTH	S :	5.29"
SOUTH	S :	22.64"

M_H^{rw}

NORTH

$M_H^{rw} = 24544 \left(\frac{b}{S}\right)$ REF: SHT 52

NORTH $S = 15.29$ - IN b : 100.65

SOUTH $S = 1522.64$ = IN

M_H^{rw}

NORTH: 4.67 E5

SOUTH: 1.09 E5

$M_{rr} = \frac{\rho h^3 b}{12 S} = \frac{24544 (100.65)}{(1)} = 2.47E6$

$M_1^f = 50852$ LB

$M_2^r = 63321$ LB

$M_1^r = 66929$ LB.

$M_H^{rf} = 164146$ 8.

$F_{kk} = \alpha (M_H^{rw} + M_2^r + M^{rr}) + \beta (M_H^{rf} + M_1^f)$

RACK 1

$F_{1,1} = .1016(4.67E5 + 63321 + 2.47E6) + .1217(164146 8 + 50852)$
= 510788.

$F_{2,2} - F_{4,4} = .1833($ " $) + .2196($ $) = 921592.$

$F_{5,5} = .1833($ " $) + .2195() = 921423$

$F_{6,6} = .1652(4.67E5 + 2.47E6)^* = 485192.$

* - ENTRAINED WATER INCLUDED IN RACK MASS

HALF FULL

RACK 2

$F_{ii} = \alpha(M_i^{f} + 2M^{ff}) + \frac{\beta}{2}(M_H^{ff} + M_i^{f})$

$F_{15,15} = .1016(6332\,1 + 2[2.47E6]) + \frac{.1217}{2}(16+1468+50852$

$= 611315$

$F_{16,16} - F_{18,18} = .1833(\quad) + \frac{.2196}{2}(\quad) = 1102925$

$F_{19,19} = .1833(\quad) + \frac{.2195}{2}(\quad) = 1102840$

$F_{20,20} = .1652(2[2.47E6]) = 816088.$

RACK 3

INCLUDE ENTRAINED WATER IN RACK MASS NODES

RACK TO RACK COUPLINGS

$F_{i,1} = F_{1,i} = \alpha[M_H^{ff}]$

$F_{15,1} = F_{1,15} = .1016(2.47E6) = 2.51E5$

$F_{16,2} = F_{2,16} = .1833(2.47E6) = 4.528E5$

$F_{17,3} = F_{3,17} =$ " "

$F_{18,4} = F_{4,18} =$ " "

$F_{19,5} = F_{5,19} =$ " "

$F_{20,6} = F_{6,20} = .1652(2.47E6) = 4.08E5$

U.S. TOOL & DIE, INC.

BY PMB DATE 8/3/88 SUBJECT STP-NRC SECT. ____ SHEET A8 OF A10
CHKD. BY A.t DATE 08/03/88 MULTI-RACK 110 CELLS PROJ. NO. 8709

$F_{29,15} = F_{15,29} = 2.51E5$

$F_{30,16} = F_{16,30} = 4.528E5$

$F_{31,17} = F_{31,17} =$ "

$F_{32,18} = F_{18,32} =$ "

$F_{33,19} = F_{19,33} =$ "

$F_{34,20} = F_{20,34} = 4.08E5$

RACK TO WALL COUPLING FOR CENTER RACK

$\alpha(M_1^r)$

$F_{15,37} = .1016\,(66929) = 6800.$

$F_{16,37} = .1833\,(66929) = 12268$

$F_{17,37} =$ " $= 12268$

$F_{18,37} =$ " $= 12268$

$F_{19,37} =$ " $= 12268$

$F_{20,37} = .1652\,(66929) = 11057.$

RACK TO WALL COUPLING FOR SIDE RACKS

$\alpha(M_1^r + M^{rw})$

		N	S
$F_{1,37} =$	$.1016\,(66929 + M_N^{rw})$	$= 54847$,	$F_{29,37} = 17874$
$F_{2,37} =$	$.1833\,($ " $)$	$= 98769$,	$F_{30,37} = 32248$
$F_{3,37} =$	$.1833\,($ " $)$	$=$ " ,	$F_{31,37} = 32248$
$F_{4,37} =$	$.1833\,($ " $)$	$=$ " ,	$F_{32,37} = 32248$
$F_{5,37} =$	$.1833\,($ " $)$	$=$ " ,	$F_{33,37} = 32248$
$F_{6,37} =$	$.1652\,($ " $)$	$= 88205$,	$F_{34,37} = 29063$

U.S. TOOL & DIE, INC.

BY WAB DATE 8/3/88 SUBJECT STP-NRC SECT. ____ SHEET A9 OF A10
CHKD. BY A.C DATE 08/03/88 MULTI-RACK 110 CELLS PROJ. NO. 8707

RACK TO FUEL

$F_{1,7} = F_{7,1} = 205955$

$F_{2,8} = F_{8,2} = 371633$

$F_{3,9} = F_{9,3} = 371633$

$F_{4,10} = F_{10,4} = 371633$

$F_{5,11} = F_{11,5} = 371464$

$F_{15,21} = F_{21,15} = 102978$

$F_{16,22} = F_{22,16} = 185817$

$F_{17,23} = F_{23,17} =$ "

$F_{18,24} = F_{24,18} =$ "

$F_{19,25} = F_{25,19} = 185732$

FUEL 1	FUEL 2
$F_{7,7} = 199767$	$F_{21,21} = 99884$
$F_{8,8} = 360466$	$F_{22,22} = 180233$
$F_{9,9} =$ "	$F_{23,23} = 180233$
$F_{10,10} =$ "	$F_{24,24} = 180233$
$F_{11,11} = 360302$	$F_{25,25} = 180151$

EMPTY RACK (3)

$F_{29,29} = 250952$

$F_{30,30} = 452751$

$F_{31,31} = 452751$

$F_{32,32} = 452751$

$F_{33,33} = 452751$

$F_{34,34} = 408044$

ATTACHMENT #2
ST-HL-AE-2764
PAGE 10 OF 10

US TOOL & DIE, INC.

BY SMC DATE 8/1/88 SUBJECT STP-NRC SECT SHEET N/C OF N/C
CHKD BY D.C. DATE 08/03/88 J RACK 110 CELLS PROJ NO 8709

RESULTS

	MAXIMUM RELATIVE DISPLACEMENT
NON-SLIDING	.34"
SLIDING (.2 FRICTION)	.61"

ATTACHMENT # 3

INFORMATION ON THE 3-RACK ANALYSIS

TABLE OF CONTENTS

ROTATIONAL INERTIA COMPARISION

UST&D presented a comparative study to the Staff on July 21, 1988 during the NRC technical audit of the UST&D facilities of both the single lumped rotational inertia located at the base of the rack and the multiple mass distribution over the vertical height of the rack. The multiple mass was distributed over six levels of the rack stick model. The individual mass of the fuel cell and the fuel assembly was combined as a single mass at each level. The analysis was performed using unconsolidated spent fuel. The results of this study indicated an eight (8) percent increase in loads for the multiple mass distribution method.

After the July 21, 1988 meeting, UST&D performed a subsequent multiple mass distribution model separating the mass of the fuel cell and the fuel at each level of the rack stick model. This model contains 12 mass points, 2 at each level. This analysis was performed using consolidated spent fuel. UST&D believes that this method of modelling the mass distribution is more representative of the actual mass distribution. This analysis resulted in a small decrease in the loads on the rack and a small increase in the factor of safety. For example, the rack pedestal bottom plate factor of safety was calculated as 1.07 using this multiple mass distribution model. Previously, using the single lumped rotational inertia located at the base of the rack, this factor of safety was calculated as 1.06.

During the July 21, 1988 meeting it was our understanding that the staff found the results of the multiple mass distribution model acceptable. We believe that our subsequent analysis, which more explicitly defines the mass distribution, provides even a higher level of confidence in our original analysis using the single lumped rotational inertia located at the base of the rack.

DISTRIBUTED ROTATIONAL INERTIA 13C RACK (CONSOLIDATED)

$M \frac{L^2}{12}$ NS: $\frac{L^2}{12} = \frac{(109.8)^2}{12} = 1004.67 \text{ in}^2$

EW: $\frac{(100.65)^2}{12} = 844.2$

6 MASS DISTRIBUTION

	NS	EW
$M_{13} = (4710 + 53627)\frac{L^2}{12} =$	5.86 E7	4.92 E7
$M_{15} = (4669 + 97111)\frac{L^2}{12} =$	1.02 E8	8.59 E7
$M_{16} = (4669 + 97111)\frac{L^2}{12} =$	1.02 E8	8.59 E7
$M_{17} = (4669 + 97111)\frac{L^2}{12} =$	1.02 E8	8.59 E7
$M_{18} = (4731 + 97111)\frac{L^2}{12} =$	1.02 E8	8.6 E7
$M_{19} = (13470 + 21025)\frac{L^2}{12} =$	3.47 E7	2.91 E7

12 MASS DISTRIBUTION (USED)

	EW	NS		EW	NS
M_{13} =	3.98 E6,	4.73 E6	M_{20} =	4.53 E7,	5.39 E7
M_{15} =	3.94 E6,	4.69 E6	M_{21} =	8.2 E7,	9.76 E7
M_{16} =	3.94 E6,	4.69 E6	M_{22} =	8.2 E7,	9.76 E7
M_{17} =	3.94 E6,	4.69 E6	M_{23} =	8.2 E7,	9.76 E7
M_{18} =	3.99 E6,	4.75 E6	M_{24} =	8.2 E7,	9.76 E7
M_{19} =	1.24 E7,	1.35 E7	M_{25} =	1.77 E7,	2.11 E7

MULTI-MASS DISTRIBUTION WITH INDIVIDUAL ROTARY INERTIA. 132-CELL RACK, REGION 2, CONSOLIDATED FUEL, 201.3125" RACK, 176" CANISTER

MAXIMUM FORCES ON TWO PEDESTALS (LB)

	FVt	Fhz
EW-SSE	376735	128344
NS-SSE	399155	166798

MAXIMUM FORCES AT BASE OF RACK (LB)

	VERTICAL $FVT = \left(FVt - \frac{SWT}{2}\right)$	HORIZONTAL Fhz
EW-SSE	163549	128344
NS-SSE	185969	166798
V - SSE	334739	0
SWT/2	213186	0
SRSS (2-PEDESTALS)	629579	210461

MAXIMUM FORCES ON ONE PEDESTAL (LB)

$\frac{FVT}{2}$ = 314790 LB VERTICAL

$\frac{Fhz}{2}$ = 105231 LB HORIZONTAL

NOTES: 1) FVt: LOAD ON TWO PEDESTALS, INCLUDING SUBMERGED WEIGHT.

2) $FVT_V = \frac{1.5(g)(DWT)}{2}$

3) $SRSS = \sqrt{(FVT_{EW})^2 + (\ldots)^2 + (FVT_V)^2} + \left(\frac{SWT}{2}\right)$

4) DWT = 490460
SWT = 426373

5) ACCEL 0.91(g)

STP-NRC
SINGLE LUMPED

U.S. TOOL & DIE, INC

BY A.T. DATE 07/29/88 SUBJECT SEISMIC LOAD SECT NRC2 SHEET 4 OF 52
CHKD. BY ____ DATE 8/1/88 SUMMARY (SSE) PROJ. NO. 8709 20

SINGLE LUMPED ROTARY MASS INPUT.
132-CELL RACK, REGION 2, CONSOLIDATED FUEL. 201.3125" RACK, 176" CANISTER

MAXIMUM FORCES ON TWO PEDESTALS (LB)

	FVt	Fhz
EW-SSE	399978	142263
NS-SSE	395750	159224

MAXIMUM FORCES AT BASE OF RACK (LB)

	VERTICAL $FVT = \left(FVt - \frac{SWT}{2}\right)$	HORIZONTAL Fhz
EW-SSE	186792	142263
NS-SSE	182564	159224
V-SSE	334739	0
(SWT/2)	213186	0
SRSS(2-PEDESTALS)	637769	213521

MAXIMUM FORCES ON ONE PEDESTAL (LB)

$\left(\frac{FVT}{2}\right)$ = 318885 LB VERTICAL

$\left(\frac{Fhz}{2}\right)$ = 106761 LB HORIZONTAL

NOTES:
1) FVt: LOAD ON TWO PEDESTALS, INCLUDING SUBMERGED WEIGHT
2) $FVT_V = \frac{1.5(g)(DWT)}{2}$
3) $SRSS = \sqrt{(FVT_{EW})^2 + (FVT_{NS})^2 + (FVT_V)^2} + \left(\frac{SWT}{2}\right)$
4) DWT = 490460 LB.
SWT = 426373 LB.
5) ACCEL.: 0.91(g)

COMPARISION OF PEDESTAL LOADS (SSE) GENERATED BY "RACKOE" COMPUTER PROGRAM FOR 132-CELL RACK IN REGION 2, (201.3125" RACK), USING INDIVIDUAL ROTARY INERTIA AT EACH NODAL MASS VS SINGLE LUMPED MASS OF ROTATIONAL INERTIA.
ALL OTHER PROPERTIES IDENTICAL.

ITEM	(1) PEDESTAL LOADS VERTICAL (KIPS)	HORIZ. (KIPS)	SRSS $\sqrt{(VERT)^2 + (HORIZ)^2}$ (KIPS)
1. SINGLE LUMPED ROTARY MASS.	318.89	106.76	336.29
2. MULTI-MASS DISTRIBUTION	314.79	105.23	331.91

CONCLUSION

MULTI-MASS DISTRIBUTION SYSTEM WILL YIELD 1.3% LOWER PEDESTAL LOADS

WITH RESPECT TO USING SINGLE LUMPED ROTARY MASS.

ASSUMING SEISMIC LOADS USED IN THE MECHANICAL REPORT ARE LOWER BY 1.3%, THE LOWEST FACTOR OF SAFETY OF 1.06 FOR PEDESTAL RACK BOTTOM PLATE TO BOX WALL, SECTION 4.2, SHOWN IN TABLE 1.2, SHALL BE INCREASED BY 1.3%

i.e. F.S. 1.06 x 1.013 = 1.07

FUEL GAP STUDY

ACTION #17:

The NRC requested that UST&D review their analysis to ensure that all possible situations of gap between the fuel assembly and rack side have been addressed and that the "worst-case" is considered. The results of this review will be submitted to the NRC during the week of July 25, 1988.

RESPONSE:

In response to the NRC request additional fuel to rack initial gap configurations runs were made to assure that the "worst-case" was considered. The results, shown below, indicate that the difference in resulting loads are negligible.

	BASE CASE	MAXIMUM DISPLACEMENT	TILTED DISPLACEMENT
Fvmax	405.8 KIPS	413 KIPS	404 KIPS
Fhmax	173 KIPS	178 KIPS	172 KIPS

ROCKOS INPUT

TILTED ASS'Y

$GAP = 4\left(\frac{.37}{20.1}\right) \approx .185$

H = 201 REF 8709-... SECT 4 SH ...

Y = 10.23, 38.9, 75.8, 112.7, 149.6, 186.5

Initial
Disp. = -.166, -.113, -.045, .022, .09, .158

FLUSH AGAINST WALL

INITIAL DISP = .185, .185, .185, .185, .185, .185 ✓

132-CELL RACK BOUNDING JUSTIFICATION

ACTION D 2:

The NRC requested that UST&D provide the bases for concluding why the Region 2, 132-cell analysis bounds the other Region 2 rack cell sizes with respect to the footing design. This information will be submitted to the NRC during the week of July 25, 1988.

RESPONSE:

The past UST&D rack design experiences have shown that the rack with the greatest mass would transmit the largest loads to the footings (pedestals). In Region 2 of the South Texas Project, 132-cell rack is the largest and the smallest is the 110-cell rack.

The seismic loads due to SSE events were generated by UST&D, for Region 2, 110-cell rack, similar to the largest 132-rack analysis.

THE RESULTS ARE SUMMARIZED BELOW:

SRSS LOAD ON ONE PEDESTAL (FOOTING):

	110-CELL RACK	132-CELL RACK
VERTICAL	430300 LBS	441129 LBS
HORIZONTAL	128476 LBS	141937 LBS
VERTICAL LIFT-OFF (E-W)	0.1869"	0.1310"

Conclusion: The 132-cell rack in Region 2 bounds the other (smaller) Region 2 racks, with respect to the footing design.

110 - CELL RACK
CONSOLIDATED FUEL

(***) SET #4: MAXIMUM FORCES ON A PEDESTAL (LBS)

	VERTICAL (FVT/2)	HORIZONTAL (FhZ/2)
NS - OBE		
EW - OBE		
V - OBE		
SRSS		
NS - SSE	133916	73600
EW - SSE	281166	105305
V - SSE	139870	0
*** SRSS	430300	128476

(***) SET #4 ARE HALF SET #3 VALUES.

