PSNH PUBLIC SERVICE
Company of New Hampshire

SEABROOK STATION
Engineering Office:
1671 Worcester Road
Framingham, Massachusetts 01701
(617) - 872 - 8100

April 23, 1982

SBN-262
T.F. B 7.1.2

RECEIVED
APR 29 1982
US NUCLEAR REGULATORY COMMISSION
DOCUMENT MANAGEMENT BR
TIDC

United States Nuclear Regulatory Commission
Washington, D. C. 20555

Attention: Mr. Frank J. Miraglia, Chief
Licensing Branch #3
Division of Licensing

References: (a) Construction Permits CPPR-135 and CPPR 136, Docket Nos. 50-443 and 50-444
(b) PSNH Letter, dated April 8, 1982 "Meeting Notes; Structural Engineering Branch Design Audit," J. DeVincentis to F. J. Miraglia

Subject: Submittal of Followup Documentation; Structural Engineering Branch Design Audit

Dear Sir:

We have enclosed followup documentation from the Structural Engineering Branch Design Audit, which was conducted at the offices of United Engineers on March 29, 1982 through April 2, 1982.

Reference (b) indicated that this information would be supplied by April 19, 1982.

Very truly yours,

YANKEE ATOMIC ELECTRIC COMPANY

John DeVincentis

John DeVincentis
Project Manager

Enclosure

8204300280

B001
S1/1

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 1, DATED 3/30/82

REF. RAI NO. 220.13

RAI 220.13 (3.7(B).1.3)

As noted in this section, R. G. 1.61, Section C.3 requires that damping values, lower than those specified in Table 3.7.1, should be used if the maximum combined stresses due to static, seismic, and other dynamic loading are significantly lower than the yield stress and 1/2 yeild stress for SSE and 1/2 SSE (or OBE), respectively. Indicate whether damping values used in the analysis are in compliance with this requirement. Also, indicate your procedure to assure such compliance. In addition, if you had to use lower damping values, provide the values used for the staff's review.

Response

Observations and measurements have shown that the damping levels may vary over a significant range. Convergence problem can be encountered when attempting to match damping values with calculated stresses. Table 220.13-1 compares the damping values used for the analysis as set forth in the USNRC R. G. 1.61 with those recommended in NUREG/CR-0098. The upper values of the pair of values in the NUREG/CR-0098 column are considered to be average or slightly above average values, and the lower values are considered to be nearly lower bounds and are therefore highly conservative. The damping values given in R. G. 1.61 and used in analysis and design of structures compare close to the lower values of the NUREG/CR-0098 and therefore are considered to be conservative and suitable for design.

UE&C's design philosophy considers a structure whose design is governed by load combinations with seismic loads which are due to ground motion consistent with requirements of Regulatory Guide 1.60 and, whose design does not have excessive conservatism, will experience stress levels consistent with the requirements for using the damping values of Regulatory Guide 1.61. The Seabrook structures are without excessive design conservatism.

RAI 220.13 (Cont'd)

TABLE 220.13-1

DAMPING VALUES

(Percent of Critical Damping)

Structure or Component	Operating Basis Earthquake R.G. 1.61	Operating Basis Earthquake NUREG/CR-0098 Recommended	Safe Shutdown Earthquake R.G. 1.61	Safe Shutdown Earthquake NUREG/CR-0098 Recommended
Vital Piping	1	1 to 2	2	2 to 3
Welded Steel Structures	2	2 to 3	4	5 to 7
Bolted Steel Structures	4	5 to 7	7	10 to 15
Prestressed Concrete Structures	2	2 to 3	5	5 to 7
Reinforced Concrete Structures	4	3 to 5	7	7 to 10

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 3, DATED 3/30/82

REF. RAI NO. 220.20

220.20 (3.7(B).2.9) With regards to peak broadening of floor response spectra, we have noted your justification (FSAR Section 1.8) for deviating from Regulatory Guide 1.122 recommendation. However, provide the assessment of impact, if you were to implement the ± 15% peak broadening as required by SRP Section 3.7.2 Subsection II.9.

RESPONSE The majority of plant components, equipment and piping systems have been qualified by either tests or modal analyses. The impact of implementing a 15% spread of response spectra peaks would require reviews and revision of qualifying documentation. Many items would require re-testing or re-analyses which, when included with the above review process, would involve considerable time and expense. Current construction schedules, estimated manpower requirements and cost projections would be negated. Because of inherent design and analysis conservatisms, modifications or redesigns would not be expected from such an implementation.

All Category I structures, for which in-structure response spectra are generated, are supported on rock and hence variability of soil properties is not the consideration in broadening the peaks of floor response spectra. The structural peaks are therefore broadened by ± 10 percent.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 4, DATED 3/30/82

REF. RAI NO. 220.18

220.18
(3.7(B).2.5) The frequency increments (Table 3.7(B)-21 of FSAR) used for calculating floor response spectra are larger than those suggested in SRP Section 3.7.1. Discuss the implications of these differences and justify your frequency intervals.

RESPONSE The frequency increments used for calculating floor response spectra are based on Table N-1226-1 of ASME Boiler and Pressure Vessel Code, Section III, Division 1, Nuclear Power Plant Components, 1980 Edition, Appendix N, 'Dynamic Analysis Methods'.

The natural frequencies of the structures were included in computing response spectra. The table shown in SRP Section 3.7.1 is for meeting the spectra-eveloping requirement of the design time history where the frequency intervals are required to be smaller.

The floor response spectra calculated at frequencies shown in above referenced Table N-1226-1 and at structural frequencies will produce accurate response spectra and will meet the intent of SRP Section 3.7.1 (and also R.G. 1.122).

A typical 1 and 4% response spectra plots, generated using frequency interval according to ASME Table N-1226-1, and the envelope of these spectra are presented in Figures 220-1 and 220-2 respectively. The floor response spectra, calculated using frequency interval according to Table in SRP Section 3.7.1 (or R.G. 1.122), are also shown in dotted lines on the same figures. The comparison of the spectra in these figures shows that the dotted line spectra have additional spectral amplitudes due primarily to different frequency interval. However, the results show that the dotted line spectra are consistently lower than the envelope of the spectra.

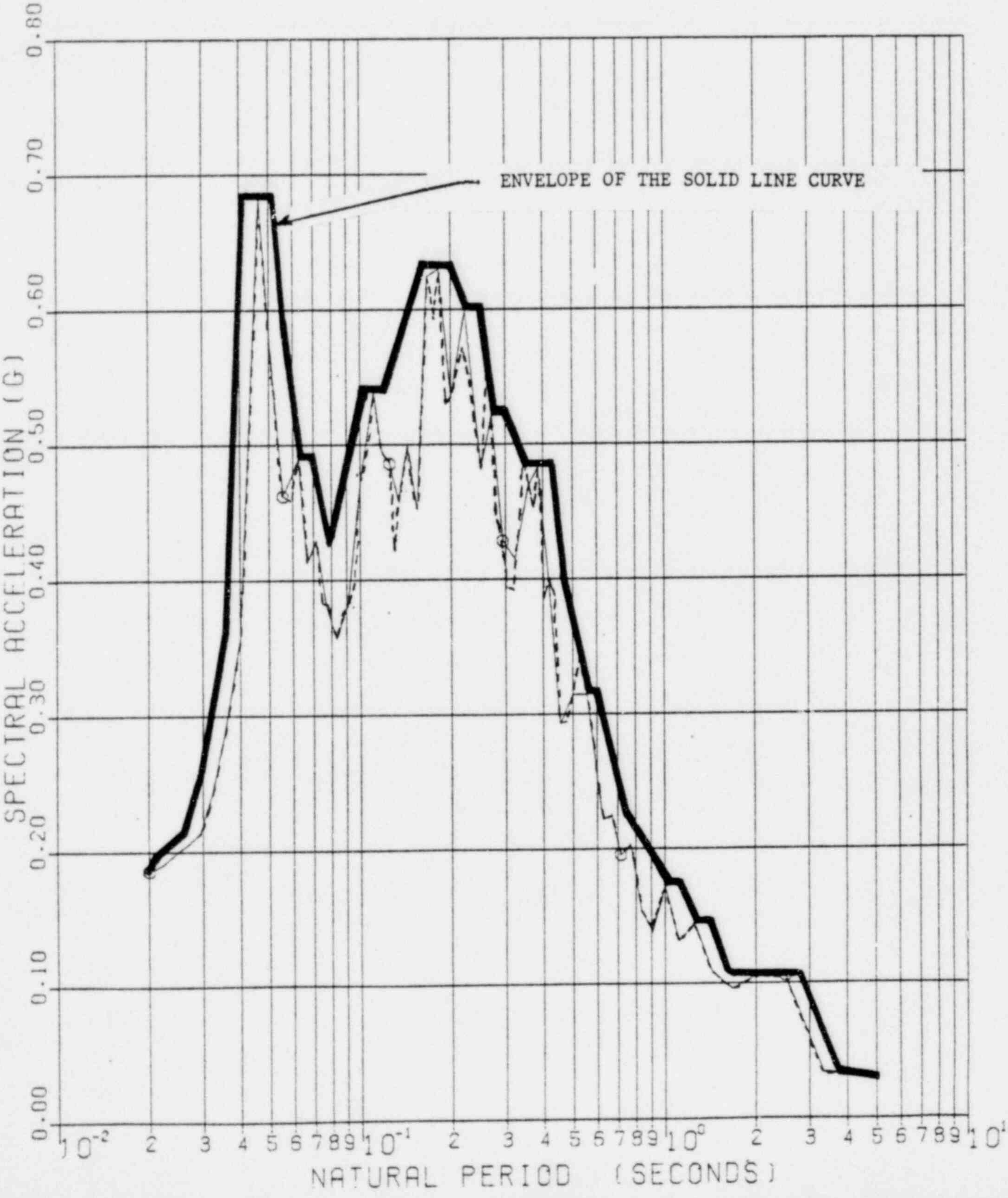
RAI 220.18 (Cont'd)

01/05/82

PSNH SEABROCK STATION, PRIMARY AUX.BLDG.-AMLIFIED RESPONSE SPECTRA (VERT)-OBE

—— ASME TABLE N.1226-1. ----- NRC REG.GUIDE 1.122 TABLE-1. PEAK SPRD= 10 %

NODE NO. 11 108.00 FT PAB VERT-OBE 4.00 % DAMPING



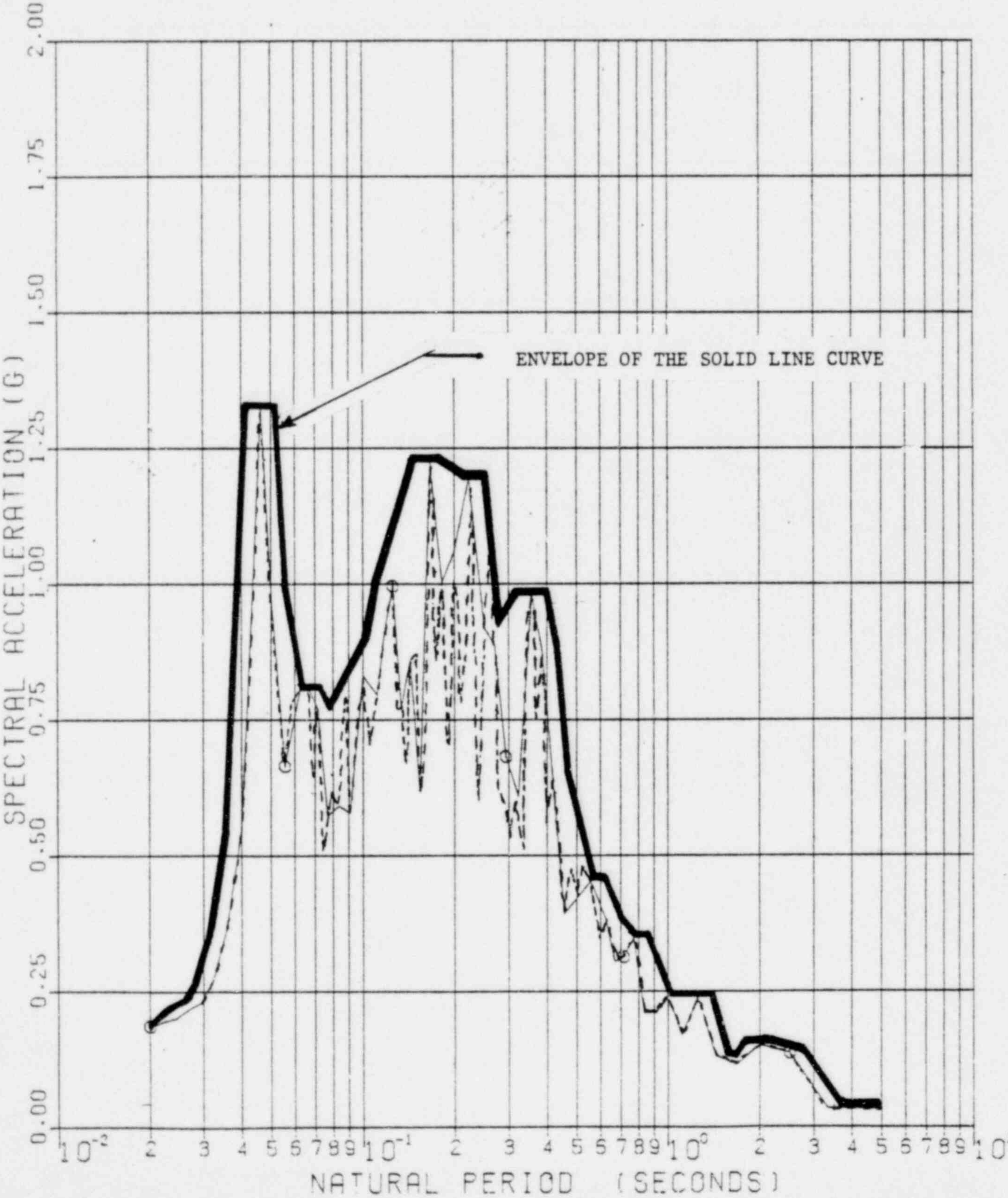
RAI 220.18 (Cont'd)

04/05/92

PSNH SEABROOK STATION. PRIMARY AUX.BLDG.-AMLIFIED RESPONSE SPECTRA (VERT)-OBE

——:ASME TABLE N.1226-1. -----:NRC REG.GUIDE 1.122 TABLE-1. PEAK SPRD= 10 %

NODE NO. 11 108.00 FT PAB VERT-OBE 1.00 % DAMPING



PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 5, DATED 3/30/82

REF. RAI NO. 220.29

RAI 220.29 (3.8.2)

The Table 3.8-6 of the FSAR shows load combinations for equipment hatch and personnel locks. It appears that you have not considered all the load combinations covered by the SRP Section 3.8.2. Confirm that the load combinations meet the requirements of the SRP Section 3.8.2. If not, justify the deviations.

RESPONSE:

The load combinations appearing in Table 3.8-6 and the stress limits of Table 3.8-10 for the equipment hatch and personnel locks are in agreement with the load combinations and stress limits defined in SRP 3.8.2, Rev. 0, 11/14/75.

The applicable design loads as described in FSAR Section 3.8.2.3 are:

- P_t - Test Pressure
- T_t - Test Temperature
- D - Dead Load
- L - Live Load
- T_o - Operational Thermal Loads
- P_o - Pressure Variation
- E - Operating Basis Earthquake
- E' - Safe Shutdown Earthquake
- P_a - Accident Pressure
- T_a - Accident Temperature

From the above applicable loads with P_a as a dominant loading and by inspecting the load combinations covered by SRP 3.8.2 Rev. 1, it is apparent that Level C Service Limit Load Combination No. (3) $(D+L+T_a+P_a+E')$, which is equivalent to Load Combination No. 5 of the FSAR Table 3.8-6, is the governing load combination. The stress limits delineated in Tabel 3.8.2-1 of SRP 3.8.2 Rev. 1 for Design Level A, B & C Service Conditions have the same allowable limit ($p_m \leq 1.0\ S_m$, $P_b \leq 1.5\ S_m$, $P_b + P_L \leq 1.5\ S_m$) as those stated in the Table 3.8 - 10 of the FSAR. Although for Testing Condition the stress limits specified in SRP 3.8.2 Rev. 1 Table 3.8.2-1 ($P_m \leq 0.75\ S_y$, $P_L \leq 1.15\ S_y$, $P_b + P_L \leq 1.15\ S_y$) are lower than those shown in the FSAR Table 3.8-10 ($P_m \leq 0.9\ S_y$, $P_L \leq 1.25\ S_y$, $P_b + P_L \leq 1.25\ S_y$), they are still higher than the corresponding stress limits for all Service Level A, B & C conditions. Therefore, this load combination $(D+L+T_a + P_a + E')$ compatible with the loads and limits delineated in SRP3.8.2 Rev. 1 will dictate the design.

Hence, all the loads applicable to the design of equipment hatch and personnel locks are listed above, others which may appear in Rev. 1 of SRP 3.8.2 do not apply. Hence, by reviewing the contents of SRP 3.8.2 Rev. 1, this design meets the current SRP requirements.

It is also confirmed that Level C service limit loading combination $D+L+T+P_o+E'$ is always lower than the combination $D + L + T_a + P_a + E'$ for Seabrook plant.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 6, DATED 3/30/82

REF. RAI NO. 220.30

RAI 220.30 (3.8.2)

The Table 3.8-10 of the FSAR shows stress limits for equipment hatch and personnel locks. Some cases in this table are not as conservative as those in SRP Section 3.8.2. The current acceptance criteria is delineated in Table 3.8.2-1 of SRP Section 3.8.2, Rev. 1 (Attachment 2). Confirm that you meet the current SRP criteria or justify the deviations from them.

Response:

See the response to RAI 220.29.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 7, DATED 3/30/82

REF. RAI NO. 220.26

220.26
(3.8.1)

Have you considered the effect on containment structural design of non-linear transient temperature gradient across the containment wall thickness caused by the LOSS-OF-COOLANT-ACCIDENT (LOCA)? If not, please include this effect in your design or justify the omission.

RESPONSE

Transient temperature gradients across the containment wall-thickness caused by LOCA were considered in the containment structural design. The design was based on the maximum forces and moments at each section for mechanical loads alone and mechanical and thermal loads combined. The liner initial temperature spike (with normal operating gradient in the concrete) was considered as an effective pressure on the concrete shell in combination with the accident pressure. The thermal gradient through the wall-thickness is initially nonlinear and becomes linear at later time into the accident. The effect of the gradient, both linear and nonlinear cases, on rebar stress is to increase the initial (due to pressure) tensile stress of the outer rebar and to decrease the tensile stress of the inner rebar. Both nonlinear and linear gradients can produce yield in the outer rebar. The general section, however, remains elastic and the yielding is a secondary effect. The linear gradient is the limiting case for maximum rebar tensile strain. The ASME B&PV Code, Section III, Div. 2 CC-3422.1 limits the calculated net rebar tensile strain to less than $2x\varepsilon_y$.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 3, DATED 3/31/82

REF. RAI NO. 220.22

Overturning Calculation (PAB)

NRC-SEB Design Audit (3/29/82 to 4/2/82): Response to Action Item #3, 3/31/82

Form 5007 Rev. 3-77

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc

CALC. SET NO.	
PRELIM.	
FINAL	28-25
VOID	
SHEET 62 OF 71	
J.O. 9763.006	

REV	COMP. BY	CHK'D BY
0	DATE	DATE
1	VDP/J.H. DATE 2-4-82 2-18-76	K.K. UBC DATE 12-27-79

NAME OF COMPANY P.S.N.H. SEABROOK STATION UNIT/S

SUBJECT [illegible] - STABILITY CHECK

D. SUMMARY

ALLOWABLE AS PER DESIGN CRITERIA-SD-66.

	TORNADO	SEISMIC	
	-	OBE	SSE
UPLIFT	1.1	1.5	1.1
SLIDING	1.1	1.5	1.1
OVERTURNING	1.1	1.5	1.1

CALCULATED RATIO.

	TORNADO				SEISMIC (O.B.E)				SEISMIC (S.S.E)			
	E-W	W-E	N-S	S-N	E-W	W-E	N-S	S-N	E-W	W-E	N-S	S-N
UPLIFT	2.27	—	—	—	2.0	-	-	—	1.72	—	-	—
SLIDING	4.2	4.87	7.83	7.2	2.18	2.55	2.29	2.09	1.22	1.46	1.3	1.18
OVERTURNING	1.90	2.00	*	*	1.97	2.08	2.62	3.05	1.67	1.16	1.37	1.67

* Obviously OK. Building dimension in N-S direction is much greater than dimension in E-W direction.

FORM 5007 REV 7/79

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc.

P.S.N.H.

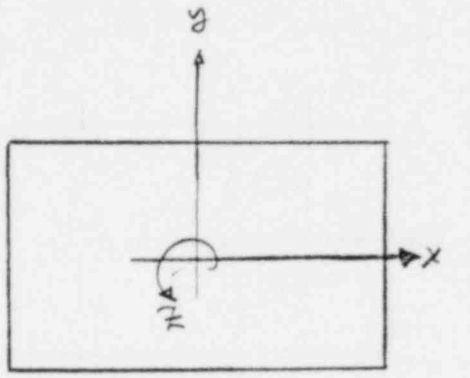
NAME OF COMPANY SEABROOK STATION UNIT/S 1 & 2

MISCELLANEOUS AREAS (190): OVT. OF RECT. BLDG ABOUT ANY AXIS

SUBJECT

CALC. SET NO.		REV	COMP. BY	CHK'D. BY
PRELIM	1	0	AL	KMK
FINAL	MA-36		DATE 4-9-82	DATE 4-12-82
VOID				
SHEET	3 OF 5		DATE	DATE
J.O.	9763006			

OVERTURNING OF RECTANGULAR BUILDING ABOUT ANY AXIS



PLAN VIEW

OVERTURNING OF BUILDINGS IS CHECKED BY CONSIDERING RATIOS OF RESISTING MOMENT TO OVERTURNING MOMENT FOR OVERTURNING ABOUT AXES IN THE x AND y DIRECTIONS SEPARATELY.