SET #5: MOVEMENT AT BASE (inches)

	ELASTIC (NON-SLIDING) (DISPLACEMENT)	SLIDING (FRICTION)	LIFTOFF (VERTICAL)
NS - OBE			
EW - OBE			
NS - SSE	0.3078		0.05144"
EW - SSE	0.2924		0.1869"

MAXIMUM FRICTION FORCES (LB) USING 0.2 FACTOR N/A

NS - OBE
EW - OBE
NS - SSE
EW - SSE

		REF: 110-CELL RACK
DWT =	409874 LB	Pg. 2
BF =	54257 LB	Pg. 2
SWT =	355617 LB	Pg. 2

110 - CELL RACK
CONSOLIDATED FUEL

SET #1: MAXIMUM FORCES IN KIPS TAKEN FROM COMPUTER SUMMARY (NON-SLIDING CASE) AT 5 GAP ELEMENTS

						TWO PEDESTAL*	
(PER COMPUTER) OUTPUT	(1 or 2)	(3 or 4)	(5 or 6)	(7 or 8)	(9 or 10)		
(PER FIGURE)	(a or b)	(c or d)	(e or f)	(g or h)	(i or j)	(vert)	(horz)
DIRECTION	1	2	3	4	5	FVt	FhZ
NS - OBE							
EW - OBE							
NS - SSE	28.03	27.25	29.28	31.30	33.51	445.64	147.2
EW - SSE	24.55	27.93	31.86	35.65	39.40	740.14	210.[illegible]

SET #2: FORCES ON INDIVIDUAL FUEL ASSEMBLY (LBS)
(TOTAL F.A. = 110

						ONE PEDESTAL (KIPS	
NS - OBE							
EW - OBE							
NS - SSE	255	248	266	285	305	222.82	73.
EW - SSE	223	254	290	324	358	370.07	10[illegible]

SET #3: MAXIMUM FORCES AT BASE OF RACK (LBS)

	VERTICAL $FVT=(FVt-\frac{SWT}{2})$	HORIZONTAL FhZ
NS - OBE		
EW - OBE		
V - OBE		
SRSS		
NS - SSE	267832	147200
EW - SSE	562332	210610
** V - SSE	279739	0
*** SRSS	860600	256952

---

* FORCES (FVt) ON TWO PEDESTALS INCLUDE THE EFFECT OF SUBMERGED WEIGHT (SWT)

$$** \; FVT_V = \frac{1.5\,(g)\,DWT}{2} = \frac{1.5 \times .91 \times \overset{4cell}{409874}}{2} = 279739$$

$$*** \; SRSS = \sqrt{(FVT_{NS})^2 + (FVT_{EW})^2 + (FVT_V)^2} + \frac{SWT}{2} \; --- \; (SET \# 3$$

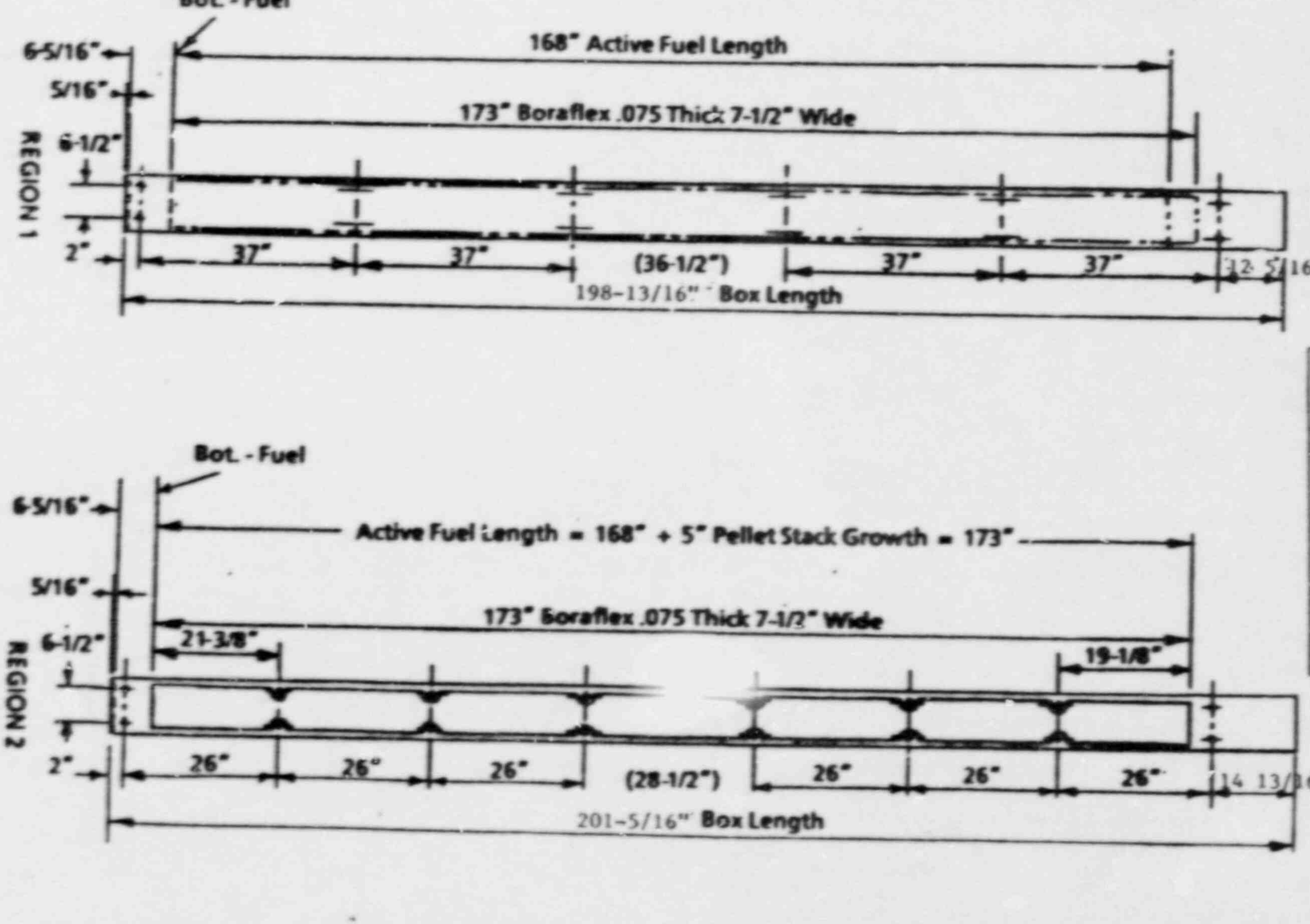


Figure 6-17
LOCATION OF
BOX FUSION WELDS AND POISON

ATTACHMENT # 3
ST-HL-AE-2764
PAGE 12 OF 54

THE SMALLEST RACK IN REGION 2 IS 110-CELL RACK PER USTD DWG. 8709-1. (11 CELLS EW X 10 CELLS NS)

REF.: SEISMIC CALCULATIONS, SECTION 6, SUBSECTION 6.2, "WEIGHT OF MASSES"

110-CELL, WEIGHT OF MASSES

FUEL BOXES

179.58 LB/CELL x 110 CELLS = 19754 LB

POISON

11.52 LB/CELL x 110 CELLS = 1267 LB

BOTTOM PLATES

10.15 LB/CELL X 110 CELLS = 1117 LB

PEDESTALS

(SAME AS 132-CELL RACK) = 800 LB

BUOYANT FORCES

RACK: 22.20 LB/CELL x 110 CELLS = 2442 LB

POISON: 6.89 LB/CELL x 110 CELLS = 758 LB

BOTTOM PLATES:
1.256 LB/CELL x 110 CELLS = 138 LB

PEDESTALS
$800 \times \frac{.0361}{.292}$ = 99 LB

CONSOLIDATED FUEL

= WT. OF FUEL + CANISTER + WATER INSIDE CANISTER (PER CELL)
= (6.20 LB/ROD x 528 RODS) + (150 LB) + (94* LB) = 3517.6 LB/CELL
(*CONSIDER WITH DRY MASSES)

WT. OF CONSOLIDATED FUEL MASS = 3517.6 LB/CELL x 110 CELLS
= 386936 LB

BUOYANT WEIGHT OF CONSOLIDATED FUEL
= SUBMERGED WEIGHT OF CANISTER
$= 8.53^2 \times 176 \times .0361 = 462$ LB/CELL

$= 462$ LB/CELL $\times$ 110 CELL $= 50820$ LB

SUMMARY (WEIGHTS OF MASSES), REGION 2, 110-CELL, CONSOLIDATED FUEL (LB)

ITEM	DRY WT (DWT)	BUOYANT WT (BWT)	SUBMERGED WT (SWT)
RACK	19754	2442	17312
POISON	1267	758	509
BOTTOM PLATES	1117	138	979
PEDESTALS (4)	800	99	701
CONSOLIDATED FUEL	386936	50820	336116
TOTAL WT. (LBS)	409874	54257	355617

RACK MODELLED WITH 6-LUMPED MASSES.
201.3125" RACK

MASS NODE		MASS DISTRIBUTION (DRY-WEIGHT)		110-CELL NODAL MASSES
1	RACK	0.1016 × 19754	=	2007
	POISON	0.1066 × 1267	=	135
	BOTTOM PL^S	1 (1117)	=	1117
	PEDESTALS	1 (800)	=	800
				4059
2	RACK	0.1833 × 19754	=	3621
	POISON	0.2133 × 1267	=	270
				3891
3 & 4	RACK	0.1833 × 19754	=	3621
	POISON	0.2133 × 1267	=	270
				3891
5	RACK	0.1833 × 19754	=	3621
	POISON	0.2535 × 1267	=	321
				3942
6	RACK	0.1652 × 19754	=	3263
	WATER MASS ABOVE FUEL	$(8.90)^2(33.3125)(.0361)(110)$	=	10478 **
				13741
7	FUEL	0.1217 × 386936	=	47090
8, 9, 10	FUEL	0.2196 × 386936	=	84971
11	FUEL	0.1646 × 386936	=	63690
12	FUEL	0.0549 × 386936	=	21243
13	ROTARY INERTIA @ BASE OF RACK DUE TO VERTICAL MOTION.	$\frac{1}{12}(L^2)$ (MASS NODE 14)		
		EW, L = 11×9.15" = 100.65"	=	3.4602 E8
		NS, L = 10×9.15" = 91.50"		2.8596 E8
14	MASS Σ 1 THRU 12		=	409873 **

EXCLUDING WATER MASS ABOVE FUEL AT NODE 6, FOR VERTICAL MOTION.

** EXCLUDED FOR VERTICAL MOTION

REFERENCE: SECTION 6, SUBSECTION 6.6, pg. 7/14
$M_H^{\tau f}$: 14922.44 PER CELL

$M_H^{\tau f}$ = 14922.44 (110) (CONS) = **1641468 LB**

M_1^f = WATER DISPLACED BY FUEL IN CANISTER
= $(8.53)^2$ (176)(0.0361)(110 cells) = 50852 LB.

M_1^τ = L.W.H. ρ_w L = 201.3125", ρ = .0361
W = H = 9.15"
= (201.3125)(9.15)(9.15)(.0361)(110 cells) = 66929 LB

M_2^τ = WATER IN CELL IN ABSENCE OF FUEL
= $(8.90)^2$ (.0361)(201.3125)(110 cells) = 63321 LB

RACK TO WALL COUPLING FORCES

$$M_H^{\tau W} = \frac{\rho \cdot h^3 \cdot b}{12S} \qquad \rho = 0.0361 \text{ LB/IN}^3 \qquad h = 201.3125"$$

$$= \frac{(.0361)(201.3125)^3}{12} \cdot \left(\frac{b}{S}\right)$$

$$M_H^{\tau W} = 24544 \left(\frac{b}{S}\right) \text{ LB} \quad \text{PER CELL}$$

b: 91.50" = WIDTH OF RACK, EAST - WEST MOTION
b = 100.65" = WIDTH OF RACK, NORTH - SOUTH MOTION
S = MAX. DISTANCE BETWEEN POOL WALL & EDGE OF RACK

EAST S = 15" MAXIMUM

WEST S = 33.72" MAXIMUM

NORTH S = 12.93" MAX.

SOUTH S = 15.0" MAX.

HYDRODYNAMIC COUPLING FORCES CALCS.

EAST (RACK TO POOL WALL)
b = 91.50", S = 15" MAX

$$M_H^{\tau W} = 24544 \left(\frac{91.50}{15}\right) = 149718 \text{ LB/CELL}$$

WEST $b = 91.50''$, $S = 33.72''$

$$M_H^{TW} = 24544 \left(\frac{91.50}{33.72} \right) = 66601 \text{ LB/CELL}$$

NORTH

$b = 100.65''$ $S = 12.93''$

$$M_H^{TI} = 24544 \left(\frac{100.65}{12.93} \right) = 191056 \text{ LB/CELL}$$

SOUTH $b = 100.65''$ $S = 15.0''$

$$M_H^{TW} = 24544 \left(\frac{100.65}{15.0} \right) = 164690 \text{ LB/CELL}$$

(*) LENGTH WEIGHTED HYDRODYNAMIC FORCE ON ONE RACK. (AVERAGE)

EAST-WEST DIRECTION

RACKS IN EAST-WEST DIRECTION PER ROW ARE 2-110 CELL & 1-121 CELL RACKS, WHICH WOULD CONTRIBUTE IN CALCULATING RACK TO WALL COUPLING FORCES.

$$M_H^{TW} = \frac{110(\overset{E}{149718} + \overset{W}{66601})}{\underset{\text{RACKS IN E-W DIRN. PER ROW}}{(*)(110 + 121 + 110)}} = \frac{69780 \text{ LB}}{(E-W)}$$

NORTH-SOUTH DIRECTION

RACKS IN NORTH-SOUTH DIRECTION ARE 3-110 CELL, 1-121 CELL & 2-132 CELL RACKS, PER ROW.

$$M_H^{TW} = \frac{110(\overset{N}{191056} + \overset{S}{164690})}{\underset{\text{RACKS IN N-S DIRN. PER ROW}}{(*)(110 + 110 + 110 + 121 + 132 + 132)}} = \frac{54730 \text{ LB.}}{(N-S)}$$

4

MATHEMATICAL STICK MODEL FOR

DOUBLE DENSITY FUEL RACK: 201.3125"

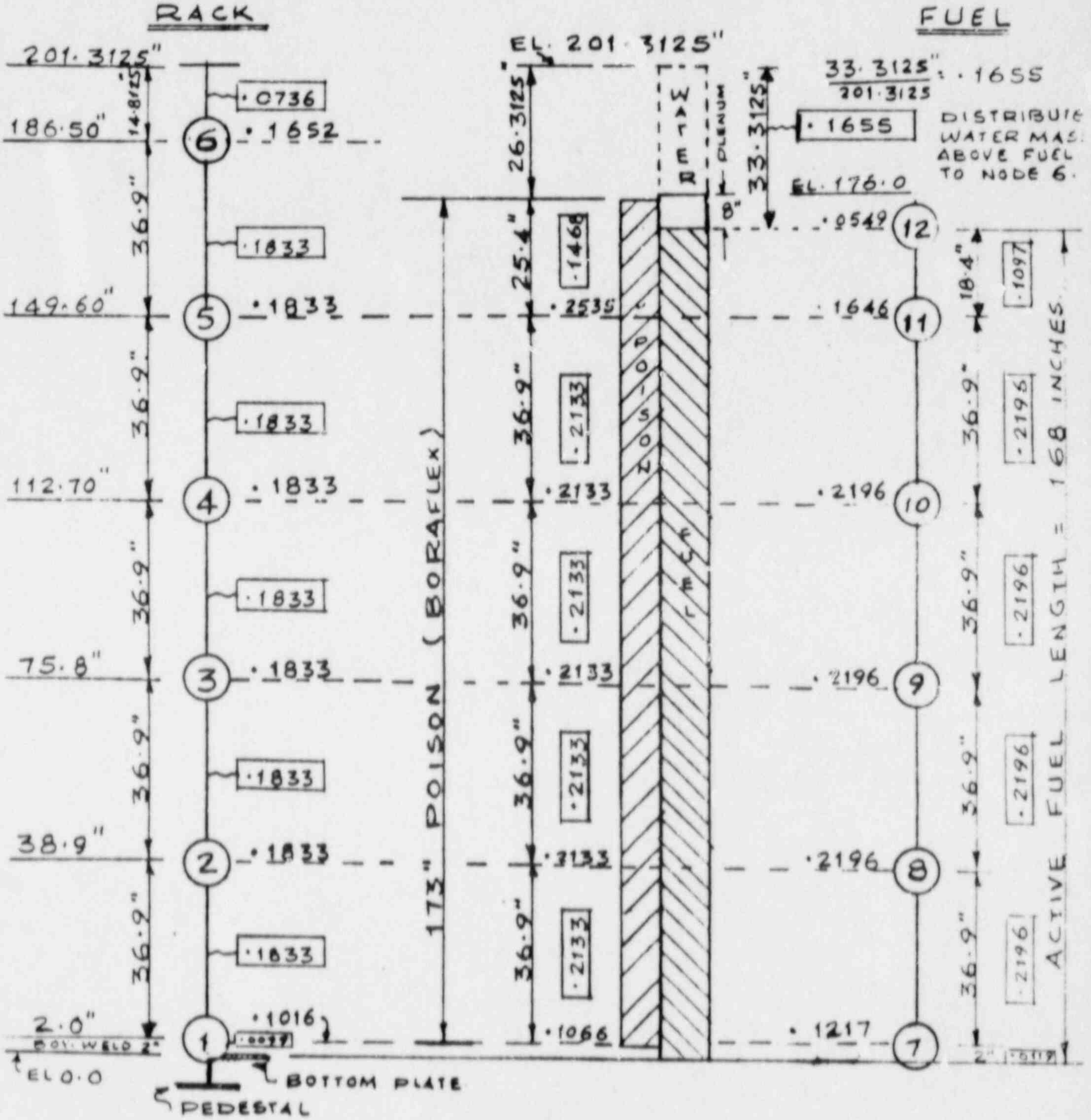


FIGURE 6.0

6.7 HYDRODYNAMIC COUPLING FORCES FOR CONSOLIDATED FUEL, 132-CELL, RACK LENGTH = 201.3125", REGION 2

$F_{1,1} = .1016(M_H^{rw} + M_2^r) + .1217(M_H^{rf} + M_1^f)$

$F_{2,2} \cdots F_{4,4}$

$= .1833(M_H^{rw} + M_2^r) + .2196(M_H^{rf} + M_1^f)$

$F_{5,5} = .1833(M_H^{rw} + M_2^r) + .2195(M_H^{rf} + M_1^f)$

$F_{6,6} = .1652(M_H^{rw})$

$F_{7,7} = .1217(M_H^{rf})$

$F_{8,8} \cdots F_{10,10} = .2196(M_H^{rf})$

$F_{11,11} = .2195(M_H^{rf})$

$F_{1,7} = F_{7,1} = .1217(M_H^{rf} + M_1^f)$

$F_{2,8} = F_{8,2} = F_{3,9} = F_{9,3} = F_{4,10} = F_{10,4}$

$= .2196(M_H^{rf} + M_1^f)$

$F_{5,11} = F_{11,5} = .2195(M_H^{rf} + M_1^f)$

WALL COUPLING FORCES

$F_{1,15} = .1016(M_H^{rw} + M_1^r)$

$F_{2,15} \cdots F_{5,15} = .1833(M_H^{rw} + M_1^r)$

$F_{6,15} = .1652(M_H^{rw} + M_1^r)$

HYDRODYNAMIC COUPLING FORCES FOR CONSOLIDATED FUEL, 110-CELL, EAST-WEST RACK 201.3125", REGION 2.