THE PURPOSE OF THIS CALCULATION IS TO PROVE THAT WHEN OVERTURNING ABOUT AN AXIS IN A COMPLETELY ARBITRARY DIRECTION IS CONSIDERED, THE RATIO OF RESISTING TO OVERTURNING MOMENT IS NO LESS THAN EITHER OF THE RATIOS COMPUTED FOR EACH DIRECTION SEPARATELY

GIVEN: M_{Rx} : RESISTING MOMENT FOR OVERTURNING ABOUT x-DIRECTION AXIS

M_{Ry} : RESISTING MOMENT FOR OVERTURNING ABOUT y-DIRECTION AXIS

M_{Ox} : APPLIED OVERTURNING MOMENT FOR OVERT. ABOUT x DIRECTION AXIS (Due to all forces)

M_{Oy} : APPLIED OVERTURNING MOMENT FOR OVERT. ABOUT y DIRECTION AXIS (Due to all forces)

M_{RR} : RESISTING MOMENT FOR OVERTURNING ABOUT ARBITRARY R-DIRECTION AXIS

M_{OR} : APPLIED OVERTURNING MOMENT FOR OVERTURNING ABOUT ARBITRARY R-DIRECTION AXIS (Due to all forces)

Toe pressures are checked not to exceed allowable bearing pressure on the foundation.

FORM 5007 REV 7/79 GENERAL COMPUTATION SHEET

(DISCIPLINE) united engineers & constructors inc

NAME OF COMPANY P.S.N.H. SEABROOK STATION UNIT/S 1 & 2

SUBJECT MISCELLANEOUS AREAS (190) : OVT OF RECT. BLDG. ABOUT ANY AXIS

CALC. SET NO.		REV	COMP. BY	CHK'D BY
PRELIM	2	0	QL	KMK
FINAL	MA-36		DATE 4-9-82	DATE 4-?-82
VOID				
SHEET	4 OF 5		DATE	DATE
J.O.	9763006			

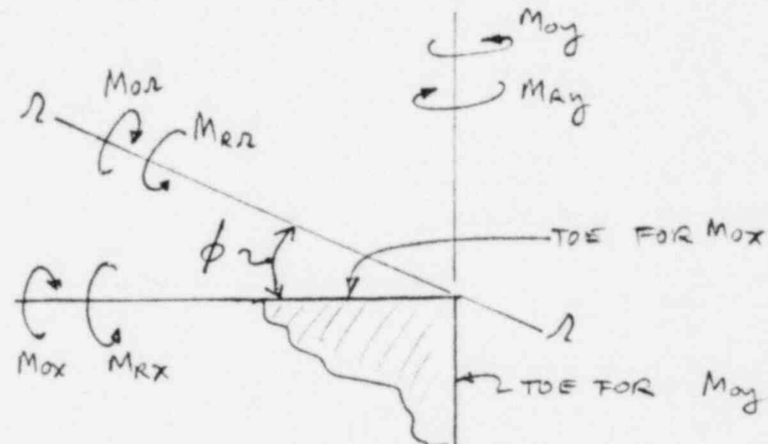
OVERTURNING OF RECT. BUILDING ABOUT ANY AXIS (CONT)

TO SATISFY DESIGN WE HAVE:

$$M_{Rx} \geq F M_{ox}$$
$$M_{Ry} \geq F M_{oy}$$

WHERE F IS THE SAFETY AGAINST OVERTURNING REQUIRED FOR DESIGN.

ϕ: ANGLE DEFINING ORIENTATION OF Ω AXIS WITH RESPECT TO x AXIS. NOTE THAT $0 < \phi < \frac{\pi}{2}$ SINCE OVERTURNING CANNOT OCCUR FOR ANY OTHER VALUE OF ϕ.



FOR ARBITRARY DIRECTION OF OVERTURNING AXIS Ω:

$$M_{R\Omega} = M_{Rx}\cos\phi + M_{Ry}\sin\phi$$

$$M_{O\Omega} = M_{Ox}\cos\phi + M_{Oy}\sin\phi$$

DESIGN CONDITION 1: RESISTANCE TO OVERTURNING ABOUT x AXIS JUST MEETS THE SAFETY FACTOR WHILE EXCEEDING IT ABOUT THE y AXIS.

$$M_{Rx} = F M_{ox}$$
$$M_{Ry} = (F+\epsilon) M_{oy} \quad \text{WHERE } \epsilon \text{ IS SOME VALUE } > 0$$

$$M_{R\Omega} = F M_{Ox}\cos\phi + (F+\epsilon) M_{oy}\sin\phi$$

$$M_{O\Omega} = M_{ox}\cos\phi + M_{oy}\sin\phi$$

$$\frac{M_{R\Omega}}{M_{O\Omega}} = \frac{F M_{ox}\cos\phi + F M_{oy}\sin\phi + \epsilon\sin\phi}{M_{ox}\cos\phi + M_{oy}\sin\phi} = \frac{F(M_{ox}\cos\phi + M_{oy}\sin\phi) + \epsilon\sin\phi}{M_{ox}\cos\phi + M_{oy}\sin\phi}$$

FORM 5007 REV. 7/79 GENERAL COMPUTATION SHEET

(DISCIPLINE) united engineers & constructors inc

P.S.N.H.

CALC. SET NO.		REV	COMP. BY	CHK'D. BY
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SHEET	5 OF 5			
J.O.	9763006			

NAME OF COMPANY SEABROOK STATION UNIT/S 1 & 2

SUBJECT MISCELLANEOUS AREAS (190): OVT OF RECT. BLDG. ABOUT ANY AXIS

OVERTURNING OF RECT. BUILDING ABOUT ANY AXIS (CONT)

$$\frac{M_{R\Omega}}{M_{O\Omega}} = F + \frac{E\sin\phi}{M_{Ox}\cos\phi + M_{Oy}\sin\phi} > F$$

since for $0<\phi<\frac{\pi}{2}$ $\cos\phi$ and $\sin\phi$ are positive and E is positive

DESIGN CONDITION 2: RESISTANCE TO OVERTURNING ABOUT BOTH AXES JUST MEETS THE SAFETY FACTOR

$$M_{Rx} = F M_{Ox}$$
$$M_{Ry} = F M_{Oy}$$

$$\frac{M_{R\Omega}}{M_{O\Omega}} = \frac{F\left(M_{Ox}\cos\phi + M_{Oy}\sin\phi\right)}{M_{Ox}\cos\phi + M_{Oy}\sin\phi} = F$$

DESIGN CONDITIONS 3 & 4

IT IS EVIDENT FROM OBSERVATION THAT WHEN RESISTANCE TO OVERTURNING ABOUT THE y AXIS JUST MEETS THE SAFETY FACTOR WHILE EXCEEDING IT ABOUT THE x AXIS, THE RATIO $M_{R\Omega}/M_{O\Omega}$ WILL BE GREATER THAN F. SIMILARLY WHEN RESISTANCE TO OVERTURNING ABOUT BOTH AXES EXCEEDS THE SAFETY FACTOR $M_{R\Omega}/M_{O\Omega}$ WILL BE GREATER THAN F.

CONCLUSION

THE FOREGOING PROVES THAT CURRENT PRACTICE OF CHECKING OVERTURNING SEPARATELY IN EACH PRINCIPAL DIRECTION GUARANTEES THAT OVERTURNING IN ANY INTERMEDIATE DIRECTION WILL NOT OCCUR since all category-I Buildings at Seabrook are rectangular in shape.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 4, DATED 3/31/82

REF. RAI NO. 220.10

Tornado Missile Calculation

/NRC-SEB Design Audit (3/29/82 to 4/2/82): Response to Item #4, 3/31/82/

Form 5604

united engineers & constructors inc.

CALCULATION CONTROL SHEET

CALC. SET NO.	
PRELIM.	
FINAL	SBSA6-1 MA
VOID	

PROJECT TITLE PSNH / SEABROOK DISCIPLINE STRUCTURAL

SYSTEM CATEGORY I STRUCTURES

SUBJECT TORNADO MISSILE PROTECTION

DESIGN CLASSIFICATION CATEGORY I

STARTED BY R.H. TOLAND DATE 3/1/79

AUTHORIZED BY K.M. KALAWADIA DATE 10/18/77

UPDATE 2/28/79

PROBLEM STATEMENT All Category I structures (except as noted) shall be designed for the effects of tornado generated missiles. Both local effects (penetration, scabbing) and overall effects (structural deformation) shall be considered. The impacted walls and slabs shall have sufficient thickness to resist perforation and the generation of secondary missiles by back surface spallation (scabbing). The impacted walls and slabs shall have sufficient capacity to resist the missile impact loads in combination with other simultaneously acting loads such that the maximum deformation does not exceed the maximum allowable defined as the permissible ductility ratio times the yield displacement. Also, the maximum displacement shall not interfere with the function of any safety related equipment within the structure.

DESIGN BASIS The missile spectrum and analysis methods shall be in conformance with the provisions of SRP's 3.5.1.4 and 3.5.3, respectively. Appendix C of ACI 349 shall be used as a guide.

Only one tornado missile is to be considered acting at a time. Tornado missile loads shall be combined with other loads as indicated in the FSAR. All structures required to resist tornado missile loads shall have external wall and roof thicknesses of minimum 2'0". Yield line theory shall be used to calculate slab and wall capacity. Equivalent single DOF elasto-plastic systems shall be defined for the dynamic response analysis. Both flexure and shear are considered.

TOTAL NUMBER OF SET COMPUTATION SHEETS 121 (3/82)

FINISHED BY R. H. TOLAND CHECKED BY D.K. Majumdar.

	CHECKER	DESIGN SUPER	COGNIZANT ENG'R	DESIGN REVIEW
BY				
DATE				

REVISION 1 STARTED DATE ____ BY ____

NRC - SEB Design Audit (3/29/82 to 4/2/82): Response to Item #4, 3/31/82

Form 5603

united engineers & constructors inc

CALCULATION SUMMARY & REFERENCE SHEET

CALC. SET NO.	
PRELIM.	
FINAL	SBSAB-1MA
VOID	
SHEET	OF
J.O.	

PROJECT TITLE PSNH / SEABROOK DISCIPLINE STRUCTURAL

SYSTEM CATEGORY I STRUCTURES

SUBJECT TORNADO MISSILE PROTECTION

DESIGN CLASSIFICATION CATEGORY I

REV	COMP. BY	CHK'D BY
0	RHT DATE 5/25/79	DKM. DATE 5/25/79
	DATE	DATE

SUMMARY/CONCLUSIONS

1. The missile spectrum and velocities used in this analysis are those found in SRP 3.5.1.4, Rev 1, paragraph 5 (6/77) and Reg. Guide 1.76. This is a minor variation from the PSAR position.
2. The analysis methods are consistent with the provisions of SRP 3.5.3
3. The two (2) foot minimum wall thickness is adequate to resist back face scabbing.
4. Structural failure is predicted for the piers of the Main Steam and Feedwater pipe chase [East & West], the roof hatch of the Service Water Pump House and the missile shield wall at El +44 of the Emergency Feedwater Bldg. In addition, the roof hatch of the Diesel Generator Bldg is marginal. Therefore these structural elements require additional capacity

Cont'd on other side.

REFERENCES: (SPECIFICATIONS, DRAWINGS, CODES, CALCULATION SETS, TEXTS, REPORTS, COMPUTER DATA PSAR ETC.)

1. Project Memo from K.M. Kalawadia to SAG, dated 10/18/77.
2. SRP 3.5.1.4, Rev 1.
3. SRP 3.5.3 Barrier Design Procedures
4. Structural Design Criteria for Seabrook Station
5. Seabrook PSAR
6. ACI 349 App. C.
7. ACI 318-77
8. Structural Analysis and Design of Nuclear Plant Facilities (Draft) ASCE, 1976.
9. Bechtel BC-TOP-9A
10. "Assessment of Empirical Concrete Impact Formula" by G.E. Sliter, EPRI, April 1979.
11. Introduction to Structural Dynamics, Biggs, McGraw Hill, 1964.
12. Vibration of Plates, Leissa, NASA SP-160

Summary / Conclusions Cont'd

5. All other structural elements investigated can resist missile impact

6. The Containment Equipment Hatch Shield Wall can also resist the missile impact within the bounds of the dimensions and assumptions.

7. The Control Room Make-up Air Intake Structure was not investigated due to lack of design data

8. The containment shell and dome was not investigated because of its ability to resist aircraft impact.

Form 5007 Rev. 3-77

(DISCIPLINE)

GENERAL COMPUTATION SHEET

united engineers & constructors inc.

NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

CALC. SET NO.		
PRELIM.		
FINAL	56SA8-1MA	
VOID		
SHEET i OF		
J.O. 9763.006		
REV	COMP. BY	CHK'D BY
0	DATE 3/12/79	RHT DATE 4/30/79
	DATE	DATE

1) NOTATION

A = Cross Sectional Area.

A_s = Area of Tensile Reinforcement

A_s' = Area of Compressive Reinforcement

a = Shorter Span Or Depth Of Stress block having Tensile Reinforcing Steel only.

a^* = Depth of Stress block having both Tensile and Compressive Reinforcing Steels.

b = Longer Span Or Width of Concrete Section.

D = Flexural Rigidity of plate. or Depth of penetration

d = Effective depth of Concrete section

d' = Concrete Cover of Reinforcing Steel.

Es = Modulus Of Elasticity of Reinforcing Steel = 29×10^6 Psi.

Ec = Modulus of Elasticity of Concrete = $57000\sqrt{f_c'}$ Psi.

F = Co-efficient for Moment of inertia of cracked Section

f_y = Yield Strength of Reinforcing Steel = 60,000 Psi.

f_c' = Specified Compressive Strength of Concrete = 3000 Psi.

f_o = Frequency

I_a = Average Moment of inertia

I_g = Moment of inertia of gross Concrete Section.

K = Co-efficient constant.

M_U = Ultimate Moment Capacity of Concrete Section having Tensile steel only.

M_U^* = Ultimate Moment Capacity of Concrete Section having both Tensile and Compressive steel.

N = Constant

n = Modular Ratio = Es/Ec

t = Overall thickness of Concrete member

T = Period of Oscillation of Vibration.

β_1 = Co-eff. = 0.85

ρ = Ratio Of Tensile Steel Area by Concrete Sectional Area = A_s/bd, Or mass/unit area

ρ' = Ratio of Comp. Steel area by Conc. Sectional Area = A_s'/bd.

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SUBJECT TORNADO MISSILE PROTECTION

I) NOTATION (CONT.)

ϕ = Capacity Reduction Factor.
ω = Frequency of Vibration in rad/sec.

V = Velocity.
W = Weight

μ = Ductility Ratio
ν = poission Ratio

X = Penetration
t_s, t_d = time in Secs.

F_1 = Force
V_0 = velocity
g = gravity acceleration
C_T, C_R = Constant

P_U = Ultimate Capacity in lbs or kips.

L = Span length
b_0 = Perimeter
λ = Constant

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REFERENCES:

1) Project memo from K.M. Kalawadia to SAG, dated 10/18/77

2) Structural Analysis & Design of Nuclear plant facilities prepared by ASCE, 1976 (Draft)

3) Structural Design Criteria for Seabrook Station by U.E.C. SD-66.

4) ACI-318-77 Reinforced Concrete Building Code, Commentary.

5) "Vibration of Plates" by A.W. Leissa; NASA-SP-160

6) "Assessment of Empirical Concrete Impact Formula" by Gorge E. Sliter, paper presented at ASCE Conference on Civil Engineering and Nuclear Power, Boston, Mass. April 2-3, 1979

7) SRP 3.5.3 "Barrier Design Procedures" 11/24/75

8) NRC Spectrum of SRP 3.5.1.4, Rev. 1.

9) REPORT BECHTEL BC-TOP-9A.

10) "Introduction to Structural Dynamics" by John M. Biggs. McGraw-Hill Book Company, 1964.

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SUBJECT TORNADO MISSILE PROTECTION

A. MISSILE SPECTRUM & VELOCITIES

NRC spectrum of SRP 3.5.1.4, Rev 1, paragraph 5 (6/8/77)

Missile	Description	W (lb)	V (fps) Zone I **
A wood plank	4"x12"x12'	200	422 (338)
B steel pipe	3"φ x 10'	78	211 (169)
C steel rod	1"φ x 3'	8	317 (253)
D steel pipe	6"φ x 15'	285	211 (169)
E steel pipe	12"φ x 15'	743	211 (169)
F wood pole *	13½"φ x 35'	1490	211 (169)
G auto *	—	4000	106 (84)

* h ≤ 30' above highest ground level within ½ mile
** Vertical velocities are 80% of the horizontal velocities

B. DESIGN FOR MISSILE PROTECTION - ANALYSIS OF MISSILE EFFECTS

1. Local Effects

Use modified Petry formulae (SRP 3.5.3) *
with charts in "Structural Design Criteria for Seabrook Station"

f'_c = 3000 PSI

criterion: wall thickness must prevent scabbing

2. Overall Response

a. High rigidity missiles that penetrate (pipes and rebar) - use a modified Williamson and Alvy method
b. Auto - use force-time history in Bechtel BC-TOP-9A
c. Utility pole - use force-time history developed using crushing strength generated by EPRI tests

criterion: ① deformations must not exceed $\mu_{max} X_e$ (allowable ductility ratio x elastic deflection)
② max deformations must not endanger safety related equipment and/or its operation

* MOD NDRC FORMULAE ARE NOW RECOGNIZED TO GIVE BEST PREDICTIONS. THE WALLS & SLABS ARE EVALUATED BY THIS METHOD ALSO.

LOCAL EFFECTS.

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Ref: ① Structural Design Criteria for Seabrook Station Appendix "A".

② SRP 3.5.3 'Barrier Design Procedures', 11/24/75

CALCULATION OF PENETRATION IN CONCRETE WALLS AND SLABS DUE TO LOCAL EFFECTS OF TORNADO MISSILES USING MODIFIED PETRY FORMULA: $D = K\frac{W}{A}\log_{10}\left[1+\frac{V^2}{215000}\right]$ in Ft

NO.	MISSILE		Description	Area of Missile A ft²	Weight of Missile W lbs	Velocity of Missile V ft/sec.	* Co-ef. for Penetration Material K	$\frac{W}{A}$ lbs/ft²	$\frac{V^2}{215000}$ = Y	$V' = \log_{10}(1+Y)$	Penetration $D = K\frac{W}{A}V'$ Ft	in	Direction
1	A	Wood Plank	4"x12"x12'	0.33	200	422	3.5×10^{-3}	600	0.8283	0.262	0.5502	6.6	H
						337.6			0.5301	0.185	0.3885	4.7	VRT
2	B	Steel Pipe	3"Φx10'	0.049	78	211	"	1589	0.2071	0.082	0.4560	5.5	H
						168.8			0.1325	0.054	0.3003	3.6	VRT
3	C	Steel rod	1"Φx3'	0.0055	8	317	"	1468	0.4674	0.167	0.8573	10.3	H
						253.6			0.2991	0.114	0.5852	7.0	VRT
4	D	Steel pipe	6"Φx15'	0.1963	285	211	"	1451.5	0.2071	0.082	0.4166	5.0	H
						168.8			0.1325	0.054	0.2743	3.3	VRT
5	E	Steel pipe	12"Φx15'	0.7854	743	211	"	946	0.2071	0.082	0.2715	3.3	H
						168.8			0.1325	0.054	0.1788	2.15	VRT
6	F	Wood pole	13½"Φx15'	0.994	1490	211	"	1499	0.2071	0.082	0.4302	5.16	H
						168.8			0.1325	0.054	0.2833	3.4	VRT
7	G	Auto		20	4000	106	"	200	0.0523	0.022	N.A		H
						84.8			0.0334	0.014	N.A		VRT

Note:- H = HORIZONTAL DIRECTION.

VRT = VERTICAL DIRECTION.

* = Value for Reinforced Concrete of $f'_c = 3000$ psi.

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SUBJECT Tornado Missile Protection

Ref ① Structural Design Criteria for Seabrook Station App. A.
② SRP 3.5.3 'Barrier Design Procedures' 11/24/75

Since barrier thickness $t < 3D$ for the 1" rebar

$$D' = D\left[1 + e^{-4(\alpha'-2)}\right] \qquad \alpha' = t/D = 24/10.3 = 2.33$$

$$D' = 10.3\left[1 + e^{-1.32}\right] = 13.05 \text{ in.}$$

Thickness of barrier req'd to prevent scabbing

use Fig A.2-3 from Ref 1. (See next page)

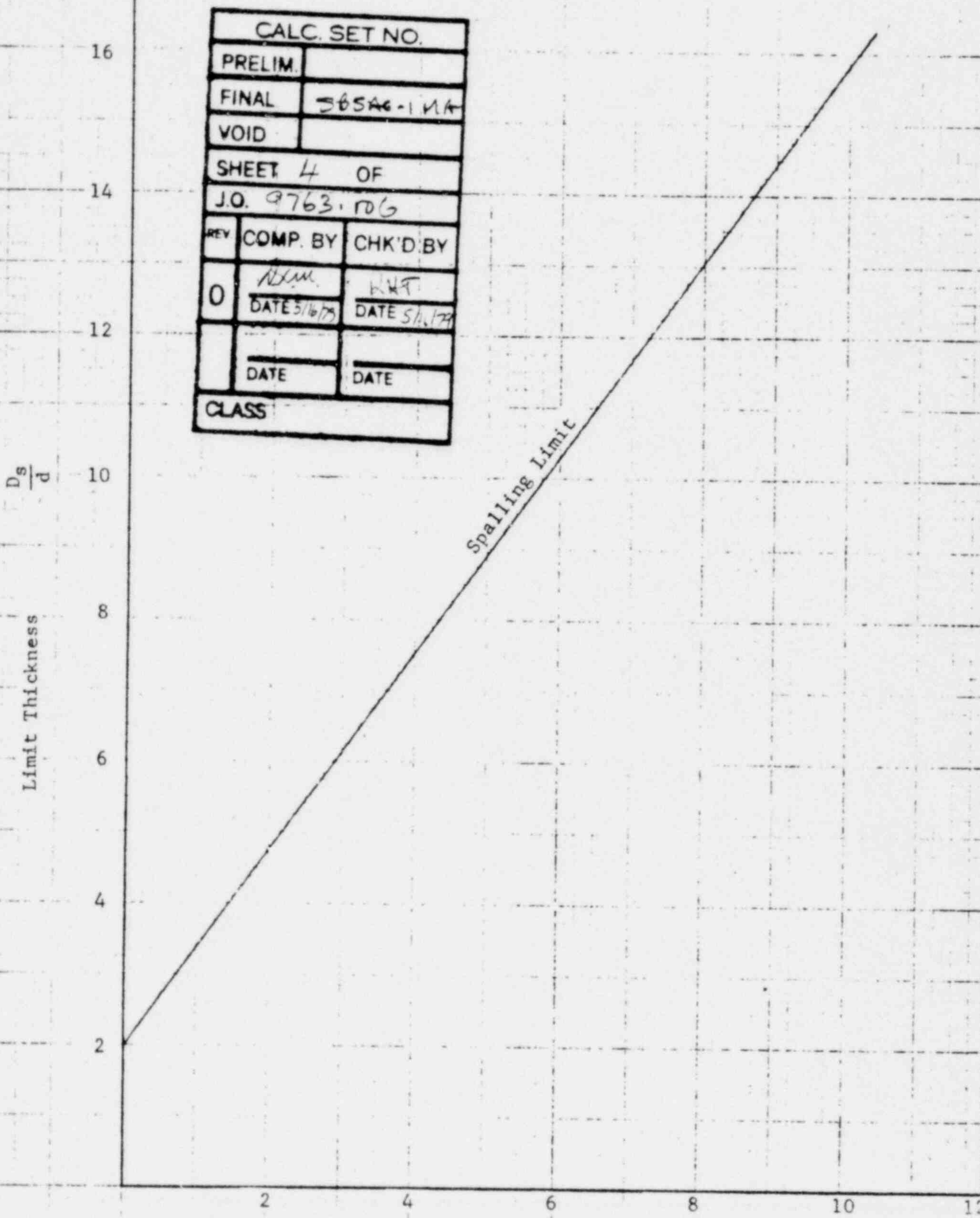
$D'/d = 13.05$ (1" rebar)

$D_s/d = 19.94$ (extrapolated from curve using TI SR51)

$\therefore D_s = 19.94$ in. + 20% $\Longrightarrow$ 23.93 in < 24 in OK

All other missiles (pipes) produce lesser effect. (by Modified Petry criterion)

FIGURE A.2-3



Relation of penetration to scabbing limit thickness.

D' = Penetration

d = Diameter of missile

D_s = Thickness required to prevent spalling.

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NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

Calculations of penetration depth and minimum req'd thickness to prevent scabbing following the recommendations in an ASCE paper by G. Sliter[1] (EPRI) presented at ASCE meeting, Boston, April 1979. *

12" ϕ pipe missile - experimental results show that this is the worst case missile for local effects.

penetration — Modified NDRC formula

$$x = \sqrt{4 KNWd \left(\frac{V}{1000 d}\right)^{1.8}} \text{ inch for } \frac{x}{d} \le 2.0$$

$K = 180/\sqrt{f'_c} = 3.286$ for $f'_c = 3000$ PSI
$N = 0.72$ flat nose (best correlation to data) - Missile Shape Factor.
$W = 743$ lbs = Wt. of Missile in lbs.
$d = d_o = 12.75$ IN. = Dia. of Missile in inch.
$V = 211$ fps = Striking Velocity of Missile (in fps)

$\boxed{X = 7.47 \text{ IN.}}$ $\frac{X}{d} = \frac{7.47}{12.75} = 0.59 < 2.0$ O.K. to use the above Formula.

thickness to prevent scabbing — NDRC Scabbing Formula:-

$$t_s/d = 2.12 + 1.36\, x/d \quad \text{for } 0.65 \le x/d \le 11.75$$

use $d = d_e = 4.47$ IN (effective diameter as though solid cylinder)
$x/d = 7.47/4.47 = 1.67 \begin{matrix} > 0.65 \\ < 11.75 \end{matrix}$ ∴ O.K.

$t_s/d = 4.39 \Rightarrow \boxed{t_s = 19.64 \text{ IN}}$ + 20% → $t_s = 23.56$ IN, < 24 IN

↑ normal scatter range in experimental results

All other pipe missiles (and rebar) produce lesser effects. Therefore, design adequacy of 24 inch walls and slabs for local effects in Zone I is established

* Sliter's recommendations were based on a review of all experimental data to date and the matching of predicted versus experimental results.

1 "Assessment of Empirical Concrete Impact Formulas," G E Sliter, ASCE Conference on Civil Eng'g and Nuclear Power, Boston, April 2-3, 1979.

OVERALL RESPONSE.

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NAME OF COMPANY SEABROOK UNIT/S

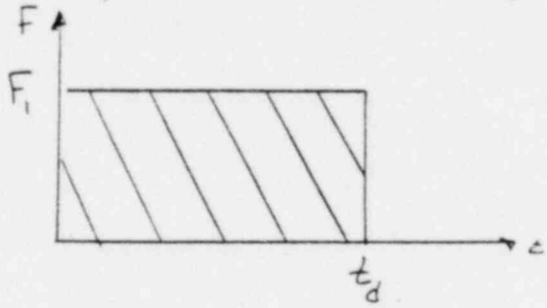
SUBJECT TORNADO MISSILE PROTECTION

Procedures and data for calculating overall response

Missile force-time definition

A. Pipes and Rebar (missiles that penetrate the target)

Modified Williamson and Alvy ; Assume a rectangular pulse forcing function. (velocity decreases linearily, force is constant)



$$F_1 = \frac{1}{2}\frac{W}{g}\frac{V_0^2}{X}$$

$$t_d = \frac{2X}{V_0}$$

where W - weight of missile (lb)
V_0 - impact velocity (IN/S)
X - penetration depth (IN)
$g = 386.4 \text{ in/s}^2$

Procedure : 1. Calculate F_1 and t_d for the given missiles and concrete strength.

2. Calculate slab (or beam) capacity, R_m, and natural period, T,

3 Calculate the static load, R_s, to be combined with the impact load and the displacement, X_s, due to the static loads.

4 Calculate the effective available resistance, $R_M' = R_M - R_S$, and the effective ductility ratio $\mu' = (X_m - X_s)/(X_e - X_s)$ where X_e is the yield displacement, $X_M = \mu X_e$ and μ is the allowable ductility ratio.

5. Form the ratios $C_T = t_d/T$ and $C_R = R_M'/F_1$, enter Figure 1 and read the corresponding X_m/X_e.

6. Compare to μ'

$X_m/X_e > \mu'$ gross failure
$\leq \mu'$ no failure
[≤ 1 elastic response
> 1 permanent displacement]

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SUBJECT TORNADO MISSILE PROTECTION

Calculation of F_1 and t_d for the pipe and rebar missiles.

Modified NDRC formula provides best correlation to experimental results

penetration depth $X = \sqrt{4KNWd\left(\frac{V}{1000d}\right)^{1.8}}$ for $X/d \le 2.0$

$K = 180/\sqrt{f'_c}$ = 3.286 for f'_c = 3000 PSI
N = .72 flat nosed bodies
.84 blunt nosed bodies ← used here
W - missile weight (lbs)
d - missile diameter (in)
V - impact velocity (fps)

Missile	Wt (lb)	V_O (fps)	V_O (in/s)	X (in.)	F_1 (KIPS)	t_d (s)	Direction
steel rod, 1"ϕ	8	317	3804	3.8	39.4	.0020	Horizontal
		253	3043	2.9	33.1	.00191	Vertical
steel pipe, 3"ϕ	78	211	2532	4.65	139.2	.00367	Horizontal
		169	2026	3.8	109.0	.00375	Vertical
steel pipe, 6"ϕ	285	211	2532	6.9	342.7	.00545	Horizontal
		169	2026	5.7	266.1	.00562	Vertical
steel pipe, 12"ϕ	743	211	2532	8.3	742.6	.00656	Horizontal
		169	2026	6.8	581.5	.00671	Vertical

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Comparing the relative effects of the pipe and rebar missiles for the Williamson and Alvy analysis (horiz. velocities only) Reference Figure 1.

Missile	C_T	C_R	$C_T/C_T(12")$	$C_R/C_R(12")$
12" ϕ pipe	$.00656\ f_0$	$.00135\ R_m'$	1	1
6" ϕ pipe	$.00545\ f_0$	$.00292\ R_m'$	.831	2.163
3" ϕ pipe	$.00367\ f_0$	$.00718\ R_m'$	.559	5.319
1" ϕ rebar	$.0020\ f_0$	$.0254\ R_m'$	.305	18.815

$f_0 = \frac{1}{T}$ = Natural frequency of the target.

For the 6" ϕ, 3" ϕ pipes and 1" ϕ rebar, the ratios $C_T/C_T(12")$ are less than 1 and the ratios $C_R/C_R(12")$ are greater than 1.

By observation of figure 1: ① fixed C_R, decreasing C_T → ratios of X_m/X_e decrease (or constant)

② fixed C_T, increasing C_R → ratios of X_m/X_e decrease

Therefore, the 12" ϕ pipe produces the largest deformations of all these missiles. Hence the analysis will only investigate the 12" ϕ pipe as the bounding case.

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B. Utility Pole and Wood Plank

A force-time history was developed for the utility pole using the Riera formula (Ref 2) as

$$F(t) = F_c A + \mu V^2(t)$$

- $V(t)$ — instantaneous velocity
- μ — mass / unit length
- A — cross-section area
- F_c — crushing strength
- $F(t)$ — instantaneous force

The experimental data from the Sandia tests for EPRI[1] were used for the crushing strength F_c. A constant F_c = 1200 PSI was used. The Riera formula was then integrated in time for the definition of F(t). The forcing function was then approximated by a rectangular pulse as below.

	V (fps)	F_1 (K)	t_d (s)
HORIZ.	211	202.5	0.054
VERT	169	192	0.044

Procedure: same as for pipes and rebar

A similar force-time history was generated for the wood plank. An approximate representation was a rectangular pulse with F_1 = 103 kips and t_d = 0.029 s. for V = 422 fps. By comparing C_T and C_R for the plank and pole, it is seen that the pole is always a more severe missile problem. Hence only the pole is considered in the subsequent calculations.

NOTE: All experimental data has dramatically shown that the wood pole disintegrates upon impact with minimal damage to walls of 12 inch thickness or greater.

1 "Full-Scale Tornado Missile Impact Tests," EPRI NP-440, Final Report, July 1977

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C. Auto

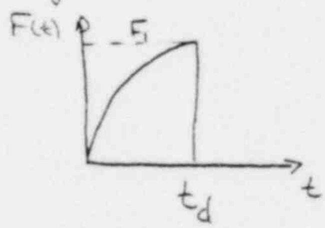
We are using a force-time history described in Bechtel's report BC-TOP-9A, Rev 2, 9/74 "Design of Structures for Missile Impact".

$F(t) = 0.625\, V_s W_m \sin 20t$ for $0 \le t \le 0.0785$ sec. i.e. $\frac{\pi}{2t_d} = 20$

$= 0$ $t > 0.0785$ sec.

V_s - impact velocity (fps)
W_m - weight of auto (lbs)

$F_1 = 0.625\, V_s W_m$
$t_d = 0.0785$ s



V (fps)	F_1 (K)
106	265
84	210

Procedure: Same as for pipes and rebar except use Figure 2

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The allowable ductility ratio is taken from the provisions of ACI 349, Sec. C.3

Flexure controls

$\mu =$ either $0.05/(\rho-\rho') \leq 10$

or determined by the rotational capacity τ_θ, in radians, of any yield hinge; limited to $0.0065(d/c)$, not to exceed 0.07 radians.

Shear controls

$\mu = 1.3$ for shear carried by concrete alone.

For flexure to control design, the load capacity of a structural element in shear shall be at least 20% greater than the load capacity in flexure.

For the relatively non-deformable missiles, pipes and rebar, shear controlled slab failure is precluded since the missile does not perforate. Therefore pipes and rebar shall be checked for local effects and flexure controlled structural effects only

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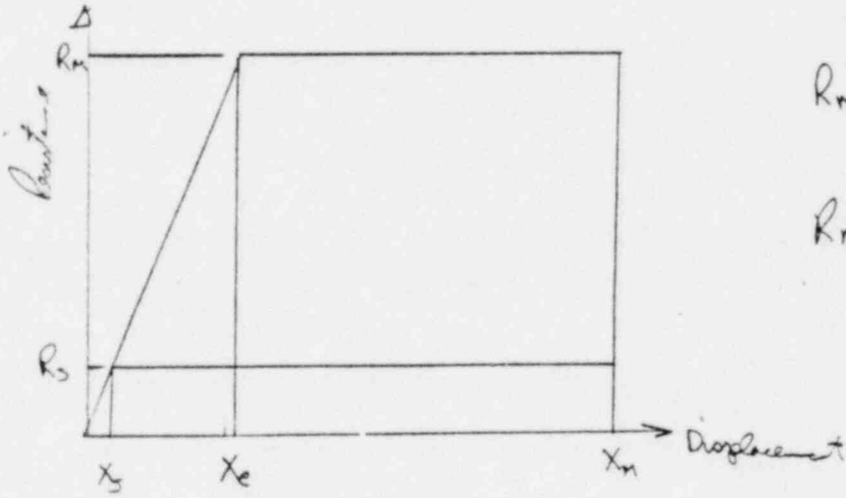
SUBJECT TORNADO MISSILE PROTECTION

COMBINING LOADS (See page #41-43 for applied loads)

R_s - static force to be combined with impactive loads

X_s - displacement due to static loads.



R_m' - effective available capacity (resistance)

$R_m' = R_m - R_s$

$$\mu_{all}' = \mu_{all}\left(\frac{X_M - X_s}{X_e - X_s}\right)$$ effective ductility ratio. (Ref. #4)

R_m', μ_{all}' - defined for each slab, wall, column of interest

For distributed loads, R_s is calculated on the basis of equivalent maximum moment.

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Several walls had boundary conditions which were difficult to analyze without F.E modelling (ex. 2 adjacent sides fixed, the other 2 free). In these cases R_m and T were calculated but R_s was not (and X_e in some instances). If ample margin against failure was present (the result in all cases), no additional analysis was deemed necessary.

Blank sections for X_e, X_m, R_s, R_m' and μ' are found in the calculations.

In these cases, R_m' was taken as R_m and $\mu_{ALL} = \mu_{ALL}$ for analysis

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FIGURE 1.

X_m/X_e CURVES FOR ELASTO-PLASTIC SYSTEM, RECTANGULAR IMPULSE LOAD

REF Biggs, 'Introduction to Structural Dynamics'

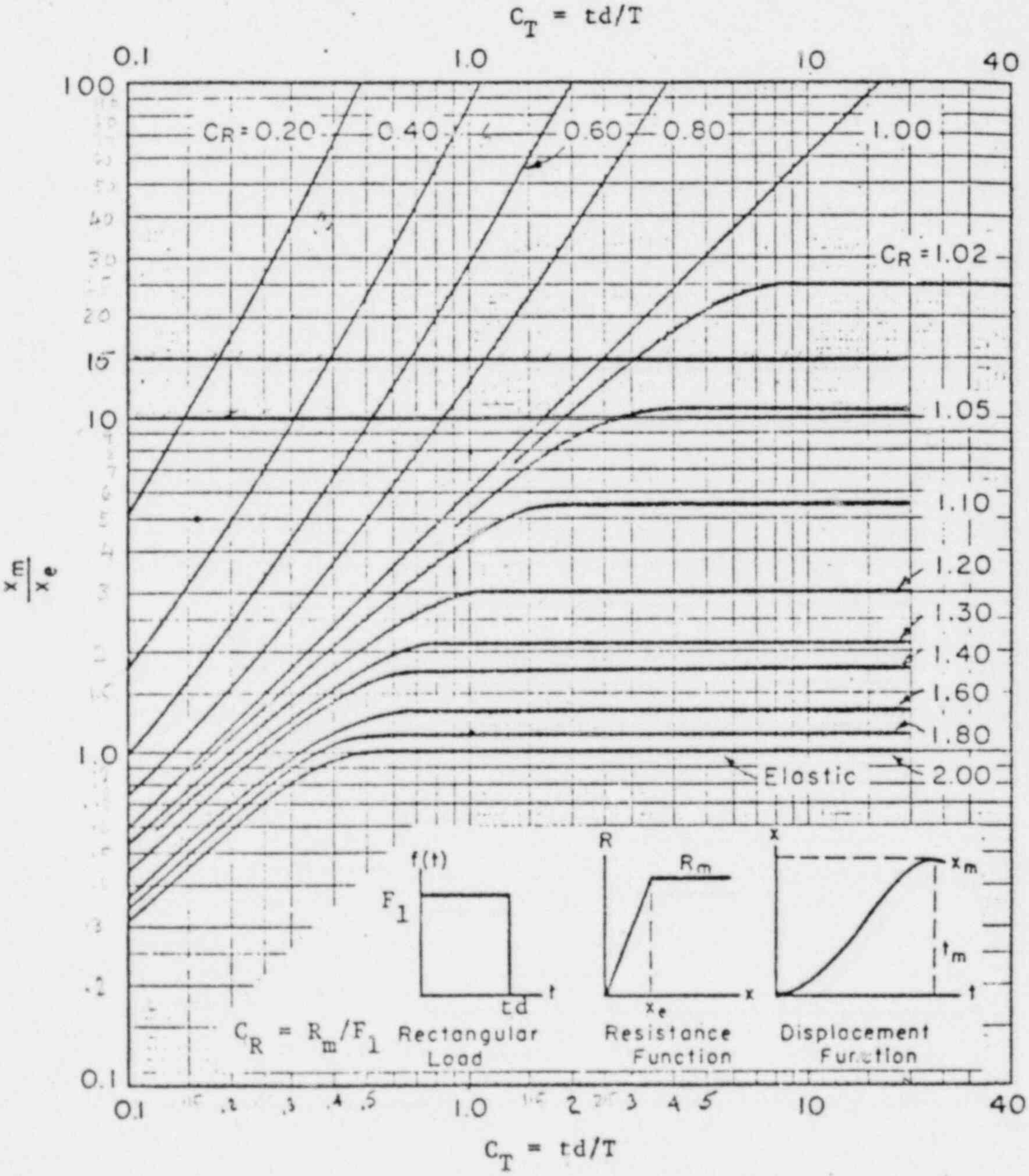

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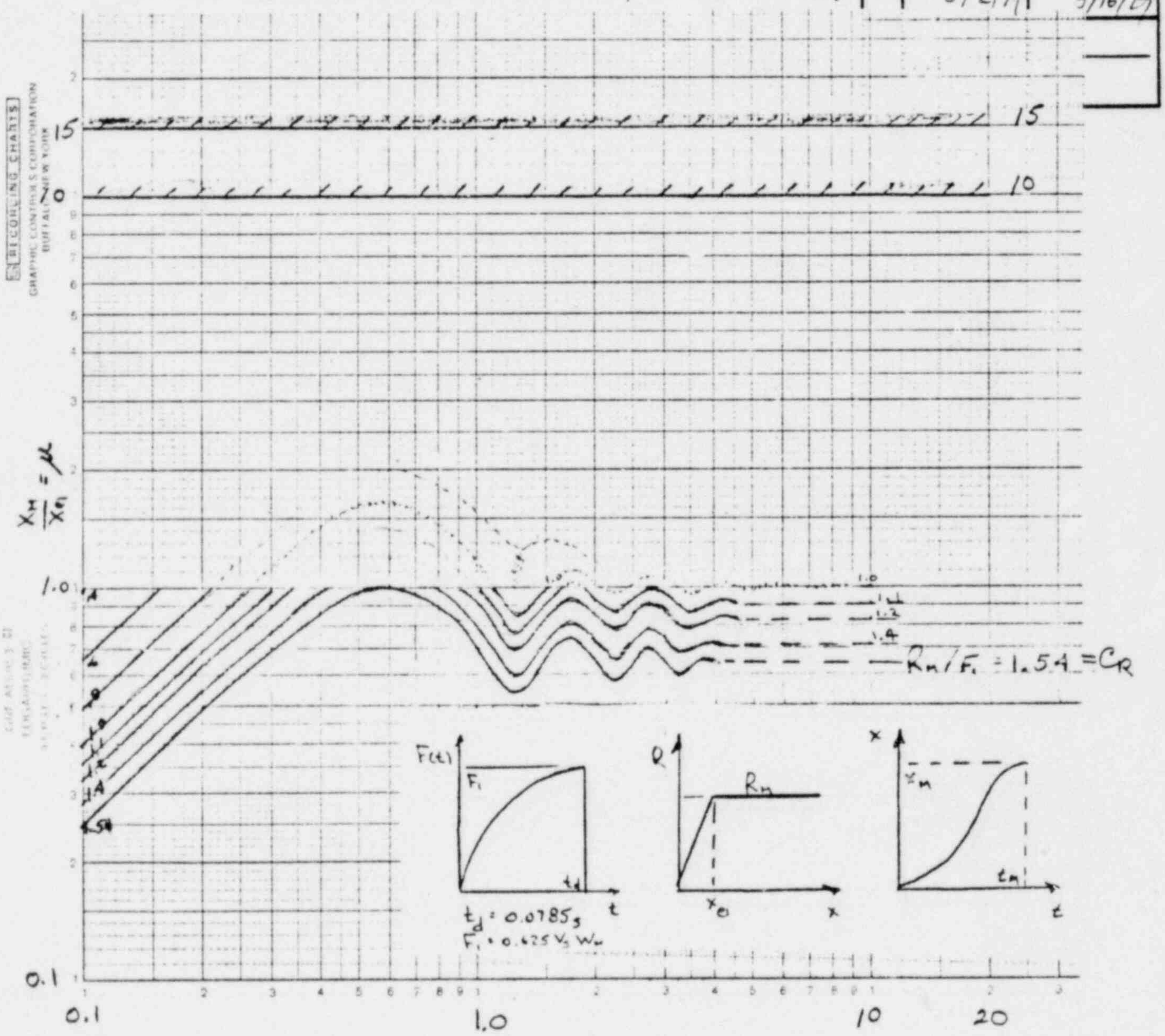
FIGURE 2.