RACK COUPLINGS

$F_{1,1} = 0.1016(69780 + 63321) + 0.1217(1641468 + 50852)$
$= 0.1016(133101) + 0.1217(1692320) = 219478$

$F_{2,2} \cdots F_{4,4}$
$= 0.1833(133101) + 0.2196(1692320) = 396031$

$F_{5,5} = 0.1833(133101) + 0.2195(1692320) = 395862$

$F_{6,6} = 0.1652(69780) = 11528$

FUEL COUPLINGS

$F_{7,7} = 0.1217(1641468) = 199767$

$F_{8,8} \cdots F_{10,10} = .2196(1641468) = 360466$

$F_{11,11} = 0.2195(1641468) = 360302$

RACK-FUEL COUPLINGS

$F_{1,7} = F_{7,1} = .1217(1641468 + 50852) = 205955$

$F_{2,8} = F_{8,2} = F_{3,9} = F_{9,3} = F_{4,10} = F_{10,4}$
$= 0.2196(1641468 + 50852) = 371633$

$F_{5,11} = F_{11,5} = 0.2195(1641468 + 50852) = 371464$

RACK- WALL COUPLINGS

$F_{1,15} = 0.1016(69780 + 66929) = 13890$

$F_{2,15} = F_{3,15} = F_{4,15} = F_{5,15}$
$= 0.1833(69780 + 66929) = 25059$

$F_{6,15} = 0.1652(69780 + 66929) = 22584$

HYDRODYNAMIC COUPLING FORCES FOR 110 CELL RACK IN NORTH-SOUTH DIRECTION

LENGTH = 201.3125", REGION 2

RACK COUPLING FORCES

$F_{1,1} = 0.1016(54730 + 63321) + .1217(1641468 + 50852)$
$= 0.1016(118051) + .1217(1692320) = 217949$

$F_{2,2} = F_{3,3} = F_{4,4}$
$= 0.1833(118051) + .2196(1692320) = \underline{393272}$

$F_{5,5} = 0.1833(118051) + .2195(1692320) = \underline{393103}$

$F_{6,6} = 0.1652(54730) = \underline{9041}$

FUEL COUPLING FORCES

$F_{7,7} = 0.1217(1641468) = 199767$

$F_{8,8} = F_{9,9} = F_{10,10} = .2196(1641468) = \underline{360466}$

$F_{11,11} = 0.2195(1641468) = \underline{360302}$

RACK-FUEL COUPLING FORCES

$F_{1,7} = F_{7,1} = 0.1217(1641468 + 50852) = 205955$

$F_{2,8} = F_{8,2} = F_{3,9} = F_{9,3} = F_{4,10} = F_{10,4}$

$= 0.2196(1641468 + 50852) = \underline{371634}$

$F_{5,11} = F_{11,5} = 0.2195(1641468 + 50852) = \underline{371464}$

RACK-WALL COUPLING FORCES

$F_{1,15} = 0.1016(54730 + 66929) = \underline{12361}$

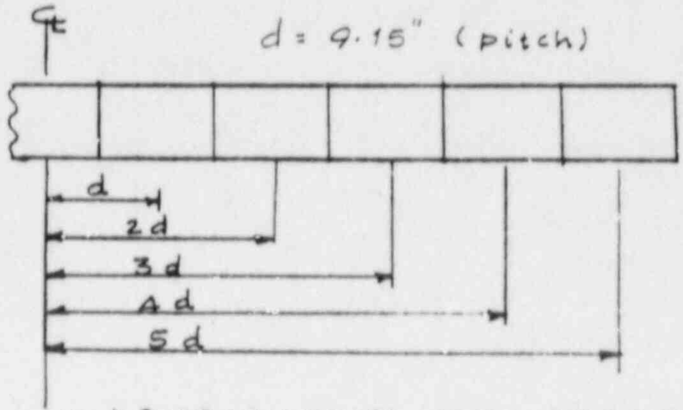
$F_{2,15} = F_{3,15} = F_{4,15} = F_{5,15}$
$= 0.1833(54730 + 66929) = 22300$

$F_{6,15} = 0.1652(54730 + 66929) = \underline{20098}$

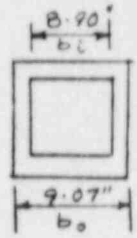
MOMENT OF INERTIA OF RACKS

PER STICK MODEL OF RACK, SHOWN IN FIGURE 6.1 OF SECTION 6, (SEISMIC), ELEMENTS 1 THRU 5, REPRESENT RACK ACTING AS A BEAM. FOR 110-CELL RACK, THE NORTH-SOUTH (E-W MOTION) DIRECTION IS 10-CELL LONG & EAST-WEST (N-S MOTION) RACK IS 11-CELL LONG.

CONSIDER ONE ROW OF BOXES, SYMMETRICAL ABOUT ℄.



LONGITUDINAL BOX SECTION
(SYMMETRICAL @ ℄)
11 - CELL LENGTH.



BOX - SECTION

$A = b_o^2 - b_i^2$
$= (9.07)^2 - (8.90)^2$
$= 3.055 \text{ in}^2$

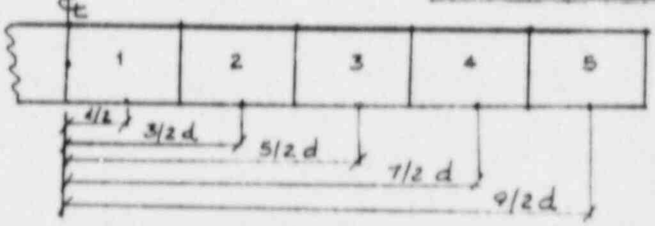
$I_{℄} = I_o + AD^2$ D: ℄ BOX TO ℄ BOX

$I_o = \dfrac{b_o^4 - b_i^4}{12} = 41.11 \text{ in}^4$ L: 11-CELLS

$I = 11\, I_o + 2\{A d^2 (1^2 + 2^2 + 3^2 + 4^2 + 5^2)\}$
$= 11(41.11) + 2\{3.055 \times 9.15^2 (55)\}$ $= 28587 \text{ in}^4/\text{row}$

FOR 110-CELL RACK, $I_{NS} = 10_{\text{ROW}} \times 28587$
$= \underline{285870 \text{ in}^4}$

CONSIDER, NS: <u>10 CELL LONG</u>



d: 9.15" PITCH
A: 3.055 in^2

10 - CELL LONGITUDINAL SECTION

$I = 10(41.11) + 2\{(3.055)(9.15)^2 [(1/2)^2 + (3/2)^2 + (5/2)^2 + (7/2)^2 + (9/2)^2]\} = 21512 \text{ in}^4/\text{row}$

$I_{EW} = 11 \text{ ROWS} \times 21512 \text{ in}^4 = \underline{\underline{236632 \text{ in}^4}}$

RACK SHEAR AREA

SHEAR AREA ASSUMED IN THE DIRECTION OF MOTION. i.e HALF METAL AREA
$A = 3.055\ IN^2$, EACH CELL

FOR 110-CELL RACK, SHEAR AREA (A_S):

$$A_S = \frac{1}{2}(3.055\ IN^2) \times (110\ CELLS) = \underline{168\ IN^2}$$

SUMMARY: RACK PROPERTIES

$I_{NS} = 285870\ IN^4$
(E-W MOTION)

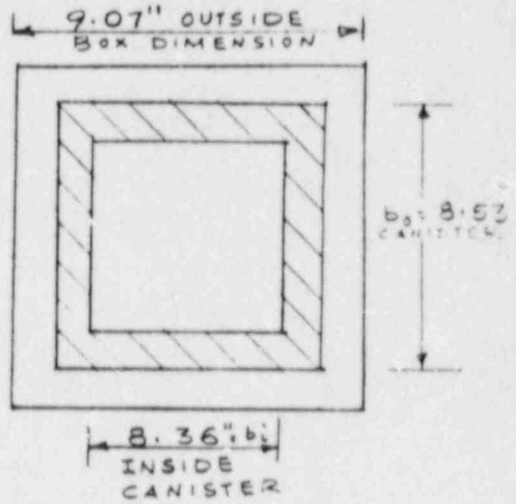
$I_{EW} = 236632\ IN^4$
(N-S MOTION)

$A_S = 168\ IN^2$

110-CELL RACK, REGION 2

MOMENT OF INERTIA (FUEL-CANISTER)

SHADED AREA REPRESENTS FUEL CANISTER



$$I_{o_F} = \frac{(8.53)^4 - (8.36)^4}{12}$$

$$= 34.13 \text{ IN}^4 / \text{ASSEMBLY}$$

FOR 110-CELL FUEL ASSEMBLY

$$I_{FUEL} = 110(34.13) \text{ IN}^4$$

$$= \underline{3754.3 \text{ IN}^4}$$

SHEAR AREA, CONSOLIDATED FUEL

$A_c = (8.53)^2 - (8.36)^2 = 2.87 \text{ IN}^2$ / FUEL CANISTER (SHADED AREA)

FUEL SHEAR AREA (A_{S_F}) = $1/2(A_c)$ PER CELL

110-CELL FUEL ASSEMBLY

$$A_{S_{FUEL}} = 110\left(\frac{2.87}{2}\right) = \underline{157.85 \text{ IN}^2}$$

SUMMARY: FUEL PROPERTIES

I_{FUEL} = 3754.3 IN^4

$A_{S_{FUEL}}$ = 157.85 IN^2

4

U.S. TOOL & DIE, INC

BY a.t. DATE 8/17/87 SUBJECT STICK MODEL OF RACK SECT. 6 SHEET 24 OF 52 PROJ. NO. 8709

CHKD. BY [illegible] DATE [illegible]

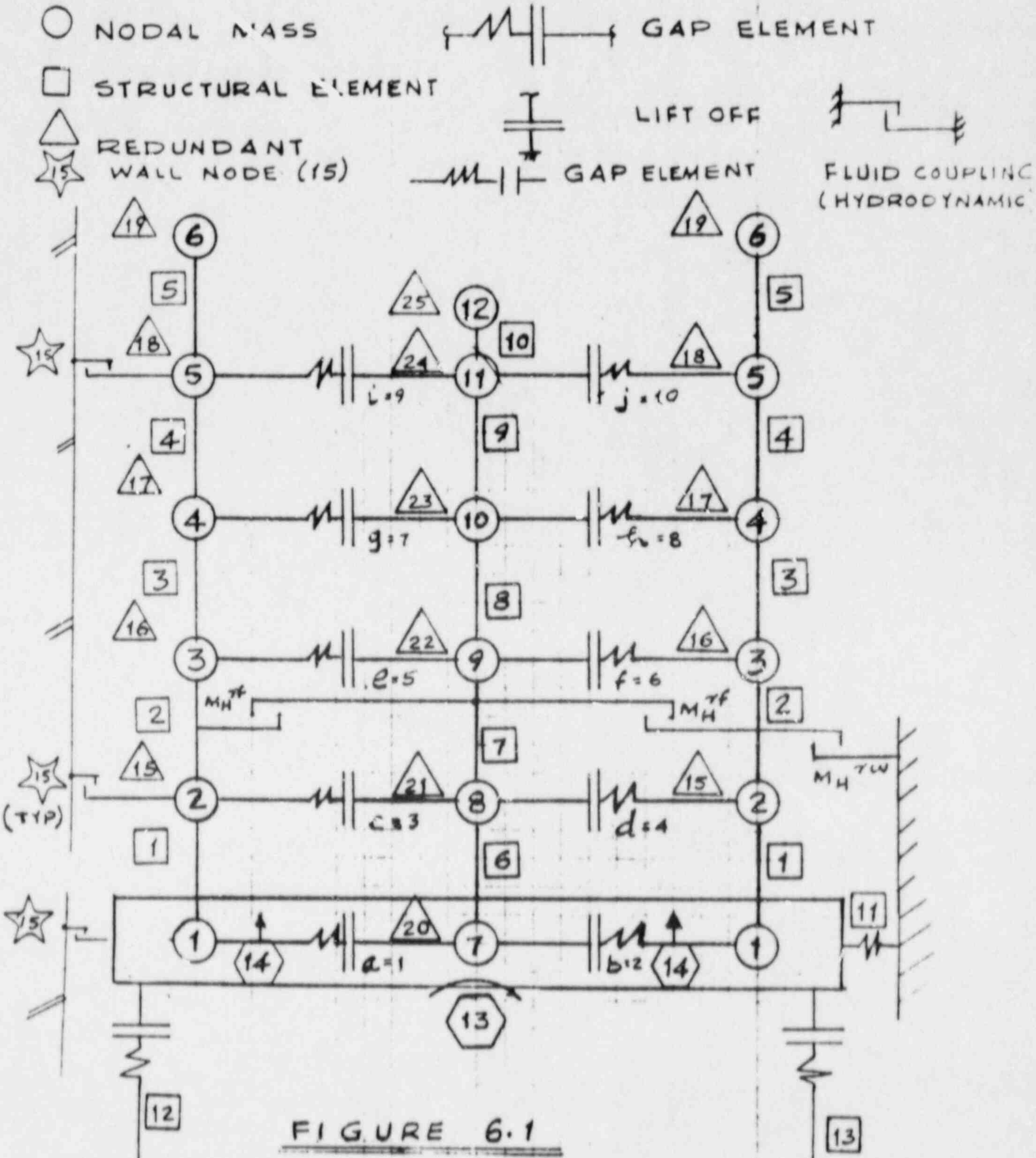
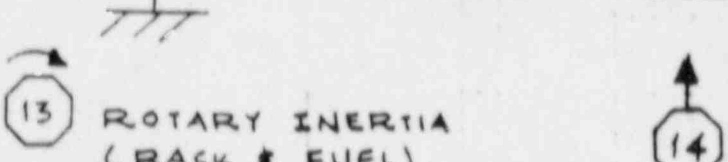


FIGURE 6.1

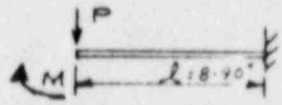


ROTARY INERTIA (RACK & FUEL)

VERTICAL MASS NODE (TOTAL DWT)

THE TOTAL FLEXIBILITY BETWEEN THE FUEL & THE RACK MASS IS REPRESENTED BY THE GAP SPRINGS. IN FIGURE 6.1, GAP SPRINGS ELEMENTS ARE IDENTIFIED BY SYMBOLS a THRU j, FROM LEFT TO RIGHT AND BETWEEN RACK MASS NODES 1 THRU 5 & FUEL MASS NODES 7 THRU 11.

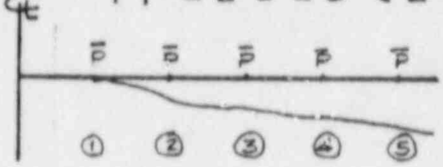
BENDING FLEXIBILITY



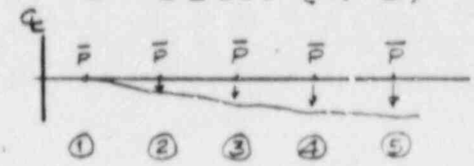
$$\frac{\Delta}{P} = \frac{l^3}{12EI}$$ (ENDS MOVING WITHOUT ROTATION)

CONSIDER 11 - CELL WIDE RACK. (E-W)
& 10 - CELL WIDE RACK (N-S)

11-CELLS (E-W)



10 - CELLS (N-S)



ASSUME ROW ① DOES NOT DEFLECT.
ROW ② ACTING ON 1 FLEXIBLE ELEMENT..... etc

ROW	# ACTING (ELEMENTS)	ROW	# ACTING (ELEMENTS)
1	0	1	0
2	1	2	1
3	2	3	2
4	3	4	3
5	4	5	4
	Σ = 10		Σ = 10

AVERAGE FACTOR $= \frac{10}{5} = 2.0\,\bar{P}$

EACH FLEXIBLE ELEMENT = 36.9" LONG = (b), t = 0.085"

$$I = \frac{bt^3}{12} = \frac{36.9 \times (0.085)^3}{12} = 0.001888 \text{ IN}^4$$

$$EI = (28E6)(.001888) = 5.288E4$$

$$\frac{\delta}{\bar{P}} = \frac{l^3}{12(EI)} = \frac{(8.90)^3}{12(5.288E4)} = 0.001111 \text{ IN/LB}$$

USING A FACTOR OF 2.0 FROM ABOVE

$$\frac{\delta}{\bar{P}} = 2.0(.001111) = 0.002222$$

GAP SPRING FLEXIBILITY:

EAST - WEST DIRECTION = 11 CELLS
NO. OF PIECES = (11 × 2) = 22 (PARALLEL)

$$\frac{\delta}{\bar{P}} = \frac{2.222E-3}{22} = 1.01003E-4$$

$$\bar{P} = \frac{P}{10}$$

$$\therefore \frac{\delta}{\bar{P}} = \frac{0.000101}{10} = 0.0000101$$

$$K_2 = \frac{1}{0.0000101} = 99007$$

USE $K_2 = 99000$

NORTH - SOUTH DIRECTION = 10 CELLS

NO. OF PIECES = 20 IN PARALLEL

$$\frac{\delta}{\bar{P}} = \frac{2.222E-3}{20} = 0.0001111$$

$$\bar{P} = \frac{P}{11}$$

$$\frac{\delta}{\bar{P}} = \frac{.0001111}{11} = 0.0000101$$

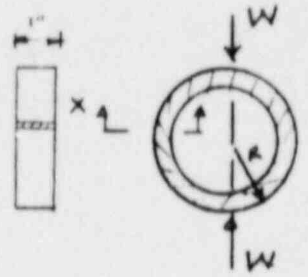
$K_2 = 99010$

USE $K_2 = 99000$

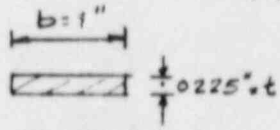
REF. #12 "FORMULAS FOR STRESS & STRAIN, FIFTH EDITION, ROARK & YOUNG TABLE 17, FORMULAS FOR CIRCULAR RINGS.