X_m/X_e CURVES FOR ELASTO-PLASTIC SYSTEM, QUARTER-SINE LOAD



ELASTIC RESPONSE ($x_m/x_e \leq 1$)
CALCULATED

REF: SHOCK & VIBRATION BOOK
Chapter 8 "Transient Response to Step and pulse functions" Ay. 2

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SUBJECT TORNADO MISSILE PROTECTION.

Calculation of Loads.

1) Dead load

a) wt. of 2' thick Slab = 2'×150 = 300 lbs/ft²
wt of Roof Slope (av. 3" thick) = $\frac{3}{12}$×150 = 37.5 lbs/ft²
wt. of Fixed Equipment loads = 50 lbs/ft²

Total D.L = 387.5 lbs/ft²

b) Total D.L. for Only R.H.R. Vault = 537.5 lbs/ft²
(Residual Heat Removal & Containment Spray Equipment Valult)

2) Live load

Roof Snow = 74 lbs/ft²

3) Wind load. (TORNADO Wind only)

q_{av} = Size factor depending on length × 332 PSF

Take av. value of 0.8 for Size factor dep up on length as the length varies for 48 ft to 134 ft. (Ref. # 3 Fig 4.4.2-1, Page 49)

∴ q_{av} = 0.8 × 332 ≃ 266 PSF

W_p = Total differential pr = 432 PSF (outward Direction)

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For Rectangular structure.

W_{w_1} = Tornado Wind External pressure

$= 0.8\, q_{tav}$ (for windward wall)

$= -0.7\, q_{tav}$ (Roof)

W_{w_2} = Tornano Wind Internal pr.

$= \pm 0.3\, q_{tav}$.

∴ Total Effect of wind on wall.

$W_w = W_{w_1} + W_{w_2}$ or $W_w = W_{w_1} + 0.5\, W_p$

$= 0.8\, q_{tav} + 0.3\, q_{tav}$ or $W_w = 0.8\, q_{tav} - 0.5 \times 432$

$= 1.1\, q_{tav}$ $= 0.8 \times 266 - 0.5 \times 432$

$= 1.1 \times 266 = 292.6$ PSF $= 212.8 - 216 = -3.2$ PSF

∴ Design Wind Effect $= W_w = 292.6$ PSF

On Roof we shall neglect the wind effect as it will act against the Tornado missile.

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SUBJECT TORNAD MISSILE PROTECTION

COMBINATION OF LOADS

1) Wall

Only Wind shall act perpendicular to wall.

$U = 292.6$ PSF

$= 2.03$ PSI

2) ROOF:

$U = D.L. + L.L$

(a) $U = 387.5 + 74 = 461.5$ PSF $= 3.20$ PSI

(b) $U = 537.5 + 74 = 611.5$ PSF $= 4.25$ PSI

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NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

Available Capacity Of Slab For Steel Pipe Missile.

(Capacity is governed by Flexure as indicated by Experimental Results.)

** – Not Calculated, see comment on Page #13

NO.	CRITICAL PANEL AREA: BUILDING	LOCATION	R_m KIPS	R_s KIPS	X_m in	X_e in	X_s in	$R'_m = R_m - R_s$ KIPS	$X'_m = X_m - X_s$ in	$X'_e = X_e - X_s$ in	$\mu_{all} = \frac{X'_m}{X'_e}$
1	PAB	Wall	1132	17.4	4.34	0.434	0.027	1114.6	4.313	0.407	10.6
		Roof	1132	3.36	0.41	0.041	3.84×10^{-4}	1128.64	4.096×10^{-1}	4.0616×10^{-2}	10.08
2	Penthouse (PAB)	Wall	1132	16	4.02	0.402	2.27×10^{-2}	1116	3.9973	3.793×10^{-1}	10.54
		Roof	904	2.66	0.25	0.025	2.25×10^{-4}	901.34	2.4978×10^{-1}	2.4775×10^{-2}	10.08
3	Control Bldg	Wall	1132	11.8	2.83	0.283	1.16×10^{-2}	1120.2	2.8184	2.714×10^{-1}	10.38
		Roof	904	2.89	0.28	0.028	2.71×10^{-4}	901.11	2.79729×10^{-1}	2.7729×10^{-2}	10.09
4	Diesel Generator Bldg.	Wall	1132	21.2	5.4	0.54	0.042	1110.8	5.358	4.98×10^{-1}	10.76
		Roof	904	1.49	0.13	0.013	5.86×10^{-5}	902.51	1.2994×10^{-1}	1.29414×10^{-2}	10.04
5	Fuel Storage Bldg.	Wall	2110	88.7	17	1.7	0.35	2021.3	16.65	1.35	12.33
		Roof	904	2.5	0.23	0.023	1.94×10^{-4}	901.5	2.2981×10^{-1}	2.2806×10^{-2}	10.08
6	Ventilation Area	Wall	904	9	1.9	0.19	6.91×10^{-3}	895	1.89309	1.8309×10^{-1}	10.34
		Roof	904	2.89	0.28	0.028	2.71×10^{-4}	901.11	2.7973×10^{-1}	2.7729×10^{-2}	10.09
7	Upper Elec. Tunnel	Roof at Grade	2153	9.04	2.4	0.24	3.6×10^{-3}	2143.96	2.3964	2.364×10^{-1}	10.14
8	Service Water Pumphouse	Wall	904	19	3.81	0.381	3.28×10^{-2}	885	3.7772	3.482×10^{-1}	10.85
		Roof	904	3.74	0.37	0.037	4.9×10^{-4}	900.26	3.6951×10^{-1}	3.651×10^{-2}	10.12
9	M.S & F.W Pipe Chase (East)	Piers	366	2.14	1.02	0.102	8.92×10^{-4}	363.86	1.019108	1.01108×10^{-1}	10.08
		Roof	721	**	**	**	**	721.	**	**	10.
		E-Wall	1090	**	**	**	**	1090	**	**	10.
		N-Wall	1090	**	**	**	**	1090	**	**	10.
10	M.S & F.W Pipe Chase (West)	E-Piers	366	2.14	1.02	0.102	8.92×10^{-4}	363.86	1.019108	1.01108×10^{-1}	10.08
		W-Piers	488	1.61	0.57	0.057	2.82×10^{-4}	486.39	5.69718×10^{-1}	5.6718×10^{-2}	10.04
		Roof(1)	721	**	**	**	**	721	**	**	10.
		Roof(2)	2153	5.32	2.4	0.24	1.75×10^{-3}	2147.68	2.39825	2.3825×10^{-1}	10.07
		N-Wall	1090	**	**	**	**	1090	**	**	10.

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NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

Available Capacity Of Slab For Steel Pipe Missile.

(Capacity is governed by Flexure as indicated by Experimental Results)

NO	CRITICAL PANEL AREA: BUILDING	LOCATION	R_m Kips	R_s Kips	X_m in	X_e in	X_s in	$R'_m = R_m - R_s$ Kips	$X'_m = X_m - X_s$ in	$X'_e = X_e - X_s$ in	$\mu_{all} = \frac{X'_m}{X'_e}$
11	Personal Hatch Area	Roof	2153	3.18	0.73	0.073	3.38×10^{-4}	2149.82	7.2966×10^{-1}	7.2662×10^{-2}	10.04
		N-wall	1720	15.1	6.69	0.669	2.4×10^{-2}	1704.9	6.6659	6.449×10^{-1}	10.34
12	Emergency Feed Water Bldg.	Roof	904	1.56	0.14	0.014	6.68×10^{-5}	902.44	1.3993×10^{-1}	1.3933×10^{-2}	10.04
		Wall	904	8.98	1.66	0.166	6.25×10^{-3}	895.02	1.65375	1.5975×10^{-1}	10.35
13	Control Bldg	Wall	904	3.35	0.55	0.055	6.78×10^{-4}	900.65	5.4933×10^{-1}	5.4322×10^{-2}	10.11
14	Diesel Generator Bldg.	Wall (N-S)	1090	**	**	**	**	1090	**	**	10.
		Wall at Stair	904	2.29	0.36	0.036	2.87×10^{-4}	901.71	3.5971×10^{-1}	3.5713×10^{-2}	10.07
		Roof Hatch	566	6.66	0.57	0.057	1.87×10^{-3}	559.34	5.6813×10^{-1}	5.513×10^{-2}	10.31
15	Fuel Storage Bldg	Wall	1310	8.98	2.41	0.241	6.25×10^{-3}	1301.02	2.40375	2.3475×10^{-1}	10.24
		Roof	904	2.5	0.23	0.023	1.95×10^{-4}	901.5	2.29805×10^{-1}	2.2805×10^{-2}	10.08
16	Service Water Pump House	E-Wall (1)	904	19	3.81	0.381	3.28×10^{-2}	885	3.7772	3.482×10^{-1}	10.85
		E-Wall (2)	904	8.54	1.57	0.157	5.58×10^{-3}	895.46	1.56442	1.5142×10^{-1}	10.33
		Roof Hatch	452	6.3	0.5	0.05	2.14×10^{-3}	445.7	4.9786×10^{-1}	4.786×10^{-2}	10.4
		Roof	904	2.34	0.22	0.022	1.67×10^{-4}	901.66	2.1983×10^{-1}	2.1833×10^{-2}	10.07
17	Hydrogen Recombiner Rm	Wall	1580	13.1	4.97	0.497	1.24×10^{-2}	1566.9	4.9576	4.846×10^{-1}	10.23
		Roof	2153	2.75	0.69	0.069	2.62×10^{-4}	2150.25	6.89738×10^{-1}	6.8738×10^{-2}	10.03
18	W.P.B. Tank Farm	W-Wall	1260	**	**	**	**	1260	**	**	10.
		E-Wall	721	**	**	**	**	721	**	**	10.
19	Emergency F. W. Bldg.	Missile Shield Wall at EL. +44	575	**	**	**	**	575	**	**	10
20	Deluxe valve Area.	Wall	1740	4.04	1.3	0.13	1.04×10^{-3}	1735.96	1.29896	1.2896×10^{-1}	10.07
		Roof	1740	4.14	0.8	0.08	6.15×10^{-4}	1735.86	7.99385×10^{-1}	7.9385×10^{-2}	10.07

** Not Calculated. See Comment in Page # 13

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NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

AVAILABLE CAPACITY OF SLAB FOR STEEL PIPE MISSILE

(Capacity is governed by Flexure as indicated by Experimental Results)

NO.	CRITICAL PANEL AREA		R_m Kips	R_s Kips	X_m in	X_e in	X_s in	$R_m' = R_m - R_s$ Kips	$X_m' = X_m - X_s$ in	$X_e' = X_e - X_s$ in	$\mu_{all} = \frac{X_m'}{X_e'}$
	BUILDING	LOCATION									
21	RHR & Cont. Spray Equip. Vault	Wall	1132	11.2	2.7	0.27	1.02×10^{-2}	1120.8	2.6898	2.598×10^{-1}	10.35
		Roof	904	3.11	0.22	0.022	2.22×10^{-4}	900.89	0.219778	2.1778×10^{-2}	10.09
22	Waste Processing Building Col. 1 & 2 & Col. A & D	Wall	904	9.64	2.	0.2	8.06×10^{-3}	894.36	1.99194	1.9194×10^{-1}	10.38
		Roof	904	2.66	0.25	0.025	2.25×10^{-4}	901.34	2.49775×10^{-1}	2.4775×10^{-2}	10.08

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NAME OF COMPANY SEABROOK UNIT/S.

SUBJECT TORNADO MISSILE PROTECTION

CAPACITY OF SLAB GOVERNED BY SHEAR FOR AUTOMOBILE

NO	CRITICAL PANEL AREA		R_m KIPS	X_e in	μ_{all}	$X_m = \mu_{all} X_e$	REMARK
	BUILDING	LOCATION					
7	Upper Electrical Tunnel	Roof at Grade	1453.87	$\frac{0.24 \times 1453.87}{2153} = 0.16$	1.3	0.208	
11	Personal Hatch Area.	Roof	1453.87	0.049	1.3	0.064	
17	Hydrogen Recombiner Rm.	Roof	1453.87	0.047	1.3	0.061	
10	M.S. & F.W Fire Case (west)	Roof (2)	1453.87	0.16	1.3	0.208	

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NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

AVAILABLE CAPACITY OF SLAB FOR AUTOMOBILE GOVERNED BY SHEAR CAPACITY.

NO.	CRITICAL PANEL AREA: BUILDING	LOCATION	R_m Kips	R_S Kips	X_m in	X_e in	X_s in	$R_m' = R_m - R_S$ Kips	$X_m' = X_m - X_s$ in	$X_e' = X_e - X_s$ in	$\mu_{all}' = \frac{X_m'}{X_e'}$
7	Upper Elec. Tunnel	Roof at Grade	1453.87	9.04	0.208	0.16	3.6×10^{-3}	1444.83	0.2044	0.1564	1.31
11	Personal Hatch Area.	Roof	,,	3.18	0.064	0.049	3.38×10^{-4}	1450.69	6.37×10^{-2}	0.048662	1.31
17	Hydrogen Recombiner Rm.	Roof	,,	2.75	0.061	0.047	2.62×10^{-4}	1451.12	0.060738	0.046738	1.3
10	M.S & F.W Pipe Chase (West)	Roof (2)	,,	5.32	0.208	0.16	1.75×10^{-3}	1448.55	0.20625	0.15825	1.3

NOTE:- The remainder areas are governed by Flexural Capacity and the corresponding Values are Same as calculated Before.

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NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

CAPACITY OF SLAB GOVERNED EITHER BY SHEAR OR FLEXURE FOR WOOD POLE MISSILE.

** - Not Calculated, See Comment on Page # 13.

NO.	CRITICAL PANEL AREAS BUILDING	LOCATION	R_m Kips	X_e in	μ_{all}	$X_m = \mu_{all} X_e$	Remark
1	PAB	Wall	498.74	$\frac{0.434}{1132} \times 498.74 = 0.191$	1.3	0.249	Shear governs.
		Roof	"	0.018	"	0.023	"
2	Pent House (PAB)	Wall	"	0.18	"	0.23	"
		Roof	"	0.014	"	0.018	"
3	Control Bldg.	Wall	"	0.125	"	0.163	
		Roof	"	0.015	"	0.02	"
4	Diesel Generator Bldg	Wall	"	0.238	"	0.31	"
		Roof	"	0.0072	"	0.009	"
5	Fuel Storage Bldg.	Wall	1056.15	0.851	"	1.106	"
		Roof	498.74	0.0127	"	0.0165	"
6	Ventilation Area.	Wall	"	0.105	"	0.136	"
		Roof	"	0.015	"	0.02	"
7	Upper Elec. Tunnel	Roof at Grade	"	0.056	"	0.07	"
8	Service Water Pump & Service	Wall	"	0.21	"	0.273	"
		Roof	"	0.020	"	0.027	"
9	M.S.+F.W Pipe Chase (East)	Piers	366	0.102	10	1.02	Flexure governs
		Roof	498.74	**	1.3	**	Shear governs.
		E-wall	"	**	"	**	"
		N-Wall	"	**	"	**	"
10	M.S+F.W Pipe Chase (West)	E-Piers	366	0.102	10	1.02	Flexure Governs
		W-Piers	488	0.057	10	0.57	"
		Roof(1)	"	**	1.3	**	Shear governs.
		Roof(2)	498.74	0.056	1.3	0.07	Shear governs
		N-Wall	"	**	1.3	**	Shear governs.

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SUBJECT TORNADO MISSILE PROTECTION

CAPACITY OF SLAB GOVERNED EITHER BY SHEAR OR FLEXURE FOR WOOD POLE MISSILE.

NO.	CRITICAL PANEL AREAS: BUILDING	CRITICAL PANEL AREAS: LOCATION	R_m Kips	X_e in	μ_{all}	$X_m = \mu_{all} X_e$	Remark.
11	Personal Hatch Area.	Roof	498.74	0.017	1.3	0.022	Shear governs.
		N-Wall	"	0.194	"	0.252	"
12	Emergency Feed Water Bldg.	Roof	"	0.0077	"	0.01	"
		Wall.	"	0.092	"	0.119	"
13	Control Bldg.	Wall	"	0.03	"	0.039	"
14	Diesel Generator Bldg.	Wall (nms)	"	**	"	**	"
		Wall at Stair	"	0.02	1.3	0.026	Shear governs
		Roof Hatch	566	0.057	10	0.57	Flexure Governs
15	Fuel Storage Bldg.	Wall	498.74	0.092	1.3	0.119	Shear Governs
		Roof	"	0.013	"	0.0169	"
16	Service Water Pump House	E-Wall (1)	"	0.21	"	0.273	"
		E-Wall (2)	"	0.087	"	0.113	"
		Roof Hatch	452	0.05	10	0.5	Flexure Governs
		Roof	498.74	0.012	1.3	0.016	Shear Governs
17	Hydrogen Recombiner Rm	Wall	"	0.157	"	0.204	"
		Roof.	"	0.016	"	0.021	"
18	W.P.B. Tank Farm	W-Wall	"	**	"	**	"
		E-Wall	"	**	"	**	"
19	Emergency Feed Water Bldg.	Missile Shield wall at EL.+44	575	**	10	**	Flexure Governs
20	Deluge Valve Area.	Wall	498.74	0.037	1.3	0.0481	Shear Governs
		Roof	"	0.023	"	0.03	"

** Not Calculated, See Comment on Page # 13

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SUBJECT TORNADO MISSILE PROTECTION

CAPACITY OF SLAB GOVERNED EITHER BY SHEAR OR FLEXURE FOR WOOD POLE MISSILE.

NO.	CRITICAL PANEL AREAS		R_m Kips	X_e in	μ_{all}	$X_m = \mu_{all} X_e$	Remark
	BUILDING	LOCATION					
21	RHR & Cont. Spray Equip. Vault	Wall	498.74	0.12	1.3	0.156	Shear Governs
		Roof	"	0.012	"	0.0156	"
22	Waste Processing Building Col. 1 & 2 Col. A & D	Wall	"	0.11	"	0.143	"
		Roof	"	0.014	"	0.0182	"

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united engineers & constructors inc.

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J.O. 9763.006	

REV	COMP. BY	CHK'D BY
0	DXm DATE 4/3/79	RHT DATE 5/7/79
	DATE	DATE

NAME OF COMPANY SEABOOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION.

AVAILABLE CAPACITY OF SLAB FOR WOOD POLE MISSILE.

(GOVERNED EITHER BY SHEAR OR FLEXURE.)

** - Not Calculated, See Comment on Page # 13.

NO.	CRITICAL PANEL AREAS: BUILDING	LOCATION	R_m Kips	R_s Kips	X_m in	X_e in	X_s in	$R'_m=R_m-R_s$ Kips	$X'_m=X_m-X_s$ in	$X'_e=X_e-X_s$ in	$\mu_{all}=\frac{X'_m}{X'_e}$
1	PAB	Wall	498.74	17.4	0.249	0.191	0.027	481.34	2.22×10^{-1}	1.64×10^{-1}	1.35
		Roof	"	3.36	0.023	0.018	3.84×10^{-4}	495.38	2.2616×10^{-2}	1.7616×10^{-2}	1.28
2	Pent House (PAB)	Wall	"	16	0.23	0.18	2.27×10^{-2}	482.74	2.073×10^{-1}	1.573×10^{-1}	1.32
		Roof	"	2.66	0.018	0.014	2.25×10^{-4}	496.08	1.7775×10^{-2}	1.3775×10^{-2}	1.29
3	Control Bldg.	Wall	"	11.8	0.163	0.125	1.16×10^{-2}	486.94	1.514×10^{-1}	1.134×10^{-1}	1.34
		Roof	"	2.89	0.02	0.015	2.71×10^{-4}	495.85	1.9729×10^{-2}	1.4729×10^{-2}	1.34
4	Diesel Generator Bldg	Wall	"	21.2	0.31	0.238	0.042	477.54	2.68×10^{-1}	1.96×10^{-1}	1.37
		Roof	"	1.49	0.009	0.0072	5.86×10^{-5}	497.25	8.9414×10^{-3}	7.1414×10^{-3}	1.25
5	Fuel Storage Bldg.	Wall	1056.15	88.7	1.106	0.851	0.35	967.45	7.56×10^{-1}	5.01×10^{-1}	1.51
		Roof	498.74	2.5	0.0165	0.0127	1.94×10^{-4}	496.24	1.6306×10^{-2}	1.2506×10^{-2}	1.3
6	Ventilation Area	Wall	"	9	0.136	0.105	6.91×10^{-3}	489.74	1.2909×10^{-1}	9.809×10^{-2}	1.32
		Roof	"	2.89	0.02	0.015	2.71×10^{-4}	495.85	1.9729×10^{-2}	1.4729×10^{-2}	1.34
7	Upper Elec. Tunnel	Roof at Grade	"	9.04	0.07	0.056	3.6×10^{-3}	489.7	6.64×10^{-2}	5.24×10^{-2}	1.27
8	Service Water Pump Service	Wall	"	19	0.273	0.21	3.28×10^{-2}	479.74	2.402×10^{-1}	1.772×10^{-1}	1.36
		Roof	"	3.74	0.027	0.02	4.9×10^{-4}	495	2.651×10^{-2}	1.951×10^{-2}	1.36
9	M.S+F.W Pipe Chase (East)	Piers	366	2.14	1.02	0.102	8.92×10^{-4}	363.86	1.019108	1.01108×10^{-1}	10.08
		Roof	498.74	**	**	**	**	498.74	**	**	1.3
		E-Wall	"	**	**	**	**	"	**	**	1.3
		N-Wall	"	**	**	**	**	"	**	**	1.3
10	M.S+F.W Pipe Chase (West)	E-Piers	366	2.14	1.02	0.102	8.92×10^{-4}	363.86	1.019108	1.01108×10^{-1}	10.08
		W-Piers	488.0	1.61	0.57	0.057	2.82×10^{-4}	486.39	5.69718×10^{-1}	5.6718×10^{-2}	10.04
		Roof(1)	498.74	**	**	**	**	498.74	**	**	1.3
		Roof(2)	498.74	5.32	0.07	0.056	1.75×10^{-3}	493.42	6.825×10^{-2}	5.425×10^{-2}	1.26
		N-Wall	498.74	**	**	**	**	498.74	**	**	1.3

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0	DXW. DATE 4/3/79	RHT DATE 5/17/79
	DATE	DATE

NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION.