CONTACT STIFFNESS - FUEL TO RACK

DV = CHANGE IN VERTICAL DIAMETER OF FUEL ROD.



FUEL-ROD RING SECT.



SECTION X-X
(1" STRIP)

E = 13E6 (ZIRCALLOY) PSI
FUEL ROD = 0.374" Φ
L_{FUEL} = 168"
W = P/L = LB / LINEAL INCH
t = CLAD THICKNESS = 0.0225"
CONSIDER 1" STRIP

$$I = \frac{1 \times .0225^3}{12} = 9.49E\text{-}7 \ \frac{IN}{IN}$$

CASE 1

$$D_V = -0.149 \frac{WR^3}{EI}$$

$$\frac{1}{K_1} = \frac{\Delta_V}{P} = -\frac{0.149 R^3}{L \cdot E \cdot I} = \frac{-0.149 \left(\frac{.374}{2}\right)^3}{(168)(13E6 \ \text{PSI})(9.49E\text{-}7 \ \frac{IN^4}{IN})}$$

$$= -4.701E\text{-}7 \ IN/LB$$

$$K_{ASSEMBLY} = 2.1272E6 \ \frac{LB}{IN}$$

$$\frac{1}{K_R} = \frac{N}{K_1} \quad K_{TOTAL} = N(K_R) = N\left(\frac{K_1}{N}\right) \quad \therefore K_{TOTAL} = K_1$$

$$\therefore K_1 = \frac{2.1272E6 \ (110)}{5 \ \text{GAP SPRINGS}} = 4.6798E7$$

TOTAL SPRING STIFFNESS

$$\frac{1}{K} = \frac{1}{K_1} + \frac{1}{K_2}$$

	K_1	K_2	USE K (LB/IN)
NORTH-SOUTH	4.6798E7	99000	98790
EAST-WEST	4.6798E7	99000	98790
	(FUEL)	(GAP SPRING)	

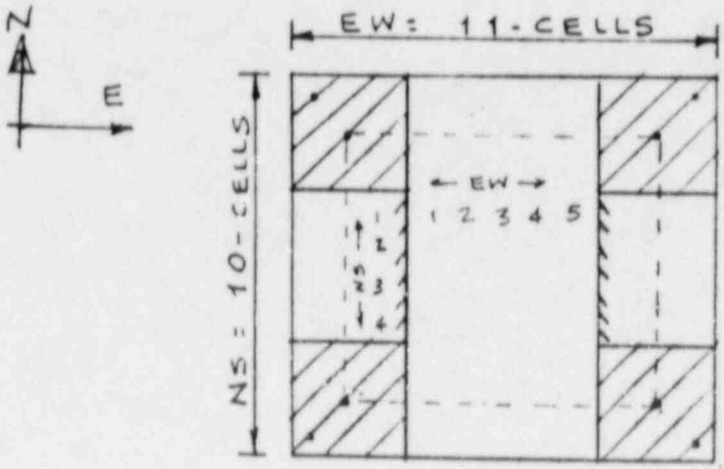
SUMMARY OF CONTACT SPRINGS (GAP)

NORTH-SOUTH K = 98790 LB/IN
EAST-WEST K = 98790 LB/IN

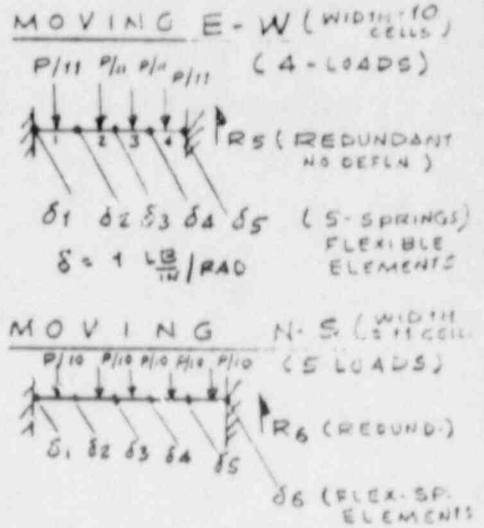
$$GAP: \left(\frac{8.90 - 8.53}{2}\right) = 0.185"$$

PER FIGURE 6.1, ELEMENT 11 IS A HORIZONTAL RESTRAINT BETWEEN THE RACK AND THE POOL WALL IN RACKOE COMPUTER, THE VALUE OF SPRING STIFFNESS IS ENTERED AS A 1x1 MATRIX. (i.e SINGLE VALUE)

ELEMENT 11 FLEXIBILITY (110-CELL)



PLAN VIEW
110-CELL



THE RACK IS MOVING IN NORTH-SOUTH DIRECTION. THE SHADED AREAS REPRESENT 4-PEDESTAL SUPPORTS. ASSUMING THE CELLS WHICH ARE IN LINE WITH PEDESTAL SUPPORTS DO NOT BEND, BUT THE 4 ROWS BETWEEN THE SUPPORTS ARE CONSIDERED TO DEFORM.

EACH CELL IS REPRESENTED AS AN EQUIVALENT SHEAR MEMBER. ONE END OF THE CELL WILL MOVE RELATIVE TO THE OTHER WITHOUT ROTATION. THE CELL FLEXIBILITY IS OBTAINED, (FOR $\delta = 1 \frac{LB\text{-}IN}{RAD}$ FLEXIBLE SPRING) USING FLEX3, FORTRAN COMPUTER PROGRAM. FLEX 3 CALCULATES FLEXIBILITY MATRIX FOR SYSTEMS HAVING SINLE EFFECT ELEMENTS AND ONE REDUNDANT (i.e THERE IS NO DEFLECTION AT THE SUPPORT, R5 or R6)

FLEX3MATRIX DUE TO UNIT LOAD PER DEFLN.
110 CELL RACK MOVING E-W DIRECTION
10-CELL WIDTH PITCH=9.15 IN.
REGION 2 FILE: FLX3110NI
07 16 88

LOADS ROD δ
4 1 5

1 1 1

1 0 1

0.5

1. 1. 1. 1. -1.

2 0 1

1.

0. 1. 1. 1. -1.

3 2 1

0. 0. 1. 1. -1.

4 2 1

0. 0. 0. 1. -1.

5 0 1

0.5

0. 0. 0. 0. -1.

```
FLEXMATRIX DUE TO UNIT LOAD PER DEFLN
1.0 CELL RACK MOVING E-W DIRECTION
10-CELL WIDTH PITCH=9.15 IN.
REGION 2 FILE FLXNS110
7/16/88
DATE(MON/DAY/YEAR).   7/19/88
TIME(HOUR.MIN.SEC).  10.54.50


NUMBER OF LOADS IS    4
NUMBER OF REDUNDANTS IS       1
NUMBER OF FLEXIBLE ELEMENTS IS    5

FLEXIBILITY METHOD USED

        .5000
  1.0  1.0  1.0  1.0 -1.0
        .5000
   .0  1.0  1.0  1.0 -1.0
   .0   .0  1.0  1.0 -1.0
   .0   .0   .0  1.0 -1.0
        .5000
   .0   .0   .0   .0 -1.0

FLEXIBILITY MATRIX
       .44        .31        .19        .06
       .31        .74        .56        .19
       .19        .56        .94        .31
       .06        .19        .31        .44
```

Σ 1.00 2.00 2.00 1.00

6.00 TOTAL SYSTEMS FLEXIBILITY

AVERAGE DEFLECTION = $\frac{6}{4 \text{ loads}}$ = 1.50

FLEX3, EW-MOTION

FLEX3MATRIX DUE TO UNIT LOAD/DEFLN
110 CELL RACK MOVING E-W DIRECTION
10-CELL WIDTH, PITCH=9.15 IN.
REGION 2 FILE: FLX3110EI
07 16 88

5 1 6

1 1 1

1 0 1

0.5

1. 1. 1. 1. 1. -1.

2 0 1

1.

0. 1. 1. 1. 1. -1.

3 2 1

0. 0. 1. 1. 1. -1.

4 2 1

0. 0. 0. 1. 1. -1.

5 2 1

0. 0. 0. 0. 1. -1.

6 0 1

0.5

0. 0. 0. 0. 0. -1.

FLEX3 MATRIX DUE TO UNIT LOAD/DEFLN
110 CELL RACK MOVING N-S DIRECTION
11-CELL WIDTH, PITCH=9.15 IN.
REGION 2 FILE: FLX110EW
7/16/80
DATE(MON/DAY/YEAR) 7/13/83
TIME(HOUR.MIN.SEC) 11.13. 0

NUMBER OF LOADS IS 5
NUMBER OF REDUNDANTS IS 1
NUMBER OF FLEXIBLE ELEMENTS IS 6

FLEXIBILITY METHOD USED

```
      .5000
 1.0  1.0  1.0  1.0  1.0 -1.0
     1.0000
  .0  1.0  1.0  1.0  1.0 -1.0
  .0   .0  1.0  1.0  1.0 -1.0
  .0   .0   .0  1.0  1.0 -1.0
  .0   .0   .0   .0  1.0 -1.0
      .5000
  .0   .0   .0   .0   .0 -1.0
```

FLEXIBILITY MATRIX

.45	.35	.25	.15	.05
.35	1.05	.75	.45	.15
.25	.75	1.25	.75	.25
.15	.45	.75	1.05	.35
.05	.15	.25	.35	.45
Σ 1.25	2.75	3.25	2.75	1.25

= 11.25 TOTAL SYSTEMS FLEXIBILITY

AVERAGE DEFLECTION = $\frac{11.25}{5}$ = 2.25

TOTAL FLEXIBILITY FOR THE SYSTEM IS OBTAINED BY ADDING VALUES FROM Pg. 162
NORTH SOUTH FLEXIBILITY
$= 11.25$ WHERE $\bar{P} = \frac{P}{11}$

AVERAGE DEFLECTION $= \frac{11.25}{5_{\text{LOADS}}} \left(\frac{P}{11_{\text{CELL WIDTH}}} \right) = 0.205\,P$

FOR UNIT FLEXIBILITIES.

THE FLEXIBILITY OF EACH PIECE IS

$$\frac{\delta}{P} = \frac{\ell^3}{12EI}$$

WHERE $\ell = 8.90''$ NOM. INSIDE CELL DIMEN
$b = 201.3125''$ (VERTICAL LENGTH OF BORES)
$h = 0.085''$ (CELL THICKNESS)

$$I = \frac{bh^3}{12} = \frac{201.3125(.085)^3}{12} = 0.0103026 \text{ IN}^4$$

$$\frac{\delta}{P} = \frac{(8.90)^3}{12(28E6)(.0103026)} = 2.0365\,E-4$$

FOR 110 CELL MOVING IN N-S DIRECTION, (WIDTH = 11 CELLS) THERE ARE $2 \times 10 = 20$ PIECES IN PARALLEL

$$\frac{\delta}{P} = \frac{2.0365E-4}{20} = 1.02\,E-5$$

SUBSTITUTING FACTOR OF 0.205 FROM ABOVE

$$\frac{\Delta}{P} = 0.205\,(1.02E-5) = 2.091E-6$$

$K_2 = \underline{4.7824E5}$ LB/IN. N-S

FOR 110-CELL MOVING IN EAST-WEST DIRECTION

TOTAL SYSTEMS FLEXIBILITY OBTAINED BY ADDING VALUES FROM PAGE # 15a

$= 6.00 \qquad \text{WHERE } \bar{P} = \frac{P}{10}$

$$\text{AVERAGE DEFLECTION} = \frac{6.00}{4_{\text{LOADS}}}\left(\frac{P}{10}\right) = 0.15P$$

FLEXIBILITY OF EACH PIECE IS

$$\frac{\delta}{P} = \frac{\ell^3}{12EI} = 2.0365E-4 \quad (\text{see pg. } 17)$$

FOR CELL MOVING IN EAST-WEST DIRECTION (WIDTH = 10 CELLS), THERE ARE 2×11 = 22 PIECES IN PARALLEL.

$$\frac{\delta}{P} = \frac{2.0365E-4}{22} = 9.2568E-6$$

SUBSTITUTING FACTOR 0.15

$$\frac{\Delta}{P} = 0.15\,(9.2568E-6) = 1.38852E-6$$

$$K_2 = 7.2019E5 \quad \frac{LB}{IN} \qquad \text{EAST-WEST}$$

ATTACHMENT #3
ST-HL-AE-2764
PAGE 36 OF 54

PROPERTIES FOR ELEMENTS 12 & 13

ELEMENTS 12 & 13 ARE VERTICAL SPRINGS ENTERED IN RACKOE COMPUTER PROGRAM AS 2×2 MATRIX. THE SPRINGS REPRESENT:

a) LOCAL DEFORMATION OF BOX

$K_1 = 5.56E7$ (REF. Pg. 4-25 of 64)

$\Delta = \frac{2(1-\mu^2)}{REf_s(1.5)}$, $R = \sqrt{\frac{4b^2}{\pi}}$

b) DEFORMATION OF PEDESTAL

$K_2 = 9.90E7$ (REF Pg 4-26 of 64)

$K = \frac{AE}{L}$

c) CONCRETE DEFORMATION

$K_3 = 6.206E7$ (REF. Pg 4-26 of 64)

$\Delta_{AVE} = \frac{1}{2RE}\left[\frac{2}{\pi} + \frac{4}{\pi^2}\right](1-\mu^2)$

d) OVERALL RACK FLEXIBILITY

DUE TO LOCAL BENDING OF CELL WALLS EACH CELL CAN MOVE RELATIVE TO THE ADJACENT CELL. THE BORAFLEX IS QUITE INCOMPRESSIBLE, SO THE ROTATION IS ABOUT EITHER THE TOP OR THE BOTTOM OF BORAFLEX. LOCAL FLEXIBILITIES OF THE CELL WALLS NEAR THE WELDS ARE REPRESENTED BY SPRINGS.
THE WALL FLEXIBILITIES ARE FOUND USING PROGRAM DEFL (APPENDIX A) IT SOLVES EQUATION NO. 145, PAGE 143 "CONCENTRATED LOAD ON A SIMPLY SUPPORTED RECTANGULAR PLATE"
REF. THEORY OF PLATES & SHELLS BY S. TIMOSHENKO & S. KRIEGER.
THE STIFFNESS OF A JOINT BETWEEN ADJACENT CELLS IS EQUIVALENT TO A TORSION SPRING.
$K_t = 1.424E9 \frac{LB \cdot IN}{RAD}$ / BOX. REFER Pg. 4-30a of 64 SEISMIC ANALYSIS

3

U.S. TOOL & DIE, INC

BY a.t. DATE 10/29/87 SUBJECT RACK SECT. 4 SHEET ~~30a~~ 36 OF ~~6~~ 52

CHKD. BY [illegible] DATE 12/11/87 FLEXIBILITY PROJ. NO. 8709

NOTE: PAGE 30 IS REPLACED BY THE FOLLOWING CALCS.

BASED ON THE NUMERICAL RESULTS DESCRIBED IN REF # 18 (FINITE ELEMENT EVALUATION OF SPENT FUEL RACK STIFFNES: STIFFNESS VALUE FOR MIDDLE WELDS = 58480 LB/INCH. THE COMPUTED VALUE K_2 = 5499 LB/IN, AS SHOWN ON P. 2[illegible] THIS IS VERY CONSERVATIVE. K_1, K_2, K_3 & K_4 REPRESENT STIFFNESS VALUES FOR TOP, MIDDLE, INTERMEDIATE BOTTOM & BOTTOM FUSION WELDS RESPECTI[VELY]

BASED ON K_2 = 58480 LB/IN, MIDDLE WELD, REF. # 18 PROPORTION K_1, K_3 & K_4, AS SHOWN BELOW:

$K_1 = \frac{2006}{5499}(58480)\ \frac{LB}{IN} = 21333$ LB/IN TOP WELD.

$K_2 = \frac{5499}{5499}(58480)\ \frac{LB}{IN} = 58480$ LB/IN. MIDDLE WELD

$K_3 = \frac{5852}{5499}(58480)\ \frac{LB}{IN} = 62234$ LB/IN. INTERMEDIATE BOTTOM WELD

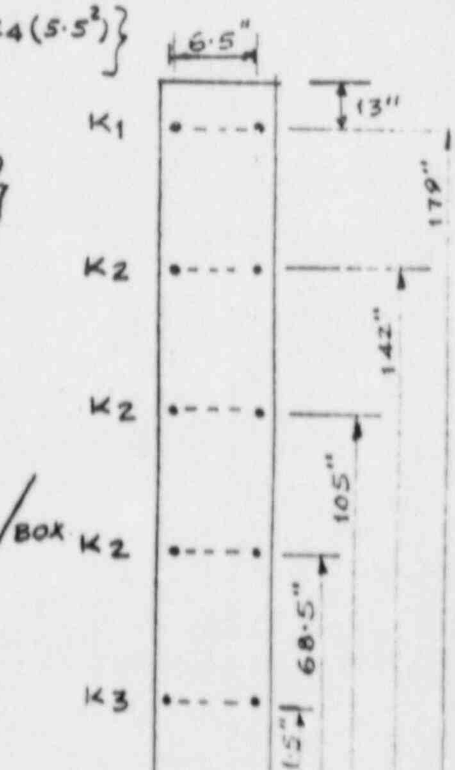
$K_4 = \frac{11498}{5499}(58480)\ \frac{LB}{IN} = 122277$ LB/IN. BOTTOM WELD.

$\Sigma M_{A-A} = 0$

$$M = \theta\left\{K_1(179^2) + K_2(142^2 + 105^2 + 68.5^2) + K_3(31.5^2) + K_4(5.5^2)\right\}$$

$$\frac{M}{\theta} = \left\{21333(32041) + 58480(35881.25) + 62234(992.25) + 122277(30.25)\right\}$$

$$\frac{M}{\theta} = 2.847E9 \text{ LB-IN/RAD/SURFACE}$$



THE VALUE FOR TWO SURFACES ON EACH BOX IS HALF THIS VALUE.

$$\frac{M}{\theta} = \frac{1}{2}\left[2.847E9\right] = 1.424E9\ \frac{\text{LB-IN}}{\text{RAD}}/\text{BOX}$$

$$K_t = 1.424E9\ \frac{\text{LB-IN}}{\text{RAD}}/\text{BOX}$$

THIS IS SLIGHTLY LOWER, BECAUSE ACTUALLY THERE ARE 8-LEVELS OF FUSION WELDS (SEE MECHANICAL REPORT, 8709-00-0120).

THE BASE OF THE RACK IS ASSUMED TO DEFORM, DUE TO THE MOMENT EXERTED BY EACH CELL.
THIS IS MODELLED AS SHOWN ON NEXT PAGE.
A FLEXIBILITY MATRIX IS CALCULATED FOR THIS SYSTEM, WHERE TERM $D_{I,J}$ IS

THE ANGULAR DEFLECTION AT CELL I DUE TO A UNIT MOMENT AT CELL J.

THE EXPANDED FLEXIBILITY MATRIX IS SHOWN ON PAGE 23, ALSO SHOWN IS FLEXIBILITY MATRIX.

THE TERMS OF THE MATRIX ARE ADDED TOGETHER AND DIVIDED BY THE NO. OF CELLS.

THE AVERAGE ANGULAR DEFLECTION DUE TO ALL MOMENTS M_I, IS SHOWN ON THE COMPUTER PRINT OUT FOR FLEX1, pg. 24 OF THIS CALCULATION (θ_T)

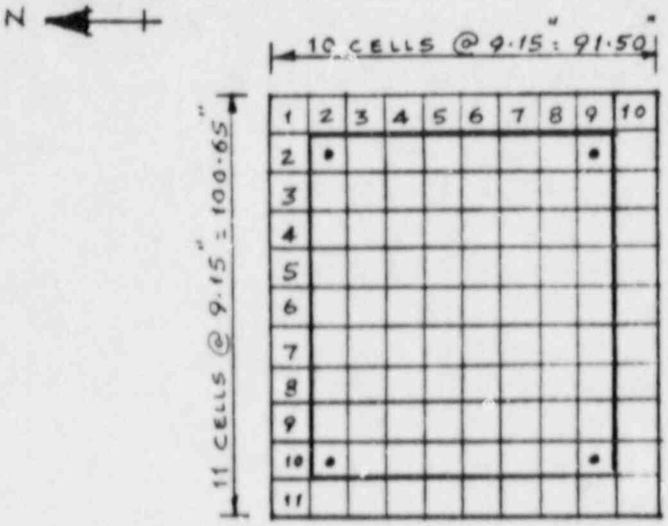
FOR $K_T = 1$, THE FOLLOWING VALUES ARE FOUND

	LENGTH		
N-S	10 - CELLS	$\Sigma D_{I,J} = \theta_T = 7.14$	(pg 24)
E-W	11 - CELLS	$\Sigma D_{I,J} = \theta_T = 7.91$	REF. SEISMIC ANALYSIS, SECTION 4, SHEET 31 OF 64 (132, CELL RACK)

THIS FLEXIBILITY IS CONVERTED TO AN EQUIVALENT PEDESTAL STIFFNESS FOR ELEMENTS 12 & 13 AS DESCRIBED IN SECTION 4, pgs. 32, 33 of 64.

$$K = \frac{2N^2 K_T}{\theta_T \cdot L^2}$$

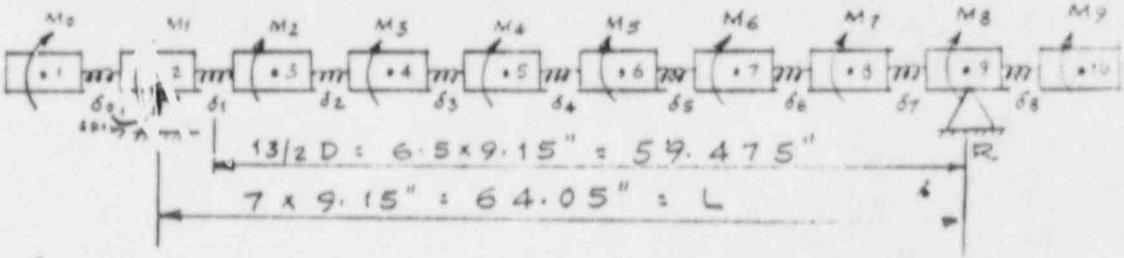
THIS VALUE IS FOR UNIT CELL WITH DETAILED CALCULATIONS ON pg. 25.
FLEX 1 INPUT ON pg. 21 & 22.
FLEX 1 OUTPUT, pg. 23, 24.



PITCH = D = 9.15"

PLAN 110-RACK

CONSIDER 10-CELL IN N-S DIRECTION



δ_0 IS A VERY FLEXIBLE ELEMENT REPRESENTING SIMPLE SUPPORT

M : DISTRIBUTED MOMENTS (EACH CELL)
δ : FLEXIBLE ELEMENTS (TORSIONAL SPRINGS)
R : REDUNDANT
N = NO OF CELLS = 10

USING FLEX1, FIND $\Sigma D_{I,J} = \theta_T$ FOR $\delta_i = 1$

I.E AVERAGE ANGULAR DEFLECTION DUE TO MOMENTS $M_1, M_2 \cdots M_9$.

```
FLEX1 DUE TO DISTRIBUTED MOMENTS (?)
110 CELL RACK, REGION 2 STP,
10 CELL LONG IN N-S DIRECTION
PITCH=9.15 INCHES          FILE: FLX1101I
JULY 18 88

      LOADS  [illegible]  FLEX. ELEM
      9    1    9
      1    1    1
      1    0    1
δ0    1000.0000
      1.   1.   1.   1.   1.   1.   1.   1.   1.   -64.05
δ1    2    0    1
      1.
      0.   1.   1.   1.   1.   1.   1.   1.   1.   -59.48
δ2    3    2    1
      0.   0.   1.   1    1.   1.   1.   1.   1.   -50.33
δ3    4    2    1
      0.   0.   0.   1.   1.   1.   1.   1.   1.   -41.18
δ4    5    2    1
      0.   0.   0.   0.   1.   1.   1.   1.   1.   -32.03
δ5    6    2    1
      0.   0.   0.   0.   0.   1.   1.   1.   1.   -22.88
δ6    7    2    1
      0.   0.   0.   0.   0.   0.   1.   1.   1.   -13.73
δ7    8    2    1
      0.   0.   0.   0.   0.   0.   0.   1.   1.   - 4.58
δ8    9    2    1
      0.   0.   0.   0.   0.   0.   0.   0.   1.     0.00
```

ATTACHMENT 3
ST-HL-AE-2764
PAGE 41 OF 54

FLEX: DUE TO DISTRIBUTED MOMENTS (OT)
110 CELL RACK REGION 2 STP
10 CELL LONG IN N-S DIRECTION
PITCH 9.15 INCHES FILE FLX11101
7/10/88
DATE(MON,DAY/YEAR) 7/10/88
TIME(HOUR MIN SEC) 17.36.28

NUMBER OF LOADS IS 9
NUMBER OF REDUNDANTS IS 1
NUMBER OF FLEXIBLE ELEMENTS IS 9

FLEXIBILITY METHOD USED

1000.0000

1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-64.1

1.0000

.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-59.5
.0	.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-50.3
.0	.0	.0	1.0	1.0	1.0	1.0	1.0	1.0	-41.2
.0	.0	.0	.0	1.0	1.0	1.0	1.0	1.0	-32.0
.0	.0	.0	.0	.0	1.0	1.0	1.0	1.0	-22.9
.0	.0	.0	.0	.0	.0	1.0	1.0	1.0	-13.7
.0	.0	.0	.0	.0	.0	.0	1.0	1.0	-4.6
.0	.0	.0	.0	.0	.0	.0	.0	1.0	.0

FLEXIBILITY MATRIX

2.32	1.39	.61	-.04	-.53	-.89	-1.10	-1.10	-1.10
1.39	1.46	.68	.04	-.46	-.82	-1.03	-1.11	-1.11
.61	.68	.89	.25	-.25	-.61	-.82	-.89	-.89
-.04	.04	.25	.61	.11	-.25	-.46	-.54	-.54
-.53	-.46	-.25	.11	.61	.25	.03	-.04	-.04
-.89	-.82	-.61	-.25	.25	.89	.68	.61	.61
-1.10	-1.03	-.82	-.46	.03	.68	1.46	1.39	1.39
-1.10	-1.11	-.89	-.54	-.04	.61	1.39	2.32	2.32
-1.10	-1.11	-.89	-.54	-.04	.61	1.39	2.32	2.32

EXPANDED FLEXIBILITY MATRIX

2.32	2.32	1.39	.61	-.04	-.53	-.89	-1.10	-1.10	-1.10
2.32	2.32	1.39	.61	-.04	-.53	-.89	-1.10	-1.10	-1.10
1.39	1.39	1.46	.68	.04	-.46	-.82	-1.03	-1.11	-1.11
.61	.61	.68	.89	.25	-.25	-.61	-.82	-.89	-.89
-.04	-.04	.04	.25	.61	.11	-.25	-.46	-.54	-.54
-.53	-.53	-.46	-.25	.11	.61	.25	.03	.04	-.04
-.89	-.89	-.82	-.61	-.25	.25	.89	.68	.61	.61
-1.10	-1.10	-1.03	-.82	-.46	.03	.68	1.46	[illegible]	1.39
-1.10	-1.10	-1.11	-.89	-.54	-.04	.61	1.39	[illegible]	2.32
-1.10	-1.10	-1.11	-.89	-.54	-.04	.61	1.39	2.32	2.32

SUMS

2.72 1.71 .43 -.43 .06 -.06 -.43 .43 1.71
2.71

TOTAL = 7.14 $= \sum D_{I,J} = \Theta_T$

CALCULATE EQUIVALENT PEDESTAL STIFFNESS:

110-CELL RACK EAST WEST

N = 11 CELL LENGTH
θ_T = 7.91 (SEISMIC, SECT 4, pg 33a of 64
L = 73.20" " " "
WIDTH = 10 CELLS IN N-S DIRECTION
K_t = 1.424E9 $\frac{LB\text{-}IN}{RAD}$ SEE pg 19 THIS CALC.

$$K_4 = \frac{2N^2 K_T (\text{WIDTH})}{\theta_T \cdot L^2} = \frac{2 \times (11)^2 \times (1.424E9)(10)}{(7.91)(73.20)^2} = \underline{8.1307E7}$$

110-CELL RACK NORTH SOUTH

N : 10 CELL LENGTH
θ_T : 7.14 pg. 24
L : 64.05" pg. 21
WIDTH : 11 CELLS IN E-W DIRECTION
K_t : 1.424E9 $\frac{LB\text{-}IN}{RAD.}$

$$K_4 = \frac{2 \times (10)^2 (1.424E9)(11)}{(7.14)(64.05)^2} = \underline{1.0695E8}$$

COMBINE THESE FLEXIBILITIES WITH OTHER FLEXIBILITIES SHOWN ON pg. 19.

i.e. DEFORMATION OF BOX, PEDESTAL & CONCRETE.

ADDING THE SERIES SPRINGS

$$\frac{1}{K} = \frac{1}{K_1} + \frac{1}{K_2} + \frac{1}{K_3} + \frac{1}{K_4}$$

RACK
110 - EAST-WEST

$$\frac{1}{K} = \frac{1}{5.56E7} + \frac{1}{9.\ \ \ {}^{-}7} + \frac{1}{6.206E7} + \frac{1}{8.1307E7}$$

$$\frac{1}{K} = 5.6499E{-8} \qquad \therefore K = 17.6994E6$$

110 - NORTH-SOUTH

$$\frac{1}{K} = \frac{1}{5.56E7} + \frac{1}{9.90E7} + \frac{1}{6.206E7} + \frac{1}{10.695E7}$$

$$\frac{1}{K} = 5.355E{-8} \qquad \therefore K = 18.674E6$$

THE ELEMENT FLEXIBILITY USED:

$$K = \begin{bmatrix} K_{11} & K_{12} \\ K_{21} & K_{22} \end{bmatrix} = \begin{bmatrix} K_{22}\ell^2 & K_{22}\ell \\ K_{22}\ell & K_{22} \end{bmatrix}$$

WHERE K_{22} IS THE VALUE FOUND ABOVE & ℓ = MOMENT ARM

ELEMENTS 12 & 13 - 110 CELL RACK

EAST WEST

$$K_{22} = 17.699E6$$

$$\ell = \frac{8\text{ CELLS} \times 9.15}{2} = 36.60''$$

$$K_{EW} = \begin{bmatrix} 23.709E9 & 6.4780E8 \\ 6.4780E8 & 17.699E6 \end{bmatrix}$$

NORTH-SOUTH

$$K_{22} = 18.674E6, \quad \ell = \frac{64.05}{2} = 32.025''$$

$$K_{NS} = \begin{bmatrix} 19.152E9 & 5.980E8 \\ 5.980E8 & 18.674E6 \end{bmatrix}$$

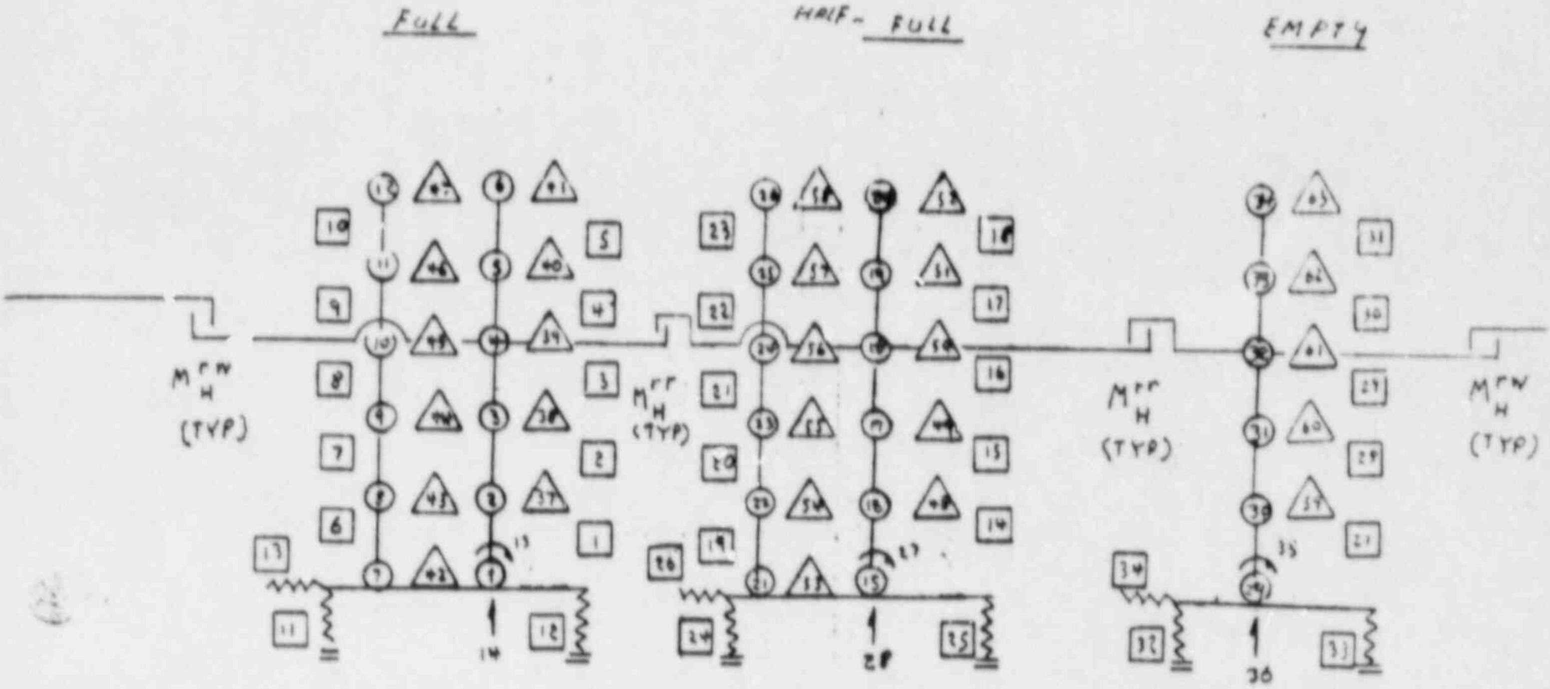
ACTION 1A:

During the July 21, 1988 meeting with the Staff, UST&D presented a detailed discussion of the hydrodynamic mass matrix model. The hydrodynamic mass matrix model for STP is based on a 1-inch rack-to-rack gap. The base model is a 3-rack model (132-cell racks) with the first rack filled with ~~consolidated~~ standard fuel, the second rack half filled with ~~consolidated~~ standard fuel, and the third rack empty. Using a 0.2 coefficient of friction (i.e., sliding occurs), the maximum relative motion between racks is 0.128 inches. Using a 0.8 coefficient of friction (i.e., no sliding occurs), the maximum relative motion between racks is 0.29 inches.

Since the hydrodynamic mass matrix model is based on a 1-inch rack-to-rack gap, there is no physical interaction of the racks. As the racks are moved closer together the hydrodynamic coupling forces increase resulting in smaller relative motion between racks. UST&D has performed a similar analysis for another rack order using a 0.25 inch gap. The relative motion between the racks was .03 inches. As in the STP analysis, there was no physical interaction between the racks. The relative motion between racks as discussed above for a 1-inch gap and a 0.25-inch gap demonstrates the increasing strength of the hydrodynamic coupling as the rack-to-rack gap is decreased, preventing physical interaction of the racks.

In addition, UST&D performed the hydrodynamic mass matrix model with a 1-inch rack-to-rack gap for a 110-cell rack using a 0.2 coefficient of friction. The maximum relative motion between racks was 0.106 inches.