AVAILABLE CAPACITY OF SLAB FOR WOOD POLE MISSILE.

(GOVERNED EITHER BY SHEAR OR FLEXURE.)

NO.	CRITICAL PANEL AREAS: BUILDING	LOCATION	R_m Kips	R_s Kips	X_m in	X_e in	X_s in	$R'_m = R_m - R_s$ Kips	$X'_m = X_m - X_s$ in	$X'_e = X_e - X_s$ in	$\mu'_{all} = \frac{X'_m}{X_e}$
11	Personal Hatch Area	Roof	498.74	3.18	0.022	0.017	3.38×10^{-4}	495.56	2.166×10^{-2}	1.666×10^{-2}	1.3
		N-Wall	"	15.1	0.252	0.194	2.41×10^{-2}	483.64	2.279×10^{-1}	0.1699	1.34
12	Emergency Feed Water Bldg.	Roof	"	1.56	0.01	0.0077	6.68×10^{-5}	497.18	9.933×10^{-3}	7.633×10^{-3}	1.3
		Wall	"	8.98	0.119	0.092	6.25×10^{-3}	489.76	0.11275	0.08575	1.31
13	Control Bldg.	Wall	"	3.35	0.039	0.03	6.76×10^{-4}	495.39	3.832×10^{-2}	2.932×10^{-2}	1.31
14	Diesel Generator Bldg.	Wall (N=3)	498.74	**	**	**	**	498.74	**	**	1.3
		Wall at Stair	498.74	2.29	0.026	0.02	2.87×10^{-4}	496.45	2.571×10^{-2}	1.971×10^{-2}	1.3
		Roof Hatch	566	6.66	0.57	0.057	1.87×10^{-3}	559.34	5.6813×10^{-1}	5.513×10^{-2}	10.31
15	Fuel Storage Bldg.	Wall	498.74	8.98	0.119	0.092	6.25×10^{-3}	489.76	1.1275×10^{-1}	8.575×10^{-2}	1.31
		Roof	"	2.50	0.0169	0.013	1.95×10^{-4}	496.24	1.6705×10^{-2}	1.2805×10^{-2}	1.30
16	Service Water Pump House	E-Wall (1)	"	19	0.273	0.21	3.28×10^{-2}	479.74	2.402×10^{-1}	1.772×10^{-1}	1.36
		E-Wall (2)	"	8.54	0.113	0.087	5.58×10^{-3}	490.2	1.074×10^{-1}	8.142×10^{-2}	1.32
		Roof Hatch	452	6.3	0.5	0.05	2.14×10^{-3}	445.7	4.9786×10^{-1}	4.786×10^{-2}	10.40
		Roof	498.74	2.34	0.016	0.012	1.67×10^{-4}	496.4	1.5833×10^{-2}	1.1833×10^{-2}	1.34
17	Hydrogen Recombiner Rm	Wall	"	13.1	0.204	0.157	1.24×10^{-2}	485.64	1.916×10^{-1}	1.446×10^{-1}	1.33
		Roof	"	2.75	0.021	0.016	2.62×10^{-4}	495.99	2.0738×10^{-2}	1.5738×10^{-2}	1.32

** - Not Calculated, See comment on Page #13

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NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

AVAILABLE CAPACITY OF SLAB FOR WOOD POLE MISSILE.

(Governed Either By Shear Or Flexure.)

NO.	Critical Panel Areas: Building	Critical Panel Areas: Location	R_m Kips	R_s Kips	X_m in	X_e in	X_s in	$R_m' = R_m - R_s$ Kips	$X_m' = X_m - X_s$ in	$X_e' = X_e - X_s$	$\mu_{all}' = \frac{X_m'}{X_e'}$
18	W.P.B Tank Farm	W-Wall	498.74	**	**	**	**	498.74	**	**	1.3
		E-Wall	498.74	**	**	**	**	498.74	**	**	1.3
19	Emergency Feed Water Bldg.	Missile Shield wall at El.+44	575	**	**	**	**	575	**	**	10.
20	Deludge Valve Area.	Wall	498.74	4.04	0.0481	0.037	1.04×10^{-3}	494.7	4.706×10^{-2}	3.596×10^{-2}	1.31
		Roof	498.74	4.14	0.03	0.023	6.15×10^{-4}	494.6	2.9385×10^{-2}	2.2385×10^{-2}	1.31
21	RHR & Cont. Spray Equip Vault	Wall	498.74	11.2	0.156	0.12	1.02×10^{-2}	487.54	1.458×10^{-1}	1.098×10^{-1}	1.33
		Roof	498.74	3.11	0.0156	0.012	2.22×10^{-4}	495.63	1.5378×10^{-2}	1.1778×10^{-2}	1.31
22	Waste Processing Building Col. 1 & 2 col A & D	Wall	498.74	9.64	0.143	0.11	8.06×10^{-3}	489.1	1.3494×10^{-1}	1.0194×10^{-1}	1.32
		Roof	498.74	2.66	0.0182	0.014	2.25×10^{-4}	496.08	1.7975×10^{-2}	1.3775×10^{-2}	1.3

** Not Calculated, See Comment on Page # 13

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NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

E) RESPONSE OF 12"Φ X 15' STEEL PIPE MISSILE ON SLAB.

NO.	CRITICAL PANEL AREAS: BUILDING	LOCATION	R'_m KIPS	F_1 KIPS	T SECS.	t_d SECS.	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #1 FOR C_T & C_R: $X_m/X'_e = \bar{\mu}$	X'_e in	X_m in	Remark
1	PAB	Wall	1114.6	742.63	0.0468	0.00656	0.14	1.5	0.55			ELASTIC
		Roof	1128.64	581.5	0.00443	0.00671	1.51	1.94	1.1			ELASTIC SLIGHTLY PLASTIC
2	Pent-House (PAB)	Wall	1116	742.63	0.03939	0.00656	0.17	1.5	0.65			ELASTIC
		Roof	901.34	581.5	0.003392	0.00671	1.98	1.55	1.5			ELASTIC SLIGHTLY PLASTIC
3	Control Bldg	Wall	1120.2	742.63	0.03053	0.00656	0.21	1.51	0.75			ELASTIC
		Roof	901.11	581.5	0.003723	0.00671	1.8	1.55	1.5			ELASTIC SLIGHTLY PLASTIC
4	Diesel Generator Bldg	Wall	1110.8	742.63	0.05823	0.00656	0.11	1.5	0.43			ELASTIC
		Roof	902.51	581.5	0.001731	0.00671	3.88	1.55	1.5			ELASTIC SLIGHTLY PLASTIC
5	Fuel Storage Bldg	Wall	2021.3	742.63	0.18367	0.00656	0.04	2.72	<<1			ELASTIC
		Roof	901.5	581.5	0.003154	0.00671	2.13	1.55	1.5			ELASTIC SLIGHTLY PLASTIC
6	Ventilation Area	Wall	895	742.63	0.02135	0.00656	0.31	1.21	1.4			"
		Roof	901.11	581.5	0.003723	0.00671	1.8	1.55	1.5			"
7	Upper Electric Tunnel	Roof at Grade	2143.96	581.5	0.013566	0.00671	0.5	3.69	<<1			ELASTIC

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NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

E) RESPONSE OF 12"ϕ X 15' STEEL PIPE MISSILE ON SLAB.

NO.	CRITICAL PANEL AREAS: BUILDING	LOCATION	R'_m Kips	F_1 Kips	T SECS.	t_d SECS.	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #1 FOR C_T & C_R $x_m/x'_e = \bar{\mu}$	X'_e in	x_m in	Remark
8	Service Water Pump House	Wall	885	742.63	0.05141	0.00656	0.13	1.19	0.62			ELASTIC
		Roof	900.26	581.5	0.005	0.00671	1.34	1.55	1.5			ELASTIC SLIGHTLY PLASTIC
9	M.S & F.W Pipe Chase (East)	Piers	363.86	742.63	0.00831	0.00656	0.8	0.49	55			FAILURE
		Roof	721	581.5	0.00979	0.00671	0.69	1.24	2.45			ELASTIC PLASTIC
		E-Wall	1090	742.63	0.3172	0.00656	0.02	1.47	<<1			ELASTIC
		N-Wall	1090	742.63	0.01573	0.00656	0.42	1.47	1.45			ELASTIC SLIGHTLY PLASTIC
10	M.S & F.W Pipe Chase (West)	E-Pier	363.86	742.63	0.00831	0.00656	0.8	0.49	55			FAILURE
		W-Piers	486.39	742.63	0.004676	0.00656	1.40	0.65	36			FAILURE
		Roof(1)	721	581.5	0.006796	0.00671	0.99	1.24	2.9			ELASTIC PLASTIC
		Roof (2)	2147.68	581.5	0.008434	0.00671	0.8	3.69	<<1			ELASTIC
		N-Wall	1090	742.63	0.01209	0.00656	0.54	1.47	1.5			ELASTIC SLIGHTLY PLASTIC
11	Personal Hatch Area	Roof	2149.82	581.5	0.004157	0.00671	1.61	3.7	<<1			ELASTIC
		Wall (N)	1704.9	742.63	0.03943	0.00656	0.17	2.3	0.5			ELASTIC

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SUBJECT TORNADO MISSILE PROTECTION

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E) RESPONSE OF 12"Φ×15' STEEL PIPE MISSILE ON SLAB.

NO.	CRITICAL PANEL AREAS		R'_m	F_1	T	t_d	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #1 FOR C_T & C_R	X'_e	X_m	Remark
	BUILDING	LOCATION	Kips	Kips	SECS.	SECS.			$X_m/X'_e = \bar{\mu}$	in	in	
12	Emergency Feed Water Bldg.	Roof	902.44	581.5	0.001848	0.00671	3.63	1.55	1.5			ELASTIC SLIGHTLY PLASTIC
		Wall	895.02	742.63	0.022432	0.00656	0.29	1.21	1.45			"
13	Control Bldg.	Wall	900.65	742.63	0.007393	0.00656	0.89	1.21	2.9			ELASTIC PLASTIC
14	Diesel Generator Bldg.	Wall (N & S)	1090	742.63	0.04391	0.00656	0.15	1.47	0.6			ELASTIC
		Wall at Stair	901.71	742.63	0.00481	0.00656	1.36	1.21	3			ELASTIC PLASTIC
		Roof Hatch	559.34	581.5	0.004792	0.00671	1.4	0.96	9			?? Borderline.
15	Fuel Storage Bldg	Wall	1301.02	742.63	0.02243	0.00656	0.29	1.75	<1			ELASTIC
		Roof	901.5	581.5	0.00315	0.00671	2.13	1.55	1.5			ELASTIC PLASTIC
16	Service Water Pump House	E-Wall (1)	885	742.63	0.05141	0.00656	0.13	1.19	0.68			ELASTIC
		E-Wall (2)	895.46	742.63	0.0212	0.00656	0.31	1.21	1.45			ELASTIC PLASTIC
		Roof Hatch	445.7	581.5	0.00521	0.00671	1.29	0.77	19			FAILURE
		Roof	901.66	581.5	0.002925	0.00671	2.29	1.55	1.5			ELASTIC PLASTIC

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SUBJECT TORNADO MISSILE PROTECTION

E.) RESPONSE OF 12"Φ x 15' STEEL PIPE MISSILE ON SLAB.

NO.	CRITICAL PANEL AREAS: BUILDING	CRITICAL PANEL AREAS: LOCATION	R'_m Kips	F_1 Kips	T Secs.	t_d Secs.	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #1 FOR C_T & C_R: $x_m/x'_e \bar{\mu}$	X'_e in	x_m in	Remark
17	Hydrogen Recombiner Rm.	Wall	1566.9	742.63	0.01828	0.00656	0.36	2.11	0.86			ELASTIC
		Roof	2150.25	581.5	0.003303	0.00671	2.03	3.7	< 1			"
18	W.P.B Tank Farm.	West Wall	1260	742.63	.1643	0.00656	0.04	1.7	<< 1			R_m' taken from [illegible] ELASTIC
		East Wall	721	742.63	0.0098	0.00656	0.67	.97	4.2			R_m' taken from [illegible]
19	Emergency Feed Water Bldg.	Missile Shield wall at EL. 44	575	742.63	0.00475	0.00656	1.38	.774	> 10			R_m' taken from [illegible] FAILURE
20	Deluge Valve Area	Wall	1735.96	742.63	0.00915	0.00656	0.72	2.34	< 1			Elastic
		Roof	1735.86	581.5	0.00561	0.00671	1.17	2.99	< 1			"
21	RHR & Cont. Spray Equipment Vault	Wall	1120.8	742.63	.0286	0.00656	0.24	1.51	≈ 1			"
		Roof	900.89	581.5	0.00293	0.00671	2.29	1.55	1.5			Elastic plastic
22	Waste Processing Building Col. 1 & 2 + Col. A & D	Wall	894.36	742.63	0.023	0.00656	0.29	1.20	1.45			Elastic plastic
		Roof	901.34	581.5	0.0034	0.00671	1.97	1.55	1.5			Elastic plastic

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G.) RESPONSE OF AUTOMOBILE MISSILE ON SLAB.

NO.	CRITICAL PANEL AREAS BUILDING	LOCATION	R'_m Kips	F_1 Kips	T SECS.	t_d SECS.	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #2 FOR C_T & C_R $x_m/X'_e\bar{\mu}$	X'_e in	x_m in	Remark
1	PAB	Wall	1114.6	265	0.0468	0.07853	1.68	4.21	<<1			ELASTIC
		Roof	1128.64	210	0.00443	"	17.73	5.37	<<1			"
2	Pent-House (PAB)	Wall	1116	265	0.03939	"	1.99	4.21	<<1			"
		Roof	901.34	210	0.003392	"	23.15	4.29	<<1			"
3	Control Bldg.	Wall	1120.2	265	0.03053	"	2.57	4.23	<<1			"
		Roof	901.11	210	0.003723	"	21.09	4.29	<<1			"
4	Diesel Generator Bldg.	Wall	1110.8	265	0.05823	"	1.35	4.19	<<1			"
		Roof	902.51	210	0.001731	"	45.37	4.3	<<1			"
5	Fuel Storage Bldg.	Wall	2021.3	265	0.18367	"	0.43	7.63	<<1			"
		Roof	901.5	210	0.003154	"	24.9	4.29	<<1			"
6	Ventilation Area	Wall	895	265	0.02135	"	3.68	3.38	<<1			"
		Roof	901.11	210	0.003723	"	21.09	4.29	<<1			"
7	Upper Elec. Tunnel	Roof at Grade.	1451.12	210	0.013566	"	5.79	6.91	<<1			" $\mu'_{all} = 1.31$

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0	[illegible] DATE 3/21/79	[illegible] DATE 5/1/79
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NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

G.) RESPONSE OF AUTOMOBILE MISSILE ON SLAB.

NO.	CRITICAL PANEL AREAS		R'_m	F_1	T	t_d	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #2 FOR C_T & C_R	X'_e	x_m	Remark
	BUILDING	LOCATION	Kips		SECS.	SECS.			$x_{m}/X'_e=\bar{\mu}$	in	in	
8	Service Water Pump House	Wall	885	265	0.05141	0.07853	1.53	3.34	<<1			ELASTIC
		Roof	900.26	210	0.005	"	15.71	4.29	<<1			"
9	M.S. & F.W. Pipe Chase (East)	Piers	363.86	265	0.00831	"	9.45	1.373	~.75			"
		Roof	721	210	0.00979	"	8.02	3.43	<<1			"
		E-Wall	1090	265	0.3172	"	0.248	4.11	<<1			"
		N-Wall	1090	265	0.01573	"	4.99	4.11	<<1			"
10	M.S. & F.W Pipe Chase (West)	E-Pier	363.86	265	0.00831	"	9.45	1.373	~.75			"
		W-Pier	486.39	265	0.004676	"	16.79	1.835	<.6			"
		Roof (1)	721	210	0.006796	"	11.56	3.43	<<1			"
		Roof (2)	1448.55	210	0.008434	"	9.31	6.9	<<1			" $\mu_{ALL} = 1.3$
		N-Wall	1090	265	0.01209	"	6.5	4.11	<<1			"
11	Personnel Hatch Area.	Roof	1458.69	210	0.004157	"	18.89	6.91	<<1			" $\mu_{ALL} = 1.31$
		Wall (N)	1704.9	265	0.03943	"	1.99	6.43	<<1			" $\mu_{ALL} = 10.34$

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SUBJECT TORNADO MISSILE PROTECTION

G.) RESPONSE OF AUTOMOBILE MISSILE ON SLAB.

NO.	CRITICAL PANEL AREAS		R'_m	F_1	T	t_d	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #2 FOR C_T & C_R	X'_e	X_m	Remark
	BUILDING	LOCATION	Kips		SECS.	SECS.			$X_m/X'_e=\bar{\mu}$	in	in	
12	Emergency Feed Water Bldg.	Roof	902.44	210	0.001848	0.07853	42.49	4.3	<<1			ELASTIC
		Wall	895.02	265	0.022432	"	3.5	3.38	<<1			"
13	Control Bldg.	Wall	900.65	265	0.007393	"	19.98	3.4	<<1			"
14	Diesel Generator Bldg.	Wall (No. S)	1090	265	0.04391	"	1.79	4.11	<<1			"
		Wall at Stair	901.71	265	0.00481	"	16.33	3.40	<<1			"
		Roof Hatch	559.34	210	0.004794	"	16.38	2.66	<<1			"
15	Fuel Storage Bldg.	Wall	1301.02	265	0.02243	"	3.5	4.91	<<1			"
		Roof	449.35	210	0.00196	"	40.07	2.14	<<1			"
16	Service Water Pump House	E-Wall (1)	885	265	0.5141	"	0.15	3.34	<<1			"
		E-Wall (2)	895.46	265	0.0212	"	3.7	3.38	<<1			"
		Roof Hatch	445.7	210	0.00521	"	15.07	2.12	<<1			"
		Roof	901.66	210	0.002925	"	26.85	4.29	<<1			"

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G.) RESPONSE OF AUTOMOBILE MISSILE ON SLAB.

NO.	CRITICAL PANEL AREAS		R'_m	F_1	T	t_d	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #2 FOR C_T & C_R	X'_e	X_m	Remark
	BUILDING	LOCATION	Kips	Kips	SECS.	SECS.			X_m/X'_e	in	in	
17	Hydrogen Recombiner Rm.	Wall	1566.9	265	0.01828	0.07853	4.3	5.91	<<1			ELASTIC μ'_{all}=10.23
		Roof	1451.12	210	0.003303	"	23.78	6.91	<<1			" μ'_{all}=1.3
18	W.P.B Tank Farm	W-Wall	1260	265	0.1643	"	0.48	4.75	<<1			ELASTIC R'_m Taken as R_m
		E-Wall	721	265	0.0098	"	8.01	2.72	<<1			ELASTIC R'_m Taken as R_m
19	Emergency F.W. Bldg.	Missile Shield wall at El. +44	575	265	0.00475	"	16.53	2.17	<1			ELASTIC R'_m Taken as R_m
20	Deluxe Valve Area	Wall	1735.96	265	0.00915	0.07853	8.58	6.55	<<1			Elastic μ'_{all}=10.07
		Roof	1735.96	210	0.00561	"	14	8.27	<<1			Elastic μ'_{all}=10.07
21	RHR & Cont. Spray Equip. Vault	Wall	1120.8	265	0.0286	"	2.75	4.23	<<1			Elastic μ'_{all}=10.35
		Roof	900.89	210	0.00293	"	26.8	4.29	<<1			Elastic μ'_{all}=10.09
22	Waste Processing Building col. 1 & 2 col. A & D	Wall	894.36	265	0.023	"	3.41	3.37	<<1			Elastic μ'_{all}=10.38
		Roof	901.34	210	0.0034	"	23.1	4.29	<<1			Elastic μ'_{all}=10.08

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F.) RESPONSE OF WOOD POLE MISSILE ON SLAB.