MULTI RACK CONNECTIVITY



NOTE: NOT SHOWN ARE THE

1) FUEL-RACK GAP SPRINGS

2) FUEL-RACK FLUID COUPLINGS

M_H^{tw}

EAST REF. 1 SECT 4 SHT 1:

$M_H^{tw} = 1.603E5$ LB

WEST

$M_H^{tw} = .7131E05$ LB

$M_{rr} = \frac{\rho h 36}{125} = 2.389E04\left(\frac{100.65}{1}\right)$

$= 2.4E6$

$M_1^f = 29684$ $M_{rr} = 2.4E6$

$M_2^r = 75325$ $M_H^{rf} = 40371$

$M_1^r = 79616$

HEIGHT FRACTION FACTOR

$F_{11} = .123(M_H^{tw} + M_2^r + M^{rr}) + .1075(M_H^{rf} + M_1^f)$

RACK 1

$F_{11} = .123(.7131E5 + \overbrace{75325 + 2.4E6}^{2.547E6})$

$+ .1075(\overbrace{40371 + 29684}^{7.E4}) = 3.208E5$

$F_{2,2} = 0.1645(2.547E6) + .1865(7.E4) = 4.33E5$

$F_{33} .. F_{44} = .185(2.547E6) + .210(7.E4) = 4.56E5$

$F_{55} = .185(2.547E6) + .286(7E4) = 4.912E5$

$F_{6,6} = .1575(2.547E6) + .086(7.E4) = 4.012E5$

HALF FULL

RACK 2

$F_{ii} = \alpha(M_2^{F} + 2M^{HF}) + \frac{B}{2}(M_H^{HF} + M_1^{F})$

$F_{15,15} = .123(4.8753E6) + \frac{.1075}{2}(7.E4)$

$= 6.0493E5$

$F_{16,16} = .1645(4.8753E6) + \frac{.1865}{2}(7E4) = 8.105E5$

$F_{17,17} \cdot\cdot F_{18,18} = .185(4.8753E6) + \frac{.210}{2}(7E4) = 9.115E5$

$F_{19,19} = .185(4.8753E6) + \frac{.286}{2}(7E4)$

$= 9.142E5$

$F_{20,20} = .1575(2.541E6) + \frac{.086}{2}(7E4) = 7.729E5$

RACK 3

INCLUDE ENTRAINED WATER IN RACK MASS NODES

RACK TO RACK COUPLINGS

$F_{i,j} = F_{j,i} = \alpha[M_H^{HF}]$

$F_{15,1} = F_{1,15} = .123(2.4E6) = 2.95E5$

$F_{16,2} = F_{2,16} = .1645(2.4E6) = 3.95E5$

$F_{17,3} = F_{3,17} = .185(2.4E6) = 4.44E5$

$F_{18,4}$ $F_{4,18}$ $=$

$F_{19,5}$ $F_{5,19}$ $= 4.44E5$

$F_{20,6}$ $F_{6,20}$ $= 3.78E5$

$F_{24,15} = F_{15,24} = 2.95E5$

$F_{30,16} = F_{16,30} = 3.95E5$

$F_{31,17} = F_{31,17} = 4.44E5$

$F_{32,18} = F_{18,32} = 4.44E5$

$F_{33,19} = F_{19,33} = 4.44E5$

$F_{34,20} = F_{20,34} = 3.78E5$

RACK TO WALL COUPLING FOR CENTER RACK

$\alpha\,(M_1^{*})$

$F_{15,37} = .123\ (7.9616E4) = 9.793E3$

$F_{16,37} = .1645\ (\ \ "\ \ ) = 1.31E4$

$F_{17,37} = .185\ (\ \ "\ \ ) = 1.47E4$

$F_{18,37} = .185\ (\ \ "\ \ ) = 1.47E4$

$F_{19,37} = .185\ (\ \ "\ \ ) = 1.47E4$

$F_{20,37} = .1575\ (\ \ "\ \ ) = 1.25E4$

RACK TO WALL COUPLING FOR SIDE RACKS

( ) EAST WAL

$\alpha\,(M_1^{*} + M^{*W})$ — WEST WAL

1.603E5 (2.95E4

$F_{1,37} = F_{24,37} = .123\ (7.9616 + .7131E5) = \underline{1.86E5}$

(3.95E

$F_{2,37} = F_{30,37} = .1645\ (\ \ "\ \ ) = 2.48E$

(4.44E4

$F_{3,37} = F_{31,37} = .185\ (\ \ "\ \ ) = \underline{2.7926E4}$

(4.44E4

$F_{4,37} = F_{32,37} = .185\ (\ \ "\ \ ) = \underline{2.7926E4}$

(4.44E4

$F_{5,37} = F_{33,37} = .185\ (\ \ "\ \ ) = \underline{2.7926E4}$

(3.78E4

$F_{6,37} = F_{34,37} = .1575\ (\ \ "\ \ ) = \underline{2.35E4}$

THE EMPTY RACK REQUIRES;

1. ELIMINATION OF FUEL MASSES, ADD ENTRAINED WATER TO MASSES

2. ELIMINATION OF GAP SPRINGS

3. REMOVE FLUID COUPLINGS BETWEEN FUEL AND RACK

1. MASSES

ENTRAINED WATER $= (8.90)^2 (201) .0361 \times 132 =$

$= 75868$ LB

$M_{29} = 5125 + .123 (75868) = 14457.$

$M_{30} = 4211 + .1645 (\quad " \quad) = 16691.$

$M_{31} = 4692 + .185 (\quad " \quad) = 18728.$

$M_{32} = 4692 + .185 (\quad " \quad) = 18728.$

$M_{33} = 4684 + .185 (\quad " \quad) = 18720.$

$M_{34} = 3700 + .1575 (\quad " \quad) = 15649.$

$M_{13} = 1/12 \, L^2 M_{14} = \frac{1}{12} (100.65)^2 \cdot 102973 = 8.69E7$

$M_{14} = \sum_{i=1}^{6} M_i = 102973$

SWT $= 27104 - 4048 = 23056$

VERTICAL STATIC DEFLECTION $= \dfrac{-23056}{2(1.877E7)} = -.0006$

HALF LOAD RACK MASS

$$M_{11} = 143308 + \frac{75868}{2} = 181242$$

$$M_{13} = \frac{181242}{12} (100.65)^2 = 1.53E8$$

HYDRODYNAMIC FORCES ON WALL EVALUATION WITH (4" RACK TO WALL GAP)

ACTION 2A, 3A:

UST&D will supply Bechtel with the hydrodynamic forces on the rack for the reduced rack to wall distance (4 inches).

UST&D will provide the rationale or calculation describing the impact on the seismic analysis due to the rack-to-wall reduction (now 4 inches to North wall).

RESPONSE:

THE HYDRODYNAMIC LOAD ON THE RACK WALL WAS DETERMINED FOR THE NORTH SOUTH DIRECTION AND PARTICULARLY FOR THE NORTH WALL (4" GAP). THE LOAD VARIED VERTICALLY WITH THE MAXIMUM OCCURING TOWARD THE TOP RACK MASS POINT

RACK NODE	LOAD
1	1.57 E4 LB
2	2.899 E4 LB
3	3.044 E4 LB
4	3.192 E4 LB
5	3.34 E4 LB
6	3.14 E4 LB

IN THE STRESS EVALUATION THE TOTAL LOAD WAS CONSIDERED TO BE 6 X 3.34E4 = 2.004E5 LB

THIS LOAD ACTS ON AN AREA = 12 X 201.313 X 9.15 = 22104 IN²

ADDITIONAL TERMS ARE NECESSARY IN THE HYDRODYNAMIC MASS MATRIX TO SEPARATELY TRACK THE VALUE OF THE HYDRODYNAMIC FORCES AT THE RACK WALL FACING THE POOL WALL (4" GAP)

NORTH WALL

M_H^{*W} (COLUMN MATRIX)

ROW	COLUMN	(HEIGHT WEIGHTED FRAC) $-\alpha\, M_H^{*W}$
1	38	-(.1016) 509584 = -51774
2	38	-(.1833) 509584 = -93406
3	38	-(.1833) 509584 = -93406
4	38	-(.1833) 509584 = -93406
5	38	-(.1833) 509584 = -93406
6	38	-(.1652) 509584 = -84183

$M_H^{*W} + M_1^{*}$

1	39	51774 + (.1016) 80315 = 59934
2	39	93406 + (.1833) 80315 = 108128
3	39	93406 + (.1833) 80315 = 108128
4	39	93406 + (.1833) 80315 = 108128
5	39	93406 + (.1833) 80315 = 108128
6	39	84183 + (.1652) 80315 = 97451

132-CELL, NORTH-SOUTH DIRECTION, CONSOLIDATED FUEL. (REF. FILE: NRC132NIWF) NS-SSE LOADS, NON-SLIDING CASE

FLUID FORCES ON RACK WALL

MAXIMUM FLUID FORCES ON RACK WALL (NODES 1 THRU NODE 6)

$$= (\overset{①}{15.4} + \overset{②}{28.99} + \overset{③}{30.44} + \overset{④}{31.92} + \overset{⑤}{33.40} + \overset{⑥}{31.43}) = 171.58^{K}$$

CONSIDER $(\underset{\text{NODES}}{6} \times \underline{33.40}) = 200.4$ KIPS (CONSERVATIVE)

AREA OF RACK RESISTING FLUID FORCES: (NS = 12 CELLS)

$$(\underset{\text{CELLS}}{12} \times \underset{t}{.085''}) \quad 201.3125'' = 205.34 \ IN^2/FACE$$

$$\text{AXIAL STRESS} = \frac{200.4 \text{ KIPS}}{205.34 \text{ IN}^2} = 0.98 \text{ KSI}$$ (NEGLIGIBLE)

THE FLUID FORCES ON THE RACK WALLS (0.98 KSI) IS INSIGNIFICANT.

THE MAXIMUM "SSE" STRESSES IN CELL WALLS, COMPUTED IN MECHANICAL REPORT 8709-00-0120, SECTION 4.0, pg. 42c of 69,

T_{TOTAL} = 3506 PSI < 31200 PSI ALLOW.; WILL NOT AFFECT THE OVERALL RESULTS DUE TO FLUID STRESSES.

CONCLUSION

THE EFFECT OF ADDITIONAL FLUID FORCES ON RACK WALLS ARE INSIGNIFICANT, CONSIDERING 4"-GAP BETWEEN RACK & POOL WALLS.

ATTACHMENT # 4

ANNOTATED REVISION TO PAGE 6-2
OF THE HIGH DENSITY SPENT
FUEL RACK SAFETY ANALYSIS REPORT

L4/NRC/bx

ATTACHMENT # 4
ST-HL-AE-2764
PAGE 1 OF 1

New Fuel Area Bridge Crane from carrying heavy loads within 15 feet of the FHB spent pool boundary. Therefore, the spent fuel cask drop accident analysis (Reference 1, Section 15.7.5) and South Texas Project heavy loads analysis (Reference 4) excluded the Cask Handling Crane and New Fuel Area Bridge Crane from further consideration with respect to possible heavy loads which could drop or fall into the spent fuel pool.

6.1.2 Description of Spent Fuel Racks

A description of the spent fuel racks, including their design and fabrication is provided in Section 3.0.

6.1.2.1 Fuel Handling

The storage of additional spent fuel assemblies in the spent fuel pool will not affect the analysis and consequences of the design basis fuel handling accidents as presented in the South Texas Project Units 1 and 2 FSAR Sections 15.7.4 and 15.7.5 or the NRC Safety Evaluation Report, Section 15.7. The spent fuel racks are designed to withstand the design basis fuel handling accident. The resulting criticality and radiological consequences of a postulated fuel assembly drop are addressed in Sections 4.3 and 7.6.1 respectively.

6.2 SEISMIC AND IMPACT LOADS

The objective of the seismic analysis of the spent fuel racks is to determine the structural responses resulting from the simultaneous application of three orthogonal seismic excitations. The method of analysis employed is the time history method.

does not affect the original seismic analysis of the FHB. Accordingly, the original design basis

Seismic floor response spectra for the spent fuel pool floor have been developed using the methods described in Subsections 3.7.1 and 3.7.2 of the South Texas Project FSAR (Reference 1). The parameters of the original lumped mass model of the Fuel Handling Building were reviewed to verify that ~~adjusted to reflect~~ the increased mass corresponding to the new high density spent fuel storage racks. ~~The resulting~~ floor response spectra ~~are~~ were used as shown in Figures 6-6 through 6-11. These spectra were then used to generate statistically independent time history excitations at the pool floor, ~~one~~ for each of the two ~~three orthogonal~~ horizontal directions. Since the spent fuel racks have no connection with the pool walls or with each other, the pool floor time histories are used as input to the dynamic analysis of the racks, as described in Subsection 6.5.2.2.1. Fluid coupling is also considered as described with the exception of the temporary dry storage of new fuel in Region 1 racks.

Deflection or movements of racks under earthquake loading is limited by design such that the nuclear parameters outlined in Section 4.0 are not exceeded. Impact loads have been considered as discussed in Subsection 6.6.4.

For the vertical direction the equivalent static method is used for the seismic analysis of the racks.

ATTACHMENT # 5

ANNOTATED REVISIONS TO SECTION 3.7
OF THE FINAL SAFETY ANALYSIS REPORT

L4/NRC/bx

Table 3.7-8

<u>Cases Where the Confirmatory-Basis Spectra are Used for the Justification of completed Seismic Analysis, Designs and/or Qualifications</u>

1. Seismic qualification of a battery rack	The confirmatory-basis spectra at El. 35 ft. of the MEAB were used for the seismic qualification of an existing design of a Class 1E, 125 VDC battery rack (Battery No. NCX-1200).
2. Verification of th DGB Foundation Mat Design	The design-basis seismic analysis of the Diesel Generator Building (DGB) in the E-W direction was originally based on fixed-base mathematical models excited with the free-field ground motion amplified by a factor of 1.4 to account for SSI (See Table 3.7-2). This fixed-base analysis was a rudimentary approximation for the SSI, and resulted in E-W acceleration responses that are very conservative with respect to both the design-basis two-step FEM solution which was performed only along the N-S direction, and the confirmatory-basis solutions along the E-W and N-S directions. Subsequently, the zero-period accelerations (ZPA's) of the conservative design-basis analysis along the E-W direction were reduced by 50% in order to reconcile the existing design of the foundation mat for the DGB. The reduced value adopted for the E-W ZPA is justified by (1) being of the same order of magnitude as the N-S ZPA determined from the design-basis analysis by two-step FEM, and (2) being at least 20% higher than the ZPA obtained from the E-W confirmatory-basis spectra. Therefore, the confirmatory-basis analyses were used only as part of te justification for the lower E-W ZPA used in the reconciliation of the DGB foundation mat design, and have not been used in any other way for structural design or seismic qualification in the DGB.
3. Seismic qualification of the MEAB 480V Motor Control Centers (MCCs)	The confirmatory-basis spectra at El. 60 ft. of the MEAB are being used for the seismic qualification of an existing design of the Class 1E, 480V MCC (3E171EMCE1C1, 1C2, 1C4).
4. Verification of of the concrete floorslab of the spent fuel pool.	The confirmatory-basis spectra at El. 22 ft. of the FHB are used to reconcile the high transverse shear load induced in the floor slab by the high-density spent fuel storage racks for the heaviest consolidated storage case only.

56

STP RMS

3.7-52

Amendment 56

ATTACHMENT # 5
ST-HL-AE-2764
PAGE 1 OF 1

ATTACHMENT # 6

ANNOTATED REVISION TO THE EVALUATION
OF NO SIGNIFICANT HAZARDS CONSIDERATIONS
CONTAINED IN REFERENCE 1

L4/NRC/bx

ATTACHMENT #6
ST-HL-AE-2764
PAGE 1 OF 1

In addition, the thermal-hydraulic analysis has shown that fuel cladding integrity is maintained. The maximum fuel cladding temperature, assuming the peak pool bulk water temperature associated with the Case D pool loading, would remain below 205°F (reference U.S. Tool & Die, Inc. Thermal Hydraulic Report No. 8709-00-0098).

The dose consequences of a postulated loss of SFP cooling have previously been evaluated as described in FSAR Section 9.1.3.3.4. As a result of the increased inventory using the high density spent fuel racks, the offsite doses as documented in FSAR Table 9.1-6 have increased slightly (LPZ thyroid dose increased from 0.47 rem to 0.54 rem). However, they are still significantly below the 10CFR100 limits.

The consequences of (3) a seismic event, have been evaluated and are discussed in the referenced report. The new racks are designed and fabricated to meet the requirements of applicable portions of NRC Regulatory Guides and industry standards. The seismic floor response spectra for the spent fuel pool have been developed using the methods described in STP FSAR Sections 3.7.1 and 3.7.2. The parameters of the original lumped mass model of the Fuel Handling Building (FHB) were ~~adjusted to reflect~~ the increased mass corresponding to the new high density spent fuel storage racks. ~~The resulting~~ floor response spectra were then used to generate statistically independent time history excitations, ~~one for each of the three orthogonal directions~~. Since the spent fuel racks have no connection with the pool walls or with each other, the pool floor time histories are used as input to the dynamic analysis of the racks. Fluid coupling is also considered as described in the referenced Safety Analysis Report.

reviewed to verify that

The interaction between the fuel assemblies and the spent fuel racks has been considered, particularly gap effects. The resulting impact loads are of small magnitudes so there is no structural damage to the fuel assemblies.

The floor loading from the new racks filled with spent fuel assemblies does not exceed the structural capacity of the FHB.

The seismic analysis was performed on an individual 132-cell Region 2 rack (the most limiting). In addition, the seismic analysis is considered applicable to each of the racks and was performed such that the analysis is applicable regardless of the installed location in the pool and independent of the total number of racks in the SFP.

Therefore, the consequences of a seismic event will not increase from previously evaluated events.

The consequences of (4) a spent fuel cask drop have been evaluated and are discussed in the referenced report. In accordance with 10CFR71, the spent fuel shipping cask is designed to sustain a free-fall in air of approximately 30 ft. onto an unyielding surface followed by a specified puncture, fire, and immersion in water with the release of no more than a specified small quantity of radioactivity. The design of the spent fuel

does not affect the original seismic analyses of the FHB. Accordingly the original design basis

at the pool floor for each of the two horizontal directions. For the vertical direction the equivalent static method is used for the seismic analysis of the racks.

ATTACHMENT # 7

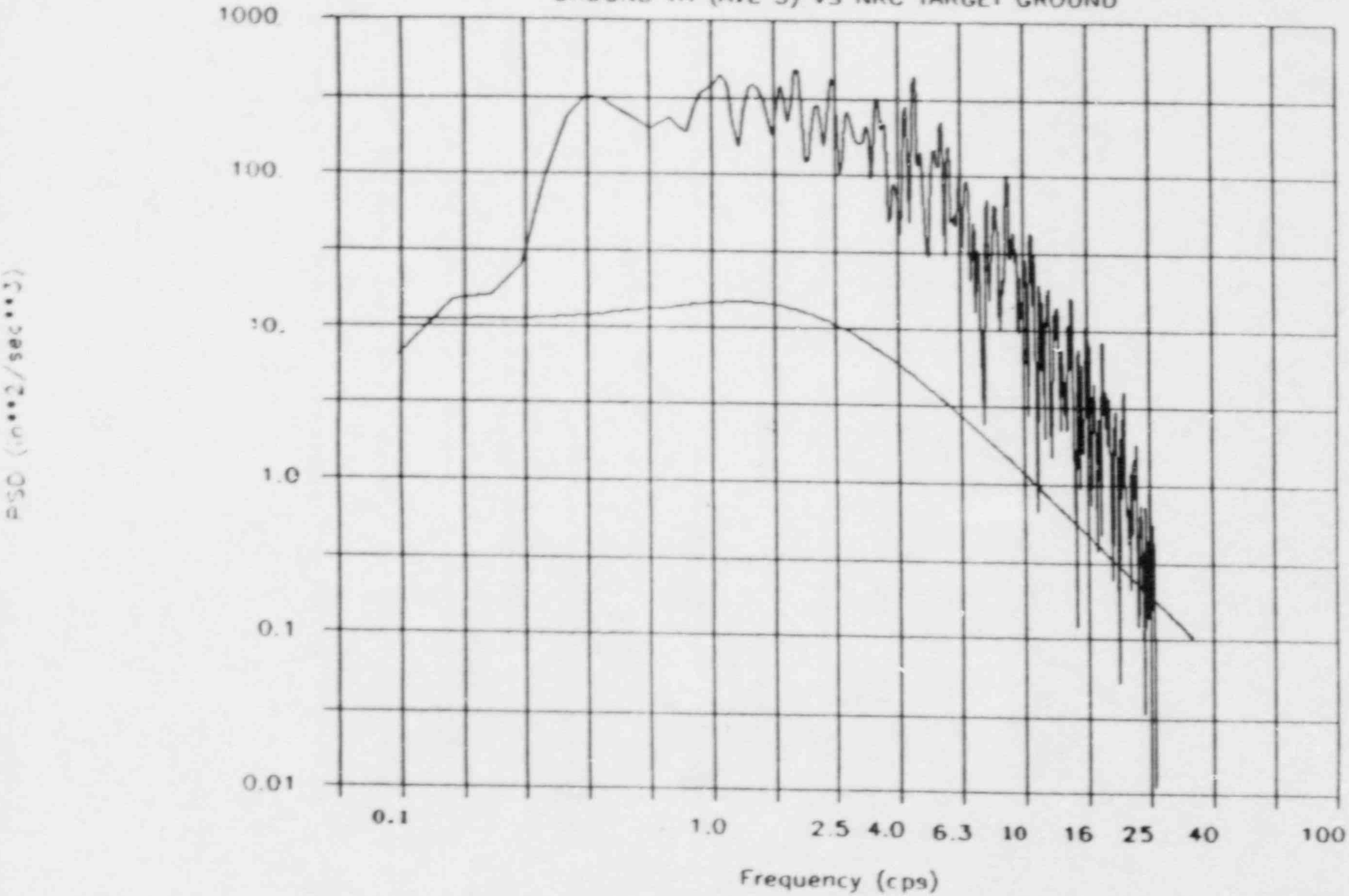
FIGURES 1 THROUGH 9 FOR
QUESTION 5 (ATTACHMENT # 1)

L4/NRC/bx

ATTACHMENT #7
ST-HL-AE-2764
PAGE 1 OF 9

PSD COMPARISON: SSE-HORIZ (NS & EW)

GROUND TH (AVE 3) VS NRC TARGET GROUND

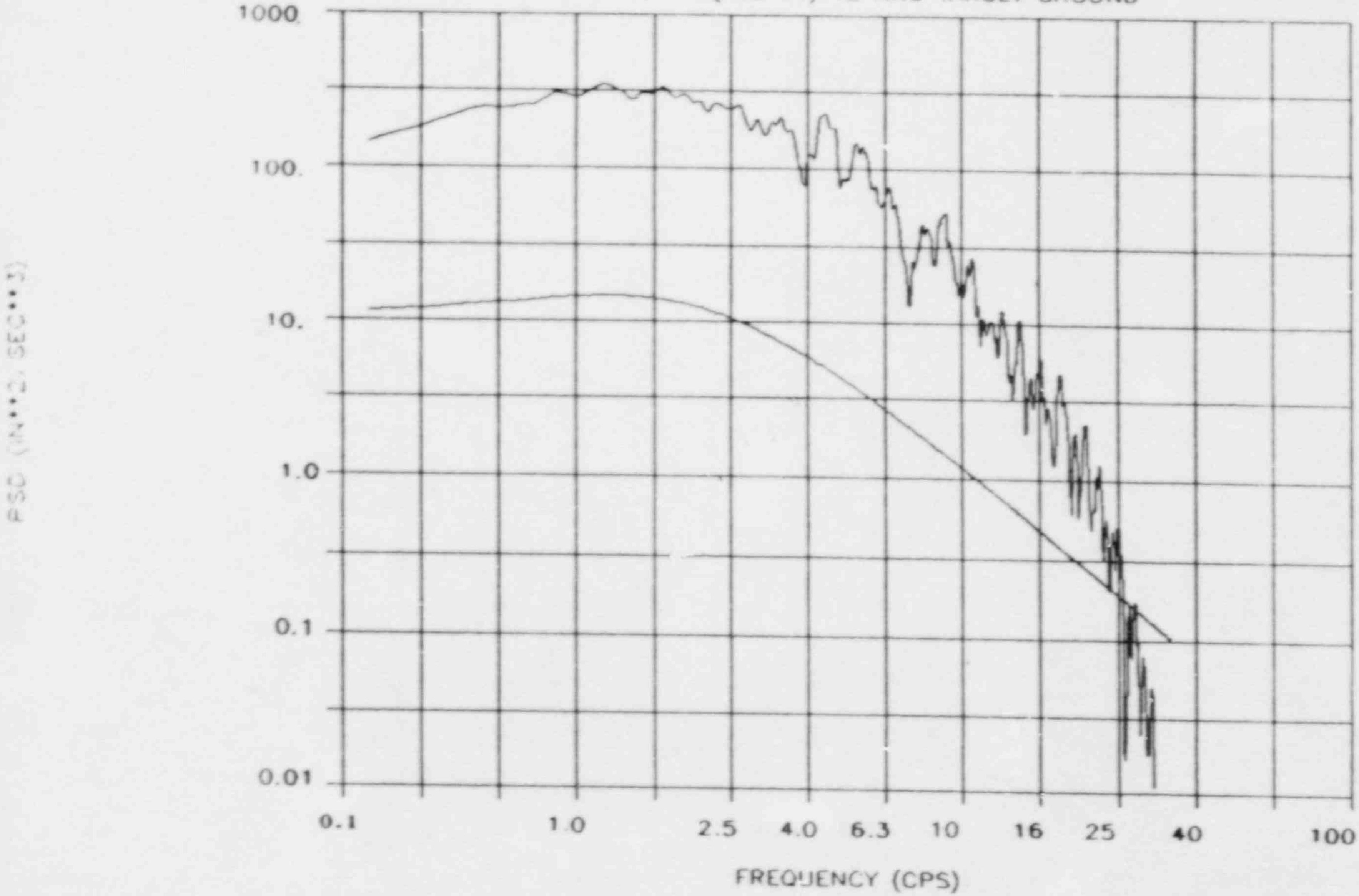


Question 5
Figure 1

ATTACHMENT #7
ST-HL-AE-2764
PAGE 2 OF 9

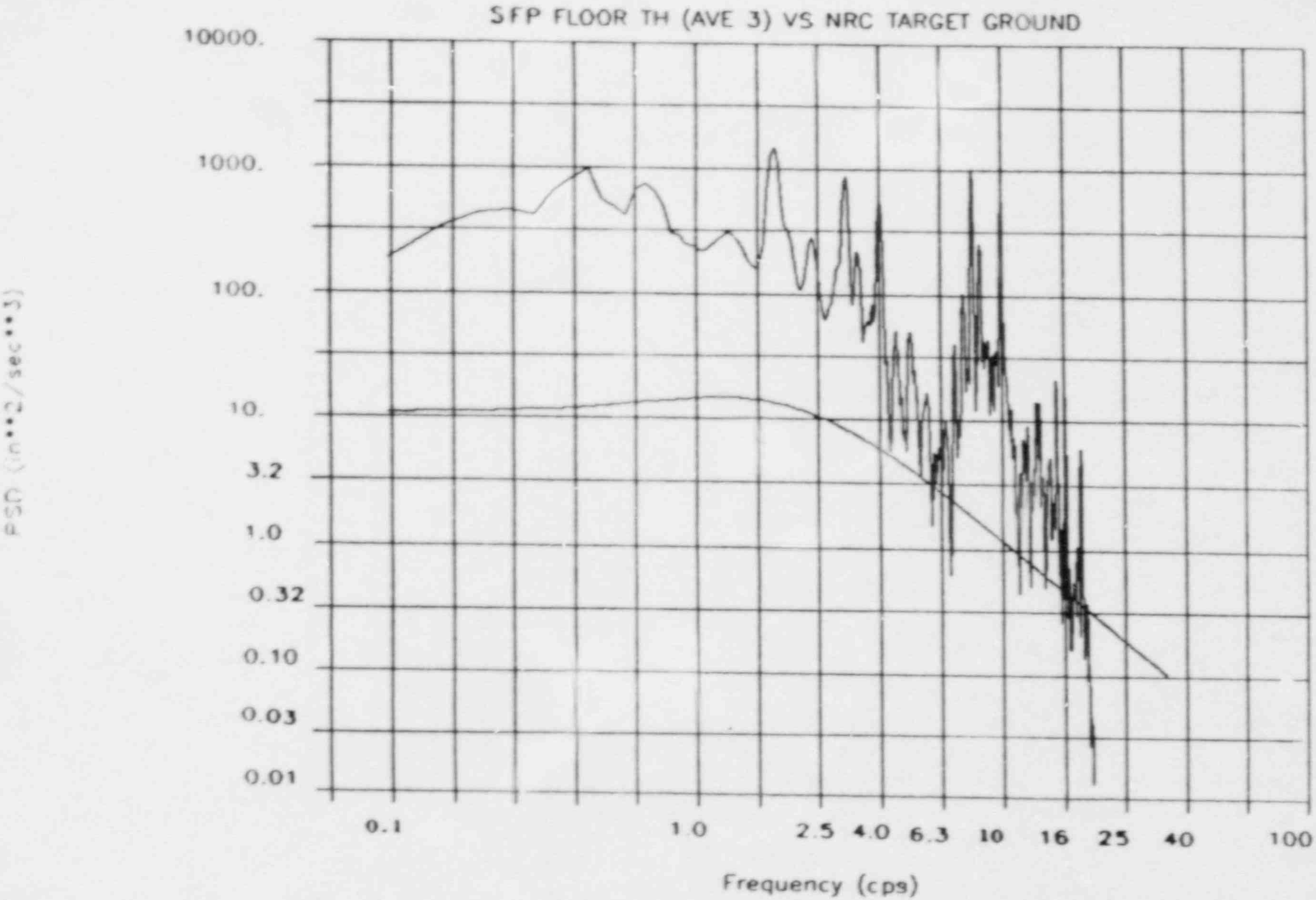
PSD COMPARISON : SSE-HORIZ (NS & EW)

GROUND TH (AVE 11) VS NRC TARGET GROUND



Question 5
Figure 2

PSD COMPARISON: N-S SSE

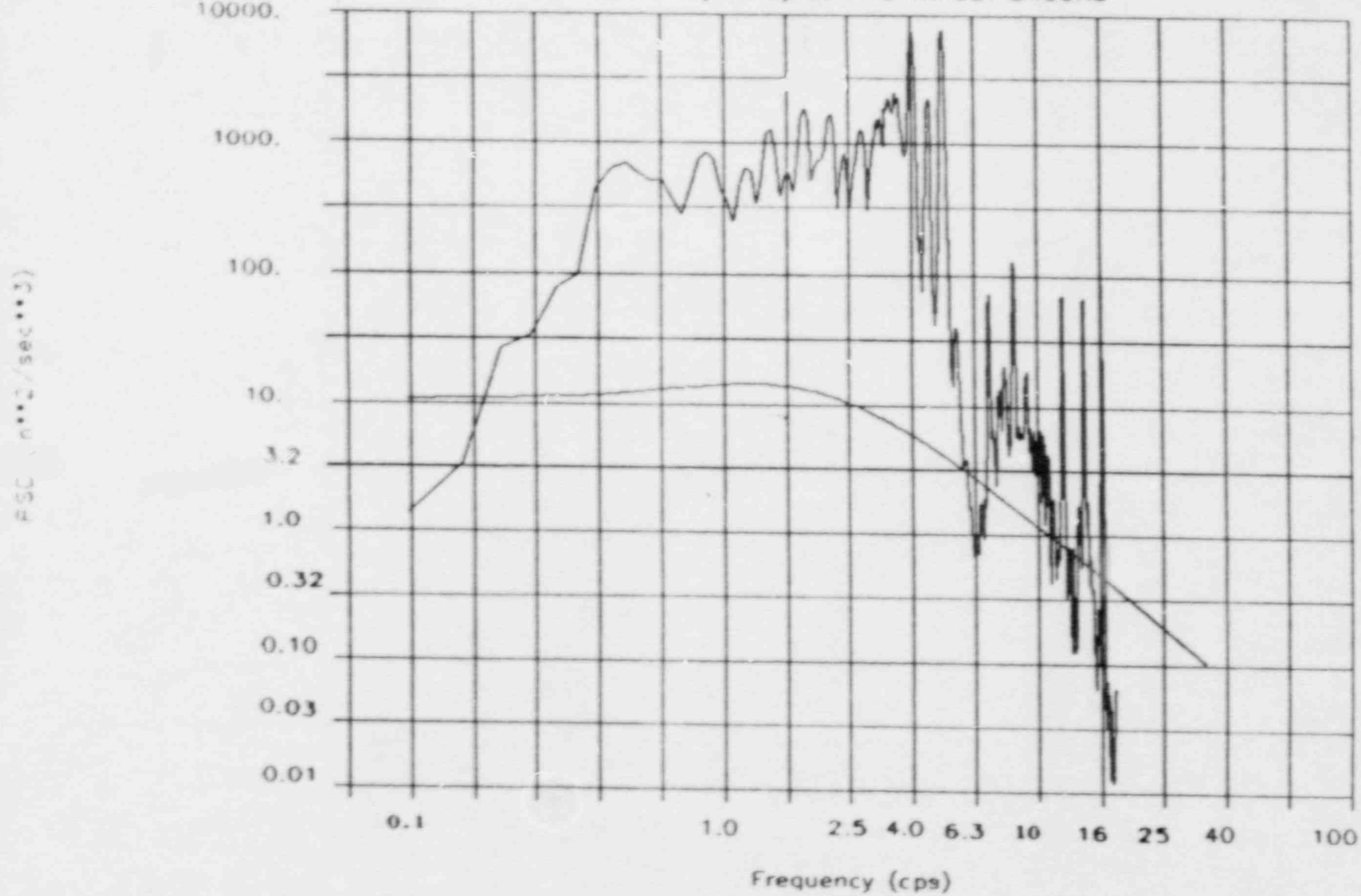


Question 5
Figure 3

ATTACHMENT #7
ST-HL-AE-2764
PAGE 4 OF 9

PSD COMPARISON : E-W SSE

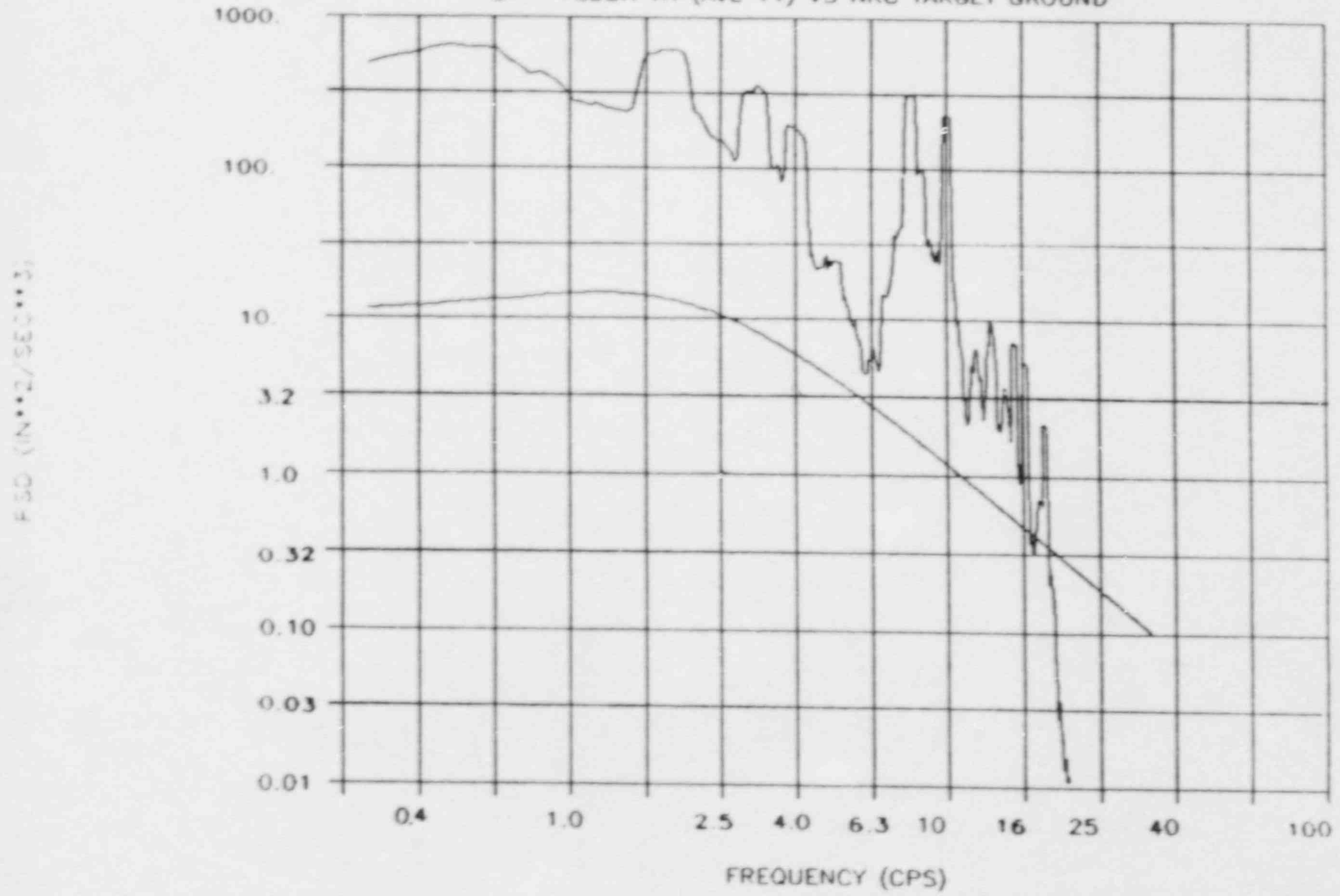
SFP FLOOR TH (AVE 3) VS NRC TARGET GROUND