NO.	CRITICAL PANEL AREAS		R'_m	F_1	T	t_d	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #1 FOR C_T & C_R	X'_e	x_m	Remark
	BUILDING	LOCATION	Kips	Kips	SECS.	SECS.			$x_m/X'_e = \mu$	in	in	
1	PAB	Wall	481.34	202.5	0.0468	0.054	1.15	2.38	< 1			ELASTIC $\mu'_{all} = 1.35$
		Roof	495.38	192	0.00443	0.044	9.93	2.58	< 1			" $\mu'_{all} = 1.28$
2	Pent-House (PAB)	Wall	482.74	202.5	0.03939	0.054	1.37	2.38	< 1			" $\mu'_{all} = 1.37$
		Roof	496.08	192	0.003392	0.044	12.97	2.58	< 1			" $\mu'_{all} = 1.29$
3	Control Bldg.	Wall	486.94	202.5	0.03053	0.054	1.77	2.40	< 1			" $\mu'_{all} = 1.34$
		Roof	495.85	192	0.003723	0.044	11.82	2.58	< 1			" $\mu'_{all} = 1.36$
4	Diesel Generator Bldg.	Wall	477.54	202.5	0.05823	0.054	0.93	2.36	< 1			" $\mu'_{all} = 1.37$
		Roof	497.25	192	0.001731	0.044	25.42	2.59	< 1			" $\mu'_{all} = 1.25$
5	Fuel Storage Bldg.	Wall	967.45	202.5	0.18367	0.054	0.29	4.78	<< 1			" $\mu'_{all} = 1.51$
		Roof	496.24	192	0.003154	0.044	13.95	2.58	< 1			" $\mu'_{all} = 1.3$
6	Ventilation Area	Wall	489.74	202.5	0.02135	0.054	2.53	2.42	< 1			" $\mu'_{all} = 1.32$
		Roof	495.85	192	0.003723	0.044	11.82	2.58	< 1			" $\mu'_{all} = 1.34$
7	Upper Electrical Tunnel	Roof at Grade	489.7	192	0.013566	0.044	3.24	2.55	< 1			" $\mu'_{all} = 1.27$

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F.) RESPONSE OF WOOD POLE MISSILE ON SLAB.

NO.	CRITICAL PANEL AREAS		R'_m	F_1	T	t_d	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #1 FOR C_T & C_R	X'_e	x_m	Remark
	BUILDING	LOCATION	Kips	Kips	SECS.	SECS.			$x_m/X'_e = \bar{\mu}$	in	in	
8	Service Water Pump House	Wall	479.74	202.5	0.05141	0.054	1.05	2.37	<1			ELASTIC $\mu'_{all} = 1.36$
		Roof	495	192	0.005	0.044	8.8	2.58	<1			" $\mu'_{all} = 1.36$
9	M.S + F.W Pipe Chase (East)	Piers	363.86	202.5	0.00831	0.054	6.5	1.8	1.1			ELASTIC SLIGHTLY PLASTIC $\mu'_{all} = 10.08$
		Roof	498.74	192	0.00979	0.044	4.49	2.6	<1			ELASTIC
		E-Wall	498.74	202.5	0.3172	0.054	0.17	2.46	<1			"
		N-Wall	498.74	"	0.01573	"	3.43	2.46	<1			"
10	M.S + F.W Pipe Chase (West)	E-Pier	363.86	"	0.00831	"	6.5	1.8	1.1			ELASTIC SLIGHTLY PLASTIC $\mu'_{all} = 10.08$
		W-Pier	486.39	"	0.004676	"	11.55	2.40	<1			ELASTIC $\mu'_{all} = 10.4$
		Roof (1)	498.74	192	0.006796	0.044	6.47	2.60	<1			" $\mu'_{all} =$
		Roof (2)	493.42	"	0.008434	"	5.22	2.57	<1			" $\mu'_{all} = 1.26$
		N-Wall	498.74	202.5	0.01209	0.054	4.47	2.60	<1			"
11	Personal Hatch Area	Roof	495.56	192	0.004157	0.044	10.58	2.58	<1			" $\mu'_{all} = 1.3$
		Wall (N)	483.64	202.5	0.03943	0.054	1.37	2.39	<1			" $\mu'_{all} = 1.34$

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F.) RESPONSE OF WOOD POLE MISSILE ON SLAB.

NO.	CRITICAL PANEL AREAS BUILDING	CRITICAL PANEL AREAS LOCATION	R'_m Kip	F_1 Kips	T Secs.	t_d Secs.	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #1 FOR C_T & C_R $x_m/X'_e = \mu$	X'_e in	x_m in	Remark
12	Emergency Feed Water Bldg.	Roof	497.18	192	0.001848	0.044	23.81	2.59	<1			ELASTIC $\mu'_{max} = 1.3$
		Wall.	487.76	202.5	0.022432	0.054	2.41	2.41	<1			" $\mu'_{max} = 1.31$
13	Control Bldg.	Wall	495.39	"	0.007393	"	7.3	2.45	<1			" $\mu'_{max} = 1.31$
14	Diesel Generator Bldg.	Wall (N or S)	498.74	"	0.04391	"	1.23	2.6	<1			"
		Wall at Stair	496.45	"	0.00481	"	11.23	2.45	<1			" $\mu'_{max} = 1.3$
		Roof Hatch	559.34	192	0.004794	0.044	9.18	2.91	<1			" $\mu'_{max} = 10.31$
15	Fuel Storage Bldg.	Wall	489.76	202.5	0.02243	0.054	2.41	2.42	<1			" $\mu'_{max} = 1.31$
		Roof	496.09	192	0.00196	0.044	22.45	2.58	<1			" $\mu'_{max} = 1.3$
16	Service Water Pump House	E-Wall (1)	479.74	202.5	0.05141	0.054	1.05	2.37	<1			" $\mu'_{max} = 1.36$
		E-Wall (2)	490.2	"	0.0212	"	2.55	2.42	<1			" $\mu'_{max} = 1.32$
		Roof Hatch	445.7	192	0.00521	0.044	8.45	2.32	<1			" $\mu'_{max} = 10.4$
		Roof	496.4	192	0.002925	0.044	15.04	2.59	<1			" $\mu'_{max} = 1.34$

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SUBJECT TORNADO MISSILE PROTECTION

F.) Response Of Wood Pole Missile On Slab.

NO.	CRITICAL PANEL AREAS: Building	CRITICAL PANEL AREAS: Location	R'_m Kips	F_1 Kips	T SECS.	t_d SECS.	$C_T = t_d/T$	$C_R = R'_m/F_1$	USING Fig. #1 FOR C_T & C_R $x_m/X'_e\mu$	X'_e in	x_m in	Remark
17	Hydrogen Recombiner Rm.	Wall	485.64	202.5	0.01828	0.054	2.95	2.4	<1			ELASTIC $\mu'_{all} = 1.33$
		Roof	495.99	192	0.003303	0.044	13.32	2.58	<1			" $\mu'_{all} = 1.32$
18	W.P.B Tank Farm	W-Wall	498.74	202.5	.1643	0.054	0.33	2.46	<1			Rm' taken as Rm, μ' taken as 1.3 ELASTIC
		E-Wall	498.74	202.5	.0098	0.054	5.51	2.46	<1			Rm' taken as Rm, μ' taken as 1.3 ELASTIC
19	Emergency Feed Water Bldg	Missile Shield Wall at El. +40	575	202.5	.00475	0.054	11.37	2.85	<1			Rm' taken as Rm ELASTIC
20	Deluge Valve Area	Wall	494.7	202.5	.00915	0.054	5.90	2.44	<1			Elastic $\mu'_{all} = 1.31$
		Roof	494.6	192	.00561	0.044	7.84	2.58	<1			Elastic $\mu'_{all} = 1.31$
21	RHR & Cont. Spray Equipment Vault	Wall	487.54	202.5	.0286	0.054	1.89	2.41	<1			Elastic $\mu'_{all} = 1.33$
		Roof	495.63	192	.00293	0.044	15.02	2.58	<1			Elastic $\mu'_{all} = 1.31$
22	Waste Processing Building Col. 1 & 2 Col A & D	Wall	489.1	202.5	0.023	0.054	2.35	2.42	<1			Elastic $\mu'_{all} = 1.32$
		Roof	496.08	192	0.0034	0.044	12.94	2.58	<1			Elastic $\mu'_{all} = 1.3$

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CONCLUSION:-

From the response calculation of 12"Φ steel pipe, Automobile and Wood pole (page 69 to page 80), we observe that all most all but few are protected from the tornado generated missiles.

The areas where failure occured are piers of both east and west Main Steam and Feed water pipe chase, Roof Hatch of Service Water Pump Service, and Missile Sheild Wall at El.+44 of Emergency feed Water Building. Roof Hatch of Diesel Generator building is also found to be in very merginal safe condition. These failures are caused by 12"Φ steel pipe missile only.

All the cases, where we could not calculate R_s, X_e and X_s as indicated in page 13, the calculated ductility ratios are found to be well

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CONCLUSION:

Within the allowable ductility ratio. Therefore, no additional analysis is required to check their capacity.

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RECOMMENDATIONS:-

In order to protect the roof hatch of Diesel generater building, Service Water Pump House and the missile sheild wall of Emergency feed water building from failure, the reinforcement must be changed to at least #11 @16 O.C. top and bottom each way.

To protect the piers of East and West of M.S & F.W. pipe chase, The strength must be increased at least to 84% for both piers in East pipe chase and East piers of West pipe chase and to 53% for West piers of West pipe chase.

CONTINUATION OF SBSAG-1MA

1. REVISIONS TO DESIGNS NOTED AS INADEQUATE IN ORIGINAL ANALYSIS.

2. ADDITIONAL STRUCTURES NOT CONSIDERED IN ORIGINAL ANALYSIS

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The Design of Piers in M.S. & F.W. Pipe Chase (East) and (West) [item 9 & 10] are revised and following information are obtained from Calculation supplied from The Project per memo # 5107A, dated Oct. 4, 1979 from A.J. Hulshizer to R.H. Toland / D. Majumder.

Present ultimate Moment Capacity of Piers = $M_U = 1460$ ft-k

The Size of piers = 2'×3' with 16-#11 bars all around

Also, $d = 25.89"$, $t = 36"$, $b = 24"$

$A_s = 6.24 + 6.24 + 3.12 = 15.6"$ $A_s' = 9.36\ in^2$

$$\therefore \rho = \frac{A_s}{bd} = \frac{15.6}{24 \times 25.89} = 0.025$$

$$\rho' = \frac{A_s'}{bd} = \frac{9.36}{24 \times 25.89} =$$

$$\rho n = 9 \times 0.025 = 0.23$$

$$\rho'/\rho = \frac{9.36}{15.6} = 0.6$$

From Fig. 6-3.7 of "Structural Analysis & Design of Nuclear plant facilities" Draft - Trial use & Comment by ASCE, Page 6-130, we get $F = 0.11$.

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$$\therefore \quad I_a = \frac{1}{2}\left[\frac{Lt^3}{12} + Fbd^3\right]$$

$$= \frac{1}{2}\left[\frac{24\times36^3}{12} + 0.11\times24\times25.89^3\right]$$

$$= 69563 \text{ in}^4$$

Length of piers in West & East $= L = 108'' = a$

Also length of piers in west only $= L = 78'' = a$

ω = frequency in West & East of pier of length 108"

$$= \frac{\lambda}{a^2}\sqrt{\frac{D}{\rho}} = \frac{22.7929}{108^2}\sqrt{\frac{3.12\times10^6\times69563}{0.1941}}$$

Mass/unit length

$$= 2066.36 \text{ r/sec.}$$

And $\omega = \frac{22.7929}{78^2}\sqrt{\frac{3.12\times10^6\times69563}{0.1941}} = 3961.5$ r/sec.

$\therefore$ T = Period in West & East, pier of length 108" $= \frac{2\pi}{\omega} = 0.003045$ sec.

T = Period in West, pier of length 78" $= \frac{2\pi}{\omega} = 0.0016$ sec.

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R_m = Static Collapse load for West & East M.S. & E.W. Pipe Chase for Piers of length 108'' $= \frac{8M_U}{L} = \frac{8 \times 1460 \times 12}{108} = 1297.8^k$

Similarly R_m for pier of length 78'' $= \frac{8 \times 1460 \times 12}{78} = 1797^k$

and Displacement $X_e = \frac{R_m L^3}{192 E I_a} = \frac{1297.8 \times 108^3}{192 \times 3.12 \times 10^3 \times 69563} = 0.039''$

and $X_e = \frac{1797 \times 78^3}{192 \times 3.12 \times 10^3 \times 69563} = 0.02''$

$\therefore X_m = \mu_{all} X_e = 10 \times 0.039 = 0.39''$ Similarly $X_m = 0.2''$

R_s and X_s are taken from old Calculation.

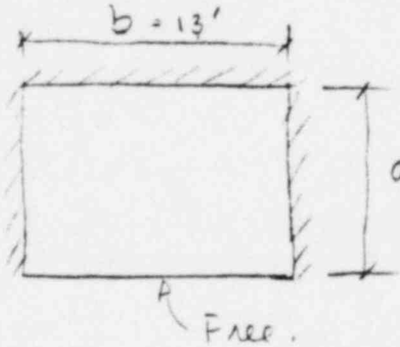
No.	CRITICAL PANEL AREA		R_m	R_s	X_m	X_e	X_s	$R'_m = R_m - R_s$	$X'_m = X_m - X_s$	$X'_e = X_e - X_s$	$\mu'_{all} = \frac{X'_m}{X'_e}$
	BUILDING	LOCATION	Kips	Kips	in	in	in	Kips		in	
9	M.S.&E.W Pipe Chase (EAST)	Piers	1297.8	2.14	0.39	0.039	8.92×10^{-4}	1295.66	0.3891	3.8108×10^{-2}	10.21
10	M.S. & E.W Pipe Chase (West)	East Piers	1297.8	2.14	0.39	0.039	8.92×10^{-4}	1295.66	0.3891	3.8108×10^{-2}	10.21
		West Piers	1797	1.61	0.2	0.02	2.82×10^{-4}	1795.29	0.199718	1.9718×10^{-2}	10.13

Item # 19

Emergency Feed Water Bldg.'s Missile

Shield Wall at EL. 44, the resived

$A_s = A_s' = 1.58 \text{ in}^2$, $a = 4'$, $b = 13'$



$\therefore I_a = \frac{1}{2}\left(\frac{bt^3}{12} + Fbd^3\right)$

where $d = 21"$, $b = 1"$, $t = 24"$

F for $(\rho = 0.00627 = \rho')$, $\rho n = 0.056$ & $\frac{\rho'}{\rho} = 1$

$= 0.04$

$\therefore I_a = \frac{1}{2} \times \left[\frac{1 \times 24^3}{12} + 0.04 \times 1 \times 21^3\right] = 761.22 \text{ in}^4/\text{in}.$

$M_U = \phi A_s f_y \left(d - \frac{a}{2}\right)$ where $a = \frac{A_s f_y}{0.85 f_c' b} = 3.1$

$= 0.9 \times 1.58 \times 60,000 \left(21 - \frac{3.1}{2}\right) = 1,659,474$ lbs-in/ft. ✓

$\therefore R_m = 8 M_U = 1.33 \times 10^7$ lbs/ft. X_e = not Calculated.

$= 1.106 \times 10^6$ lbs/in.

R_s as before = not Calculated.

$\mu_{all} = 10$ $\omega = \text{frequency} = \frac{5.22}{48^2}\sqrt{\frac{3.12 \times 10^6 \times 761.22}{(1 - .15^2) \times 0.00539}} = 1521$ /sec

$T = \frac{2\pi}{\omega} = 0.00413$ sec. & $R_m' = R_m = 1106^k$

Form 5007 Rev. 3-77

(DISCIPLINE)

GENERAL COMPUTATION SHEET

united engineers & constructors inc.

NAME OF COMPANY SEABROOK UNIT/S

SUBJECT TORNADO MISSILE PROTECTION

CALC. SET NO.		
PRELIM.		
FINAL		
VOID		
SHEET 88 OF		
J.O. 9763.606		
REV	COMP. BY	CHK'D BY
0	Dkm. DATE 10-30-79	LHT DATE 11/12/79
	DATE	DATE

Ref: Page #70.

REVISION ON RESPONSE OF 12"Φ x 15' STEEL PIPE MISSILE ON SLAB.

NO.	Critical Panel Areas. Building	Location	R_m' Kips	F_1 Kips	T Secs	t_d Secs.	$C_T = t_d/T$	$C_R = R_m'/F_1$	Using Fig. #1 for C_T & C_R $x_m/x_e' = \mu$	x_e' in	x_m in	Remark
9	M.S. & E.W. Pipe Chase (EAST)	Piers	1295.66	742.63	0.00304	0.00656	2.16	1.74	1.35			Elastic Plastic
10	M.S. & E.W. Pipe Chase (West)	East Piers	1295.66	742.63	0.00304	"	2.16	1.74	1.35			Elastic Plastic
		West Piers	1795.39	742.63	0.0016	"	4.1	2.42	<1			Elastic
19	Emergency F. W. Bldg.	Missile Shield wall at El. 44	1106	742.63	0.00413	"	1.59	1.49	1.5			Elastic Plastic

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 5(a) to (d), DATED 3/31/82

REF.: Cable Tray Design

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

ACTION ITEM NO. 5 (CABLE TRAY DESIGN), DATED 3/31/82

a. Question

Why is static load/deflection test suitable for the Dynamic Case?

Response

(1) The static analysis approach used a 1.5 factor to account for higher mode contributions.

(2) In the static load test, the load application is monotonic increasing, allowing non-linear effects to continue unrestrained. In addition, a true dynamic application would result in greater strength.

(3) Definition of tray failure:
Failure occurs at the first indication of load plastic deformation at any tray region, but has been observed to occur at the rung/side rail joint, near support point. This is extremely conservative.

(4) The load distribution and boundary conditions applied in the static tests, result in a static mode shape which is equivalent to the significant dynamic mode shape, justifying the static test approach.

b. Question

Show that cable integrity is maintained when tray is deformed to proportional load limit + 1/3 (ultimate load limit - Prop. Load Limit).

Response

Design limits on tray deflection are 0.65 in. vertical and 0.33 in. horizontal directions for dead weight plus SSE condition.

ACTION ITEM NO. 5 DATED 3/31/82 *(Cont'd)*

The resultant deflection is .73 in for a 10 ft. span, the change in cable length is 0.0088 in., which is negligible, in a cable which is loosely placed within trays.

c. Question

What is load criteria for connection of horizontal strut to vertical strut, with cable tray placed on horizontal strut.

Response

See attached Sheet No. 4 (Page 25 from Unistrut Catalog).

d. Question

How is differential displacement between consecutive supports accounted for?

Response

Specific calculations addressing the loading due to differential displacement between consecutive supports are not included in the design calculations.

An evaluation of a floor region subjected to high level of loading & floor response indicates that a differential vertical displacement of 0.140 inches may occur between consecutive supports. For the continuous tray system, this displacement represents an additional bending stress of 2,500 psi. This additional stress is acceptable due to the following design considerations:

(1) The static analysis approach used a 1.5 factor to account for higher mode contributions.

(2) Load/deflection limits are imposed in both the vertical and horizontal directions. Invariably, the attainment of the permissible limit in one direction, precludes the attainment of the permissible limit in the other direction.

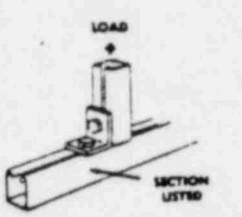
ACTION ITEM NO. 5 DATED 3/31/82 (cont'd)

(3) The design is based upon a conservative cable loading, instead of the smaller "as-built" loading.

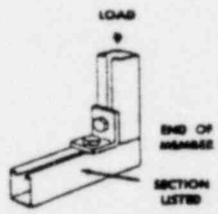
The limiting vertical load corresponds to a bending stress of 26,600 psi. The additional stress due to support displacements represents a stress increase of less than 10%.

When consideration is given to the above conservative features, the additional stress due to the differential displacement would not increase the actual stress beyond the permissible limits.

UNISTRUT All Purpose Metal Framing

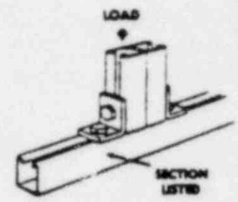


SECTION	*RECOMMENDED LOAD IN LBS.*
P 1000	5000
P 1100	3500
P 2000	2000
P 3000	5000
P 3300	6000
P 4000	2200
P 4100	3400
P 5500	5000
P 5000	4000



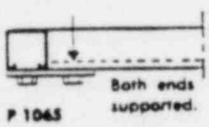
SECTION	RECOMMENDED LOAD IN LBS.*
P 1000	3500
P 1100	2500
P 2000	1500
P 3000	3500
P 3300	4000
P 4000	1700
P 4100	2600
P 5500	3500
P 5000	2000

*Safety factor — 2½



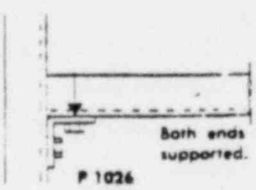
SECTION	RECOMMENDED LOAD IN LBS.*
P 1000	8000
P 1100	5500
P 2000	3000
P 3000	8000
P 3300	9000
P 4000	3500
P 4100	4800
P 5500	8000
P 5000	5500

Safety factor = 2½ based on ultimate strength of connection.
Load diagrams indicate up to three design loads, one for 12 gauge sections (listed as P 1000), one for 14 gauge sections (P 1100), and one for 16 gauge sections (P 2000).

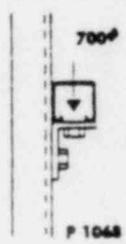


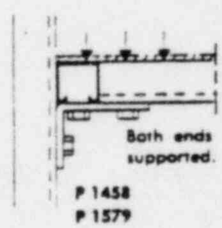
P 1000	1000 lbs.
P 1100	800 lbs.
P 2000	600 lbs.