Question 5
Figure 4

PSD COMPARISON: N-S SSE

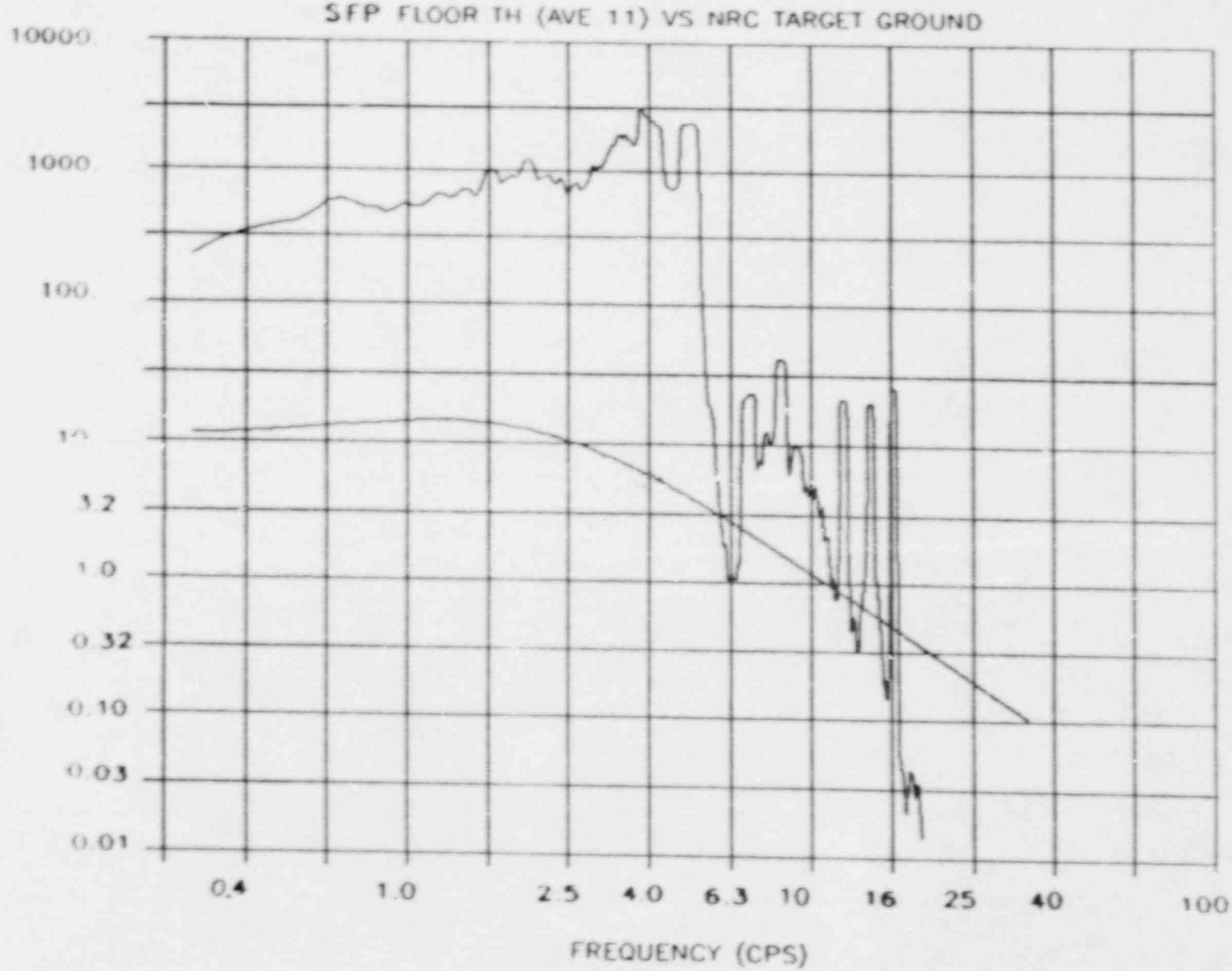
SFP FLOOR TH (AVE 11) VS NRC TARGET GROUND



Question 5
Figure 5

ATTACHMENT # 7
ST-HL-AE-2764
PAGE 5 OF 9

PSD COMPARISON : E-W SSE



Question 5
Figure 6

ATTACHMENT 7
ST-HL-AE-2764
PAGE 4 OF 9

P.93

STP, Seismic Reanalysis of the FHB, CC-9061

Originator: M. [illegible] 8/2/88

Checker: SCWu 8/3/88

ATTACHMENT #7
ST-HL-AE-2764
PAGE 7 OF 9

Question 5
Figure 7

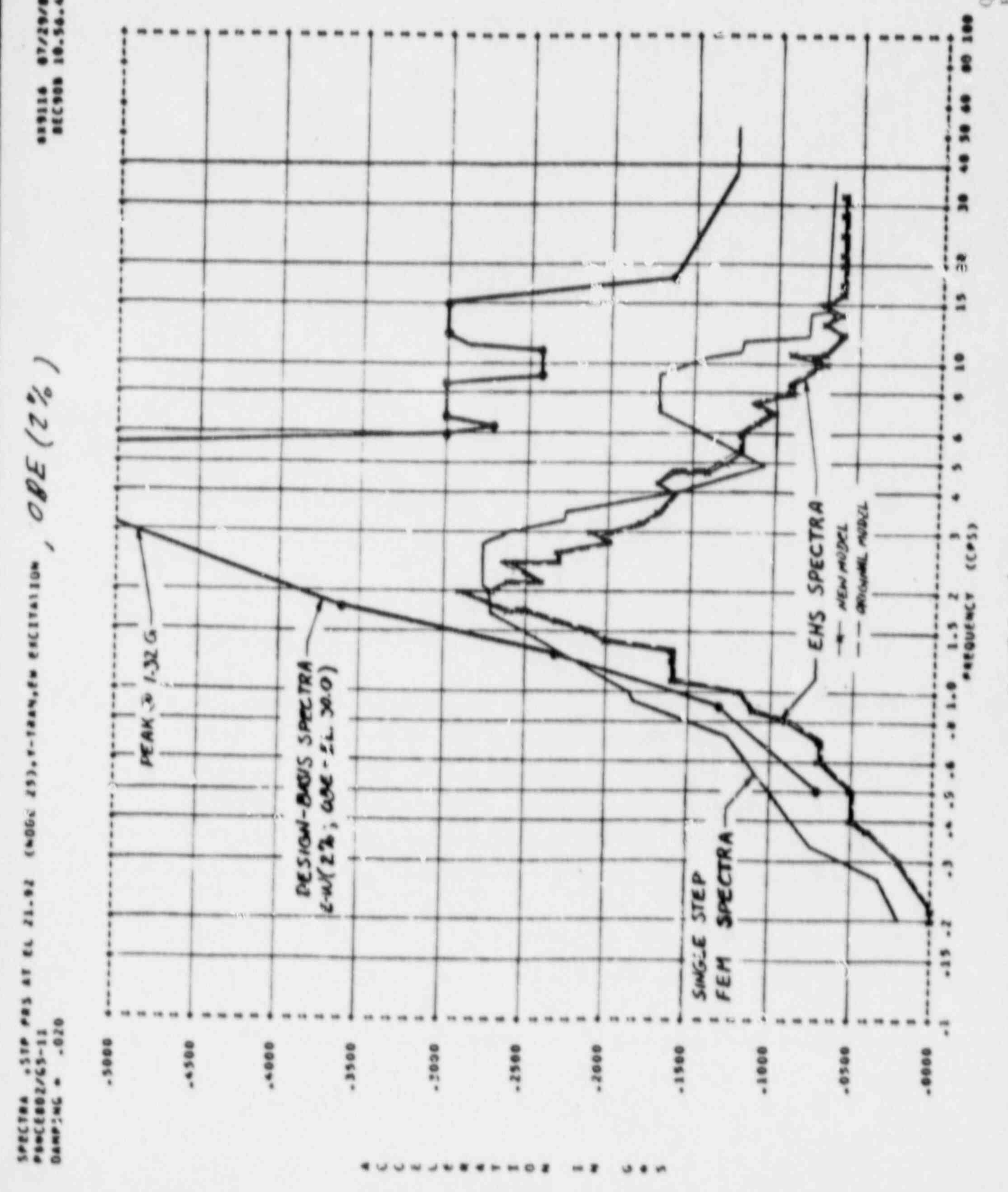


FIGURE 1

STP, SEISMIC REANALYSIS OF THE FHB CC-9061 REV.0

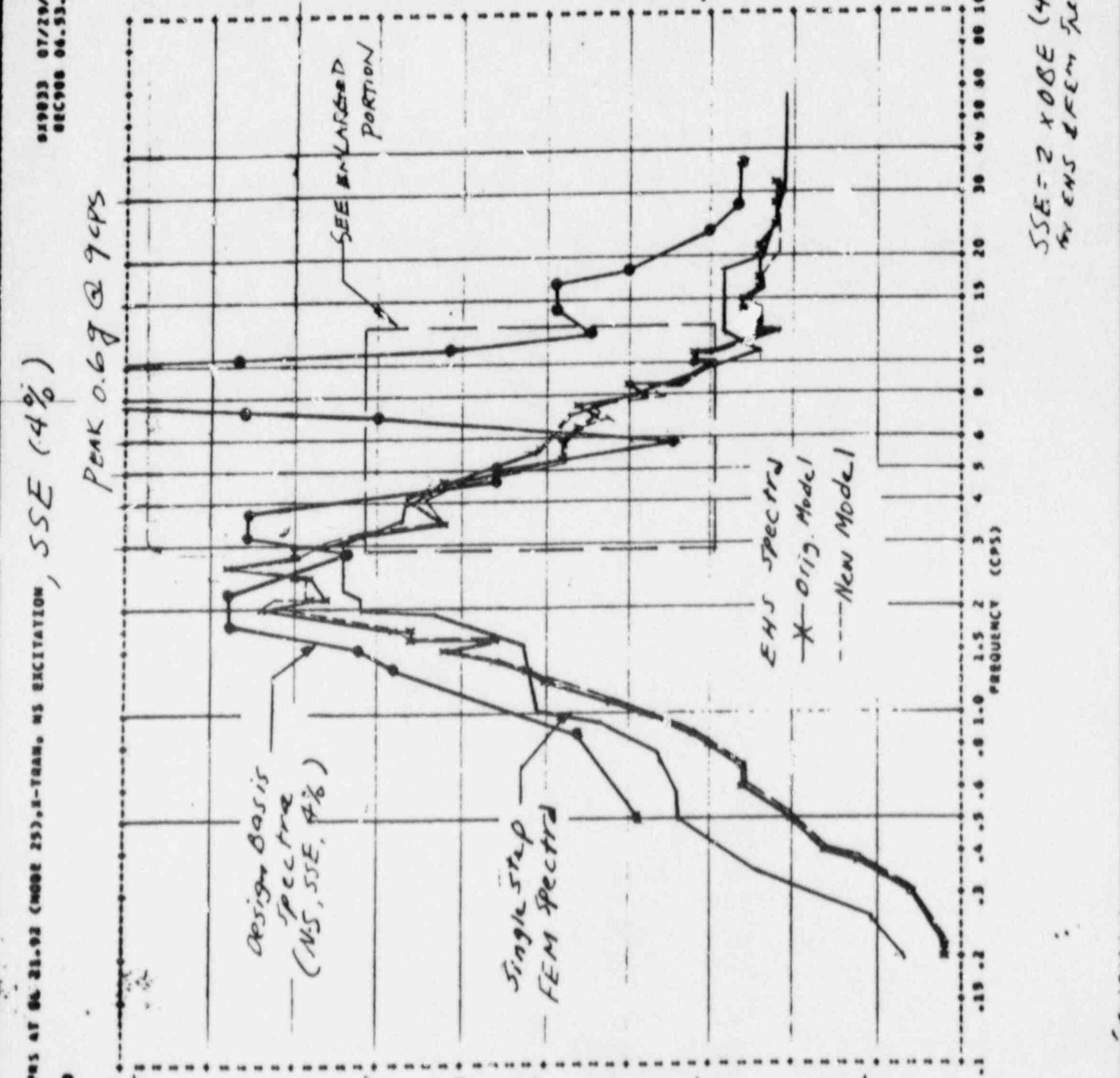
ORIGINAL HORIZONTAL MODEL

ATTACHMENT 7
ST-HL-AE-2764
PAGE 8 OF 9

ORIGINATOR: M. Guerfali

CHECKER: SCW 9/4/88

Question 5
Figure 8



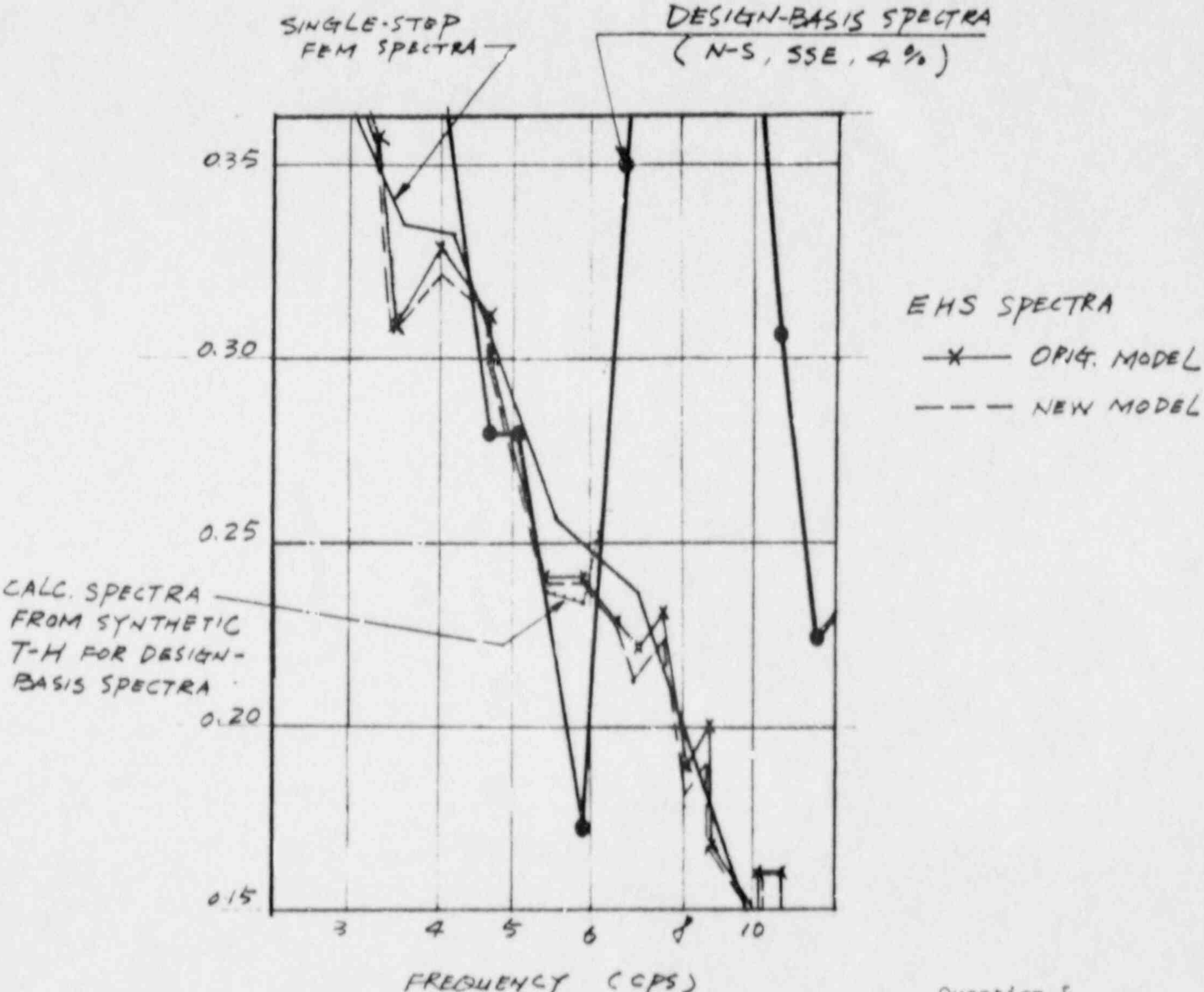
SOUTH TEXAS PROJECT
JOB NO. 14926
CALCULATION SHEET

CALC. NO. CC 9061

SUBJECT SEISMIC REANALYSIS OF THE FHB

SHEET NO. 97A

REV.	ORIGINATOR	DATE	CHECKER	DATE	REV.	ORIGINATOR	DATE	CHECKER	DATE
0	[illegible]	8/11/88							



Question 5
Figure 9

ENLARGED PORTION