(When used in position shown)

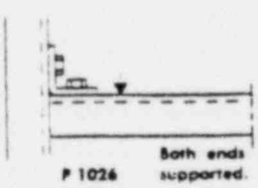


P 1000	1500 #
P 1100	1000 #
P 2000	750 #

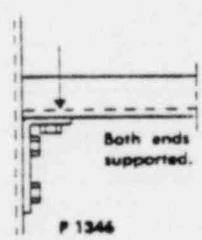




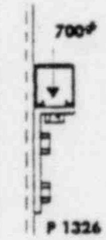
P 1000	1500 lbs.
P 1100	1000 lbs.
P 2000	1000 lbs.

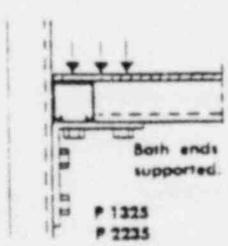


P 1000	1000 #
P 1100	650 #
P 2000	500 #

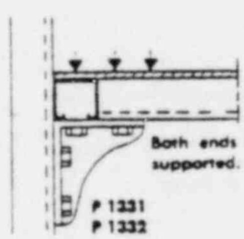


P 1000	2000 lbs.
P 1100	1500 lbs.
P 2000	900 lbs.

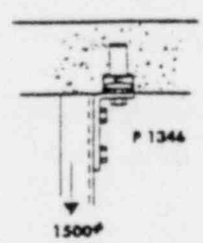




P 1000	2000 lbs.
P 1100	2000 lbs.
P 2000	1500 lbs.



P 1000	3000 lbs.
P 1100	2000 lbs.
P 2000	1500 lbs.



[NRC-SEB Design Audit (3/29/82 to 4/2/82): Response to Action Item #7, dated 3/31/82]

Form 5007 Rev. 3-77

GENERAL COMPUTATION SHEET

(DISCIPLINE) Ref. Action Item #7, dated 3/31/82

united engineers & constructors inc.

CALC. SET NO.	
PRELIM.	
FINAL	CS-15
VOID	
SHEET	1 OF 14
J.O.	9763.006

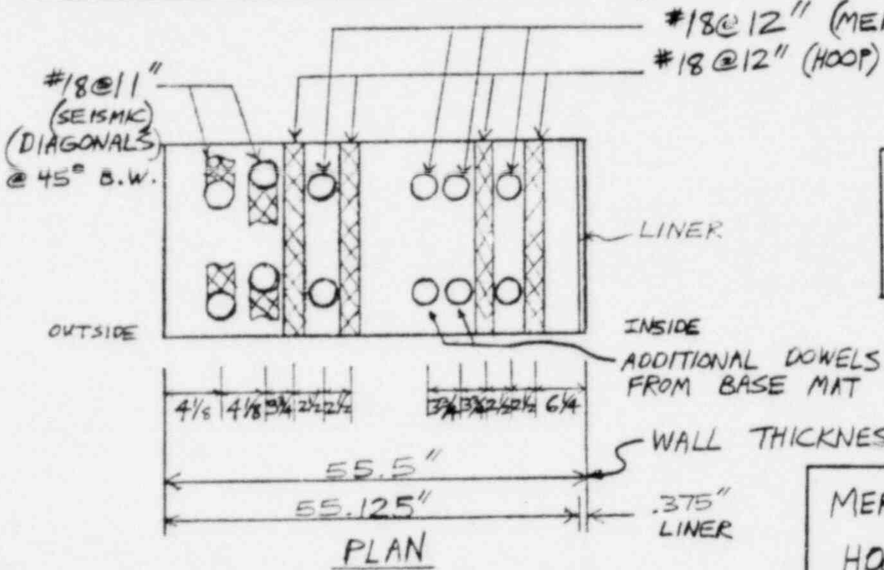
REV	COMP. BY	CHK'D BY
1	JAM DATE 8/17/79	PD DATE 9-18-79
	DATE	DATE

NAME OF COMPANY P.S.N.H. SEABROOK STATION UNIT/S 1+2

SUBJECT CONTAINMENT - Structure (017) SHELL + DOME DESIGN

SECTION PROPERTIES @ SHELL-MAT DISCONTINUITY

(Liner was not considered as a strength element in the design)



PLAN

SECTION IS APPROXIMATELY 1 FT. ABOVE BASE MAT (DISCONTINUITY REGION)

MERID.: 4-#18 @ 12" = 16 IN²/FT
HOOP: 4-#18 @ 12" = 16 IN²/FT
SEISMIC: #18 @ 11" @ 45° B.W. = 4.36 IN²/FT

I. MERID. DIRECTION

a.) INSIDE FACE

A (in²)	Y (in)	AY
4	8.75	35
4	15	60
4	18.75	75
12	$\bar{Y}$=14.17	170

0.375" REDUCTION FOR LINER → $\bar{Y}$ = 13.79"

b.) OUTSIDE FACE

	A (in²)	Y (in)	AY
MERID. COMPONENT OF DIAGONALS	2.18	4.125	8.99
	2.18	8.25	17.99
	4	14.5	58
	8.36	$\bar{Y}$=10.16"	84.98

II. HOOP DIRECTION

a.) INSIDE FACE

A (in²)	Y (in)	AY
4	6.25	25
4	11.25	45
8	$\bar{Y}'$=8.75"	70

0.375" REDUCTION FOR LINER → $\bar{Y}$ = 8.38"

b.) OUTSIDE FACE

	A (in²)	Y (in)	AY
HOOP COMPONENT OF DIAGONALS	2.18	4.125	8.99
	2.18	8.25	17.99
	4	12	48
	4	17	68
	12.36	$\bar{Y}$=11.57"	142.98

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 7, DATED 3/31/82

REF.: Typical containment design data for mechanical loads and the resulting stresses/strains in rebars and concrete at shell-base mat junction and membrane zone.

FORM 5007 REV 7/79

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc

NAME OF COMPANY P.S.N.H. SEABROOK STATION UNIT/S 1+2

SUBJECT CONTAINMENT - Structure (017)

CALC SET NO.		REV	COMP BY	CHK'D BY
PRELIM		0	J. MOTT	[illegible]
FINAL			DATE 4/1/82	DATE 4/1/82
VOID				
SHEET 2 OF 14			DATE	DATE
J.O. 9763.006				

DESIGN LOADS FOR CONTAINMENT SECTION

AT SHELL-MAT DISCONTINUITY

FACTORED LOAD CONDITION -- LOAD CASE $D + P_a + SSE$

(FOR MECHANICAL LOADS ONLY)

(P_a = 52 psig)

MERID. FORCE = 368 KIPS PER FOOT LENGTH

HOOP FORCE = 34 KIPS PER FOOT LENGTH

INPLANE SHEAR FORCE = 129 KIPS PER FOOT LENGTH

MERID. MOMENT = 6387 INCH-KIPS PER FOOT LENGTH

HOOP MOMENT = 673 INCH-KIPS PER FOOT LENGTH

(REF.: CALC. SET CS-15, SHT. 164)

NOTATION: POSITIVE FORCE IS TENSILE FORCE.

POSITIVE MOMENT PRODUCES TENSION ON LINER SIDE.

DESIGN PROPERTIES OF MATERIALS:

f'_c = 3 KSI E_c = 3122 KSI

F_y = 60 KSI E_s = 29,000 KSI

FORM 5007 REV 7/79

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc

CALC SET NO.		REV	COMP BY	CHK'D BY
PRELIM		0	J. MOTT DATE 4/1/82	DATE 4/1/82
FINAL				
VOID				
SHEET 3 OF 14			DATE	DATE
J.O. 9763.6				

NAME OF COMPANY P.S.N.H. SEABROOK STATION UNIT/S 1+2

SUBJECT CONTAINMENT - Structure (017)

FINAL COMPUTED STRESSES + STRAINS (FROM PROGRAM LESCAL)

LOAD CASE $D + P_a + SSE$ @ MAT DISCONTINUITY

REBAR STRESSES

MERIDIONAL : INSIDE = 35.64 KSI

OUTSIDE = 2.20 KSI

HOOP : INSIDE = 10.29 KSI

OUTSIDE = 6.67 KSI

SEISMIC (DIAGONALS) P_3 = 18.24 KSI (TENSION DIAG.)

P_4 = −9.38 KSI (OTHER DIAG.)

CONCRETE STRESS + STRAIN

$\sigma_{MERID} = -0.823$ KSI

$\varepsilon_{MERID} = 0.000354$ IN/IN

OUTPUT OF PROGRAM LESCAL

SHELL WALL EL(-)30.0' D+P+SSE 30 3 8/29/79DUCHON EQS

DIRECT SOLUTION

TOTAL CROSS SECTIONAL AREA = 661.56 SQ.IN.
MODULUS OF ELASTICITY OF CONCRETE = 3122.00 KSI
MODULUS OF ELASTICITY OF REINFORCING STEEL = 29000.00 KSI

SEABROOK STATION

CONTAINMENT - Structure (017)

FLAG FOR ANGLE OF INCLINED REBAR ------- 0
.EQ.0 ALPHA3=-ALPHA4=45 DEG.
.EQ.1 INPUT ALPHA3 AND ALPHA4

THICKNESS OF CROSS SECTION = 55.13 IN.
TOTAL AREA OF MERID. REBAR = 16.00 SQ. IN.
TOTAL AREA OF HOOP REBAR = 16.00 SQ. IN.
AREA OF SEISMIC REBAR (AS3) = 4.36 SQ. IN.
INSIDE (AS3I) = 0. SQ. IN.
OUTSIDE (AS3O) = 4.36 SQ. IN.
AREA OF SEISMIC REBAR (AS4) = 4.36 SQ. IN.
INSIDE (AS4I) = 0. SQ. IN.
OUTSIDE (AS4O) = 4.36 SQ. IN.

SHELL-MAT DISCONTINUITY

Pg. 4 OF 14

MEMBRANE VERTICAL FORCE = 367.67 K/FT.
MEMBRANE HORIZONTAL FORCE = 33.65 K/FT.
MEMBRANE SHEAR FORCE = 128.52 K/FT.
RATIO OF AS IN VERT. DIR. TO AG = 0.024185
RATIO OF AS IN HORI. DIR. TO AG = 0.024185
RATIO OF AS IN 3-DIR. TO AG = 0.006590
RATIO OF AS IN 4-DIR. TO AG = 0.006590

ANGLE BETWEEN DIR. I=1 AND I=3 (+ CLOCKWISE)= 45.00 DEG.
ANGLE BETWEEN DIR. I=1 AND I=4 (+ CLOCKWISE)= -45.00 DEG.

****
* STRESSES TABLE *
****

NORMAL STRESS IN MERI. DIR. = 21.703 KSI
NORMAL STRESS IN HORI. DIR. = 5.757 KSI
NORMAL STRESS IN (3 - TENS.) DIR = 27.539 KSI
NORMAL STRESS IN (4 - COMP.) DIR. = -0.079 KSI
STRESS IN THE CONCRETE = -0.238 KSI

****
* BETA AND STRAINS TABLE *
****

ANGLE BETWEEN VERT. DIR. AND MAX. PRINC DIR.= 29.999 DEG

MAXIMUM PRINCIPAL (MOST TENSILE) STRAIN = 0.00102328
MINIMUM PRINCIPAL STRAIN = -0.00007639
SHEAR STRAIN = 0.00095233

CALCULATED VERTICAL FORCE = 367.670 K/FT.
CALCULATED HORIZONTAL FORCE = 33.650 K/FT.
CALCULATED IN-PLANE SHEAR = 128.520 K/FT.

```
SHELL WALL EL(-)30.0' D+P+SSE                30           3     8/29/79(MERIDIAN)

    LOADS APPLIED AT PLASTIC CENTROID (TENSION) OF BARS   (ECCENTRICITY =  -0.97 IN. )

CONCRETE COMP. STRENGTH             =       3.00 KSI
SPECIFIED YIELD STRESS OF REBAR =      60.00 KSI
MODULUS OF ELASTICITY OF REBAR  =  29000.00 KSI

THICKNESS OF CROSS SECTION          =      55.13 IN.
AREA OF INSIDE REBAR                =      12.00 SQ. IN.
AREA OF OUTSIDE REBAR               =       8.36 SQ. IN.
LINER FACE TO INSIDE REBAR          =      13.79 IN.
OUTSIDE FACE TO OUTSIDE REBAR       =      10.16 IN.

SECTION FORCE                       =     367.67 K/FT.
SECTION MOMENT                      =    6386.57 IN-K/FT.

    N O T E - MOMENT PRODUCES TENSION IN LINER

NO. OF ITERATION      =        119
RESIDUAL OF FORCE     =      0.002
RESIDUAL OF MOMENT    =     -0.257
RESIDUAL COMP. ZONE =    0.00000
RELAXATION FACTOR     =    0.10000

CONCRETE COMP. STRESS    =     -0.823 KSI
STRESS IN OUTSIDE REBAR =      0.668 KSI
STRESS IN INSIDE REBAR  =     34.226 KSI

DISTANCE OF COMP. ZONE   =      9.540 IN.

STRAIN IN CONCRETE       =   0.000354
STRAIN IN OUTSIDE REBAR =   0.000023
STRAIN IN INSIDE REBAR  =   0.001180
```

P.S.N.H.
SEABROOK STATION

CONTAINMENT - Structure (017)

SHELL-MAT DISCONTINUITY

Pg. 5 OF 14

SHELL WALL EL(-)30.0' D+P+SSE 30 3 8/29/79FINAL DATA

P.S.N.H.
SEABROOK STATION

CONTAINMENT - Structure (017)

SHELL-MAT DISCONTINUITY


```
***********************************
* SECTION FORCES AND MOMENTS TABLE *
***********************************

 MERIDIONAL FORCE   =   367.67 K/FT.
 HOOP FORCE         =    33.65 K/FT.
 TANGENTIAL SHEAR   =   128.52 K/FT.
 MERIDIONAL MOMENT  =  6744.02 IN-K/FT.
 HOOP MOMENT        =   600.16 IN-K/FT.

**********************
* REBAR STRESS TABLE *
**********************

OUTSIDE REBAR STRESSES

 MERIDIONAL    =     2.198 KSI
 HOOP          =     6.666 KSI
 SEISMIC (3)   =    18.241 KSI
 SEISMIC (4)   =    -9.377 KSI

INSIDE REBAR STRESSES

 MERIDIONAL    =    35.636 KSI
 HOOP          =    10.288 KSI
 SEISMIC (3)   =     0.    KSI
 SEISMIC (4)   =     0.    KSI

**********************
* REBAR STRAIN TABLE *
**********************

OUTSIDE REBAR STRAINS

 MERIDIONAL   =  0.000076
 HOOP         =  0.000230
 SEISMIC (3)  =  0.000629
 SEISMIC (4)  = -0.000323

INSIDE REBAR STRAINS

 MERIDIONAL   =  0.001229
 HOOP         =  0.000355
 SEISMIC (3)  =  0.
 SEISMIC (4)  =  0.

 SHEAR STRAIN  (GAMMA)  =  0.000952
```

Form 5015 Rev. 11/74

GENERAL COMPUTATION SHEET

united engineers & constructors inc.

(DISCIPLINE)

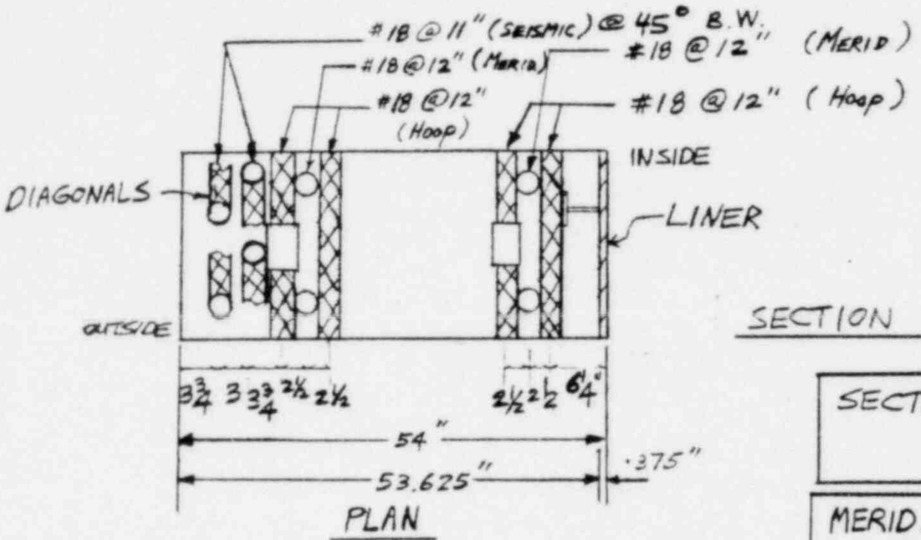
P.S.N.H.
SEABROOK STATION

NAME OF COMPANY P.S.N.H. SEABROOK STATION UNIT/S 1&2

SUBJECT CONTAINMENT - Structure (017) SHELL + DOME

CALC. SET NO.	
PRELIM.	
FINAL	CS15
VOID	
SHEET 8 OF 14	
J.O. 9763-6	

REV	COMP. BY	CHK'D BY
0	WPH DATE 4-12-76	CJL DATE 8/31/76
1	JAM DATE 8/20/79	[illegible] DATE 1/1/81



PLAN

SECTION PROPERTIES @ MEMBRANE REGION

SECTION IS 100 FT. ABOVE BASE MAT (MEMBRANE REGION)

MERID. : 2 – #18 @ 12" = 8 IN²/FT
HOOP : 4 – #18 @ 12" = 16 IN²/FT
SEISMIC: #18 @ 11" @ 45° B.W. = 4.36 IN²/FT

I. MERID DIRECTION

I-a) INSIDE FACE

A	Y
4	8.75

0.375" REDUCTION FOR LINER → $\bar{Y} = 8.38"$

I-b) OUTSIDE FACE

	A	Y	AY
MERID. COMPONENT OF DIAGONALS	2.18	3.75	8.175
	2.18	6.75	14.715
	4	13	52.0
	8.36	8.96"	74.89

REV. 1

II. HOOP DIRECTION

II a) INSIDE FACE

$A_{si} = 8 \square''$

$\bar{Y}' = 8.75"$

0.375" REDUCTION FOR LINER → $\bar{Y} = 8.38"$

II b) OUTSIDE FACE

$A_{so} = 12.36 \square''$ — INCLUDES HOOP COMPONENT OF DIAGONALS

$\bar{Y} = 10.27"$

NOTE: LINER WAS NOT CONSIDERED AS A STRENGTH ELEMENT IN THE DESIGN.

P.S.N.H.
SEABROOK STATION

CONTAINMENT - Structure (017)

SHELL - MAT DISCONTINUITY


```
SHELL WALL EL(-)30.0' D+P+SSE                    30              3       8/29/79(HOOP)

    LOADS APPLIED AT PLASTIC CENTROID (TENSION) OF BARS   (ECCENTRICITY =   2.17 IN. )

CONCRETE COMP. STRENGTH          =       3.00 KSI
SPECIFIED YIELD STRESS OF REBAR  =      60.00 KSI
MODULUS OF ELASTICITY OF REBAR   =   29000.00 KSI

THICKNESS OF CROSS SECTION       =      55.13 IN.
AREA OF INSIDE REBAR             =       8.00 SQ. IN.
AREA OF OUTSIDE REBAR            =      12.36 SQ. IN.
LINER FACE TO INSIDE REBAR       =       8.38 IN.
OUTSIDE FACE TO OUTSIDE REBAR    =      11.57 IN.

SECTION FORCE                    =      33.65 K/FT.
SECTION MOMENT                   =     673.24 IN-K/FT.

    N O T E - MOMENT PRODUCES TENSION IN LINER

NO. OF ITERATION      =       122
RESIDUAL OF FORCE     =    -0.001
RESIDUAL OF MOMENT    =    -0.009
RESIDUAL COMP. ZONE = 0.00000
RELAXATION FACTOR     =   0.10000

CONCRETE COMP. STRESS    =    -0.066 KSI
STRESS IN OUTSIDE REBAR =     0.408 KSI
STRESS IN INSIDE REBAR   =     3.949 KSI

DISTANCE OF COMP. ZONE   =     7.515 IN.

STRAIN IN CONCRETE       =  0.000026
STRAIN IN OUTSIDE REBAR  =  0.000014
STRAIN IN INSIDE REBAR   =  0.000136
```

FORM 5007 REV 7/79

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc

CALC. SET NO.		REV	COMP. BY	CHK'D. BY
PRELIM		0	J. MOTT	[illegible]
FINAL			DATE 4/1/82	DATE 4/1/82
VOID				
SHEET 9 OF 14			DATE	DATE
J.O. 9763.006				

NAME OF COMPANY P.S.N.H. SEABROOK STATION UNIT/S 1+2

SUBJECT CONTAINMENT - Structure (017)

DESIGN LOADS FOR TYPICAL CONTAINMENT SECTION IN MEMBRANE REGION (100' ABOVE MAT)

FACTORED LOAD CONDITION - LOAD CASE $D + P_a + SSE$

(P_a = 52 psig)

MERID. FORCE = 280 KIPS PER FOOT LENGTH

HOOP FORCE = 509 KIPS PER FOOT LENGTH

INPLANE SHEAR FORCE = 98 KIPS PER FOOT LENGTH

MERID. MOMENT = -1143 INCH-KIPS PER FOOT LENGTH

HOOP MOMENT = -713 INCH-KIPS PER FOOT LENGTH

(REF.: CALC. SET CS-15, SHT. 168)

NOTATION: POSITIVE FORCE IS TENSILE FORCE.

POSITIVE MOMENT PRODUCES TENSION ON LINER SIDE.

DESIGN PROPERTIES OF MATERIALS:

f'_c = 3 KSI E_c = 3122 KSI

F_y = 60 KSI E_s = 29,000 KSI

FORM 5007 REV. 7/79

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc

NAME OF COMPANY P.S.N.H. SEABROOK STATION UNIT/S 1+2

SUBJECT CONTAINMENT - Structure (017)

CALC. SET NO.		REV	COMP. BY	CHK'D BY
PRELIM		0	J. MOTT	[illegible]
FINAL			DATE 4/1/82	DATE 4/2/82
VOID				
SHEET	10 OF 14		DATE	DATE
J.O.	9763.006			

FINAL COMPUTED STRESSES + STRAINS (FROM PROGRAM LESCAL)

LOAD CASE $D + P_a + SSE$ IN MEMBRANE REGION

REBAR STRESSES

MERIDIONAL : INSIDE = 14.21 KSI

OUTSIDE = 26.84 KSI

HOOP : INSIDE = 22.81 KSI

OUTSIDE = 26.34 KSI

SEISMIC (DIAGONALS) : P_3 = 49.06 KSI (TENSION DIAG.)

P_4 = 4.13 KSI (OTHER DIAG.)

STRAINS

MAX. REBAR = 0.00169 (P_3) (TENSION DIAG. REBAR)

CONCRETE : $\varepsilon_{MERID} = 0$

$\varepsilon_{HOOP} = 0$

(SHEAR) $\gamma = 0.001549$

OUTPUT OF PROGRAM LESCAL

P.S.N.H.
SEABROOK STATION
CONTAINMENT - Structure (017)

MEMBRANE REGION
(100' ABOVE MAT)
Pg. 11 OF 14

```
SHELL WALL EL 70-0 D+P+SSE                    70            3      8/29/79DUCHON FQS
                                        DIRECT SOLUTION

TOTAL CROSS SECTIONAL AREA                   =     643.44 SQ.IN.
MODULUS OF ELASTICITY OF CONCRETE            =    3122.00 KSI
MODULUS OF ELASTICITY OF REINFORCING STEEL   =   29000.00 KSI

FLAG FOR ANGLE OF INCLINED REBAR -------  0
 .EQ.0  ALPHA3=-ALPHA4=45 DEG.
 .EQ.1  INPUT ALPHA3 AND ALPHA4

THICKNESS OF CROSS SECTION  =  53.62       IN.
TOTAL AREA OF MERID. REBAR  =   8.00 SQ. IN.
TOTAL AREA OF HOOP   REBAR  =  16.00 SQ. IN.
AREA OF SEISMIC REBAR (AS3) =   4.36 SQ. IN.
         INSIDE   (AS3I)    =   0.   SQ. IN.
         OUTSIDE  (AS3O)    =   4.36 SQ. IN.
AREA OF SEISMIC REBAR (AS4) =   4.36 SQ. IN.
         INSIDE   (AS4I)    =   0.   SQ. IN.
         OUTSIDE  (AS4O)    =   4.36 SQ. IN.

MEMBRANE VERTICAL FORCE             =     280.18 K/FT.
MEMBRANE HORIZONTAL FORCE           =     509.19 K/FT.
MEMBRANE SHEAR FORCE                =      97.95 K/FT.
RATIO OF AS IN VERT. DIR. TO AG     =   0.012433
RATIO OF AS IN HORI. DIR. TO AG     =   0.024866
RATIO OF AS IN 3-DIR. TO AG         =   0.006776
RATIO OF AS IN 4-DIR. TO AG         =   0.006776

ANGLE BETWEEN DIR. I=1 AND I=3  (+ CLOCKWISE)=  45.00 DEG.
ANGLE BETWEEN DIR. I=1 AND I=4  (+ CLOCKWISE)= -45.00 DEG.

      *******************
      * STRESSES  TABLE *
      *******************

NORMAL STRESS IN MERI. DIR.          =    22.092 KSI
NORMAL STRESS IN HORI. DIR.          =    25.359 KSI
NORMAL STRESS IN (3 - TENS.) DIR.    =    46.191 KSI
NORMAL STRESS IN (4 - COMP.) DIR.    =     1.260 KSI
STRESS IN THE CONCRETE               =     0.    KSI

      **************************
      * BETA AND STRAINS TABLE *
      **************************

ANGLE BETWEEN VERT. DIR. AND MAX. PRINC DIR.=       47.079 DEG

MAXIMUM PRINCIPAL ( MOST TENSILE ) STRAIN    =  0.00159485
MINIMUM PRINCIPAL  STRAIN                    =  0.00004140
SHEAR STRAIN                                 =  0.00154935

CALCULATED VERTICAL FORCE    =   280.180 K/FT.
CALCULATED HORIZONTAL FORCE  =   509.190 K/FT.
CALCULATED IN-PLANE SHEAR    =    97.950 K/FT.
```

SHELL WALL EL 70-0 D+P+SSE 70 3 8/29/79(MERIDIAN)

LOADS APPLIED AT PLASTIC CENTROID (TENSION) OF BARS (ECCENTRICITY = 6.11 IN.)

P.S.N.H.
SEABROOK STATION
CONTAINMENT - Structure (017)

```
CONCRETE COMP. STRENGTH            =      3.00 KSI
SPECIFIED YIELD STRESS OF REBAR =        60.00 KSI
MODULUS OF ELASTICITY OF REBAR  =     29000.00 KSI

THICKNESS OF CROSS SECTION         =     53.62 IN.
AREA OF INSIDE REBAR               =      4.00 SQ. IN.
AREA OF OUTSIDE REBAR              =      8.36 SQ. IN.
LINER FACE TO INSIDE REBAR         =      8.38 IN.
OUTSIDE FACE TO OUTSIDE REBAR      =      8.96 IN.

SECTION FORCE                      =    280.18 K/FT.
SECTION MOMENT                     =  -1143.22 IN-K/FT.
```

MEMBRANE REGION

Pg 12 OF 14

NOTE - MOMENT PRODUCES COMPRESSION IN LINER

```
****************************************
*  STRESSES AND STRAINS DUE TO MOMENT  *
****************************************

STRAIN IN OUTSIDE REBAR =  0.000130
STRESS IN OUTSIDE REBAR =       3.77 KSI
STRAIN IN INSIDE  REBAR = -0.000272
STRESS IN INSIDE  REBAR =      -7.88 KSI
```

```
SHELL WALL EL 70-0 D+P+SSE                    70            3      8/29/79(HOOP)

     LOADS APPLIED AT PLASTIC CENTROID (TENSION) OF BARS    (ECCENTRICITY =    2.80 IN. )

CONCRETE COMP. STRENGTH             =       3.00 KSI
SPECIFIED YIELD STRESS OF REBAR =          60.00 KSI
MODULUS OF ELASTICITY OF REBAR   =  29000.00 KSI

THICKNESS OF CROSS SECTION          =      53.62 IN.
AREA OF INSIDE REBAR                =       8.00 SQ. IN.
AREA OF OUTSIDE REBAR               =      12.36 SQ. IN.
LINER FACE TO INSIDE REBAR          =       8.38 IN.
OUTSIDE FACE TO OUTSIDE REBAR       =      10.27 IN.

SECTION FORCE                       =     509.19 K/FT.
SECTION MOMENT                      =    -712.74 IN-K/FT.

     NOTE - MOMENT PRODUCES COMPRESSION IN LINER

          ****************************************
          *  STRESSES AND STRAINS DUE TO MOMENT  *
          ****************************************

STRAIN IN OUTSIDE REBAR =  0.000057
STRESS IN OUTSIDE REBAR =        1.65 KSI
STRAIN IN INSIDE  REBAR = -0.000088
STRESS IN INSIDE  REBAR =       -2.55 KSI
```

P.S.N.H.
SEABROOK STATION

CONTAINMENT - Structure (017)

MEMBRANE REGION

Pg. 13 OF 14

SHELL WALL EL 70-0 D+P+SSE 70 3 8/29/79FINAL DATA

P.S.N.H.
SEABROOK STATION

CONTAINMENT - Structure (017)

MEMBRANE REGION


```
***********************************
* SECTION FORCES AND MOMENTS TABLE *
***********************************

  MERIDIONAL FORCE   =     280.18 K/FT.
  HOOP FORCE         =     509.19 K/FT.
  TANGENTIAL SHEAR   =      97.95 K/FT.
  MERIDIONAL MOMENT  =   -1143.22 IN-K/FT.
  HOOP MOMENT        =    -712.74 IN-K/FT.

**********************
* REBAR STRESS TABLE *
**********************

OUTSIDE REBAR STRESSES

  MERIDIONAL    =    26.844 KSI
  HOOP          =    26.344 KSI
  SEISMIC (3)   =    49.059 KSI
  SEISMIC (4)   =     4.128 KSI

INSIDE REBAR STRESSES

  MERIDIONAL    =    14.214 KSI
  HOOP          =    22.811 KSI
  SEISMIC (3)   =    0.     KSI
  SEISMIC (4)   =    0.     KSI

**********************
* REBAR STRAIN TABLE *
**********************

OUTSIDE REBAR STRAINS

  MERIDIONAL    =  0.000926
  HOOP          =  0.000908
  SEISMIC (3)   =  0.001592
  SEISMIC (4)   =  0.000142

INSIDE REBAR STRAINS

  MERIDIONAL    =  0.000490
  HOOP          =  0.000787
  SEISMIC (3)   =  0.
  SEISMIC (4)   =  0.

  SHEAR STRAIN  (GAMMA) =  0.001549
```

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 4, DATED 4/1/82

REF. RAI NO. 220.16

Calculation of lateral soil pressures on rigid wall

Fo. 5007 Rev. 3-77

GENERAL COMPUTATION SHEET

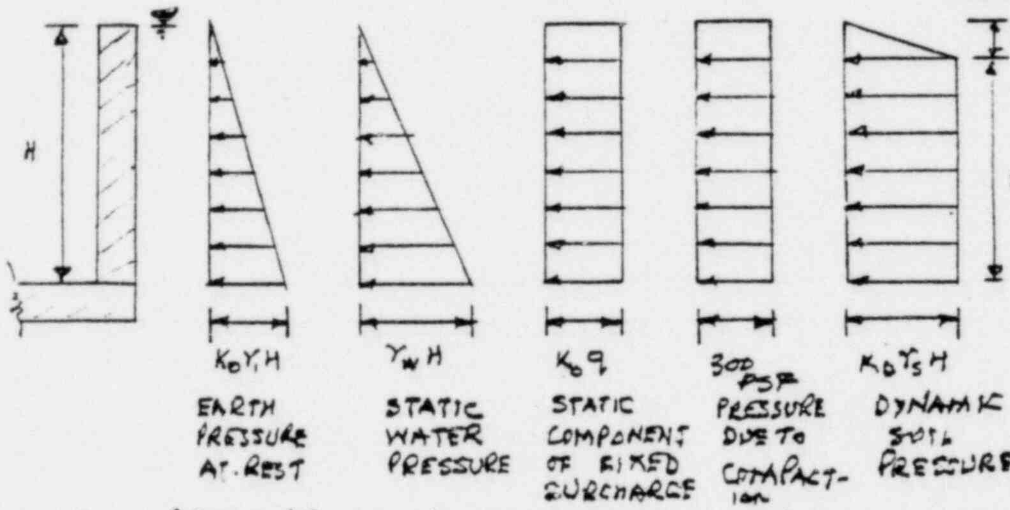
(DISCIPLINE)

united engineers & constructors inc.

NAME OF COMPANY PSNH UNIT/S 1 & 2

SUBJECT SEABROOK STATION – LATERAL PRESSURES RIGID WALL.

CALC. SET NO.		
PRELIM.		
FINAL		
VOID		
SHEET OF		
J.O. 9763.006		
REV	COMP. BY	CHK'D BY
0	4/1/82 DATE DTP	4/7/82 DATE SAF
	DATE	DATE



$h = H \frac{K_D}{K_P}$
$= 63 \times \frac{0.28}{3.3} = 5.35'$

LATERAL PRESSURE COMPONENTS AT THE BASE OF WALL:

EARTH PRESSURE AT-REST

$K_0 \gamma_1 H = 0.5 \times 62.5 \times 63 = \underline{1968.8}$ PSF

STATIC WATER PRESSURE

$\gamma_w H = 62.5 \times 63 = \underline{3937.5}$ PSF.

DYNAMIC SOIL PRESSURE

$K_D \gamma_s H = 0.28 \times 125 \times 63 = \underline{2205}$ PSF.

NRC - SEB Design Audit, 3/29/82 to 4/2/82

This calculation is submitted in response to Action Item No. 4, dated April 1, 1982.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 5, DATED 4/1/82

REF. RAI NO. 220.36

220.36
(3.7.1.2) Figure 3.7(B)-16 and 3.7(B)-17 of the FSAR show 0.5% and 1% critical damping response spectra respectively, in which time history responses have more than ten (10) points falling below the design response spectra. According to SRP Section 3.7.1 subsection II.1.b, the spectra-enveloping requirment is that no more than five (5) points of the spectra obtained from time history should fall below the design response spectra.

RESPONSE We have not used 0.5% damping in the design of structure, component and equipment for seismic analysis in vertical direction. The Figure 3.7(B)-16 will be removed from the FSAR.

For 1% damping spectra for vertical direction shown in Figure 3.7(B)-17, the mean of the spectral amplitude ratios for time history and the R.G. 1.60 spectra calculated between the frequencies of 0.5 Hz. and 33 Hz. is 1.22, which is greater than 1.0.

The mean of the spectral amplitude ratios is calculated using the following expression:

$$\frac{1}{n} \sum_{i=i}^{n} \frac{(TS)_i}{(DS)_i}$$

where:

n = total number of frequencies between 0.5 Hz. to 33 Hz.

$(TS)_i$ = Spectral amplitude of time history motion at i^{th} frequency.

$(DS)_i$ = Spectral amplitude of design response spectra at i^{th} frequency.

Therefore, the vertical time history spectra exceeds the target spectra on the average. This high value of mean of ratios of the spectral amplitudes indicates the severity of the postulated excitation and therefore the response of structure, system and components will be conservative.

This method of establishing correlation between the time history spectra and the R.G. 1.60 spectra was considered appropriate during original design of the plant when SRP was not in effect.

The design vertical time history motion is not used directly for computing response of any structure, system or components having 1.0% damping.

The subsystems having 1% damping are invariably located on the structures and hence the vertical design time history motion is not used directly for the design of these subsystems. The design time history motion is filtered through structures having 4% damping for OBE and 7% damping for SSE, and these filtered time history motions are then used for the design of subsystem.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 7, DATED 4/1/82

REF. RAI NO. 220.9

RAI 220.9 (3.4.2)

The methods by which the dynamic effects of design basis flood are applied to safety related structures are not clearly mentioned in FSAR. Since the flood level is above the proposed plant grade, such dynamic loads and their determination is an important concern to NRC staff. SRP Section 3.4.2 Subsection II.3 delineates an acceptance method. Clearly mention the methods and procedures used, stating whether or not you meet the SRP criteria.

RESPONSE:

Dynamic effects of the design basis flood were considered, but found to be negligible. As stated in the FSAR (Subsections 2.4.5.3 and 3.4.1), the maximum depth of stillwater is 0.6 feet above plant grade, and the maximum wave runup in local regions is 1.8 feet above plant grade. Any dynamic effects produced by these occurrences were evaluated and found to be negligible and, due to the relatively large masses of the reinforced concrete structures, can be neglected.

Note, however, that hydrostatic effects of the flood are considered in the design of structures with regard to buoyancy and associated behavior.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 8, DATED 4/1/82

REF. RAI NO. 220.14

220.14

(3.7(B).2.3 In this section, you have stated that in the modelling of the containment's internal structure the NSSS component and their supports are modelled. However, Figure 3.7(B)-23 of FSAR indicates only four mass model for this structure without any NSSS components. Clarify this apparent contradiction. Also, if you have not included detailed models of NSSS component (reactor vessel, steam generators) in the seismic analysis of structures, justify their exclusion.

RESPONSE Westinghouse and UE&C performed separate but coordinated analyses of this structural system. UE&C developed a dynamic model of the concrete internal structures which Westinghouse incorporated into their coupled dynamic model of the structure and NSSS system. UE&C added the NSSS system masses into their uncoupled model of the internal structure. A figure representing the model of the coupled system is not available from Westinghouse, therefore, Figure 3.7(B)-23 of the FSAR illustrates the UE&C uncoupled model only.

Total seismic base shears and moments generated by the Westinghouse coupled system analysis were used by UE&C in the design of the internal structures.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 8, DATED 4/1/82

REF. RAI NO. 220.15

220.15
(3.7(B).2.3) Your decoupling criteria between system and subsystem is not clearly stated in this section. Demonstrate that your decoupling criteria are either equivalent or more conservative than those given in SRP Section 3.7.2.II.(b), which are acceptable to the staff.

RESPONSE The approach to seismic system analysis assumes that all seismic subsystems except the NSS system are decoupled from the seismic system, including both the analysis of the primary structure for the response spectra generation and the analysis of structural components for amplified response spectra (ARS) generation. The masses of equipment, piping, etc., are included in the system model.

UE&C has considered the effects of assuming decoupled seismic system and subsystem in the design of equipment, piping, cable trays, ducts, etc. Consideration was given to groups of seismic subsystems as follows:

a) Ducts, conduits, panels and small piping. These systems satisfy low mass ratio criteria.

b) Pumps, heat exchangers, tanks, etc. These systems satisfy low mass criteria in the horizontal direction and a frequency ratio criteria in the vertical direction.

c) Larger piping and cable trays. These systems are recognized to be potentially coupled with structural components (i.e. slabs) for vertical motion.

A review indicates that the approach embodying items a) and b) satisfactorily ensure that the system and subsystem are substantially de-

RAI 220.15 (Cont'd)

coupled in their responses. The piping and cable trays of item c) satisfy low mass ratio criteria in the response of the seismic system to horizontal motion, but coupling effects may be present in response to vertical motions. These coupling effects are believed to be insignificant and to be enveloped by conservatisms inherent in the application of the ARS to design and the analytical basis used in generating ARS.

UE&C has undertaken a study to demonstrate the design adequacy of representative groups of systems wherein coupling may exist giving consideration to the decoupling guidelines of SRP 3.7.2 or equivalent.

The NSS system was coupled to the Containment concrete internal structures. The UE&C seismic models of shell and internal structure were incorporated by Westinghouse into the detailed NSSS seismic model. Total base shears and moments obtained from the Westinghouse coupled system analysis were used by UE&C in the design of the concrete internal structures. Response spectra for NSSS components were supplied by Westinghouse. Response spectra for the Containment internal structures were generated both by UE&C and Westinghouse using uncoupled and coupled models, respectively. The response spectra from the uncoupled model envelope the spectra from the coupled model for all directions and elevations except for a secondary peak in the operating floor spectra in the E-W direction. The most significant effect of the coupling is the reduction in magnitude of the primary peaks in the

RAI 220.15 (Cont'd)

spectra. Piping systems associated with the NSS system were designed for spectra enveloping both UE&C and Westinghouse response spectra. Smaller piping systems and equipment, etc. were designed/qualified using the UE&C spectra. The presence of the secondary peak in the operating floor E-W spectra has no significant effect on seismic subsystems on the operating floor. It's effect on the response spectra generated for the steel frame supported by the internal structures is under review.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 10, DATED 4/1/82

REF. RAI NO. 220.24

Question: 220.24 (3.7(B).3.7)

The modal response for closely spaced modes is obtained by equation (1) & (2) given in Section 3.7(B).3.7 of the FSAR. Confirm that equation (1) gives conservative results and meets the intent of the criteria of Regulatory Guide 1.92, Rev. 1, 1976. If not, justify the deviation.

Response:

See response to RAI 220.1 (3.7.2, 3.7.3), Amendment 44, February 1982.

Conservatism in combining modal response is equal to or greater than that recommended in Regulatory Guide 1.92.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 13, DATED 4/1/82

REF. RAI NO. 220.8

RAI 220.8 (3.3.2)

In Section 3.3.2.2 of the FSAR, it is mentioned that maximum velocity pressure is given by the formula $q_{max} = 0.00256V^2$. Confirm that the velocity pressure is assumed to be constant with height, and that maximum velocity pressure applies at the radius of the tornado funnel at which the maximum velocity occurs. If not, then clearly mention your assumptions.

Also, clarify how you have considered the variation of tangential velocity with the radial distance from the center of the tornado core.

RESPONSE:

Velocity pressure is assumed to be constant with height. Maximum velocity pressure is based on the maximum tornado wind velocity and is assumed to occur at the radius of the tornado funnel at which the maximum velocity occurs.

Variation of tangential velocity with radial distance from the tornado is determined as follows:

$$V_t = \frac{r}{r_m} \times V_{t\ max} \text{ for } 0 < r < r_m$$

$$V_t = \frac{r_m}{r_r} \times V_{t\ max} \text{ for } r_m < r < r_{75}$$

where

V_t = tangential velocity at radius r
V_t = maximum tangential velocity (290 mph)
r = radius from centerline of tornado
r_m = radius of maximum tangential velocity (150 ft)
r_{75} = radius at which tangential velocity equals 75 mph (580 ft)

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 14, DATED 4/1/82

REF. RAI NO. 220.27

RAI 220.27 (3.8.1.6)

Confirm that the materials of construction are in accordance with Article CC-2000 of ASME Section III Division 2 Code, augmented by Regulatory Guide 1.136. If not, identify the deviations and justify the same.

Response:

Seabrook containments are built to ASME Code Section III, Division 2, 1975 except that prepackaged grout and epoxies were not addressed in the Code. The Code committies are currently in the process of revising the Code to allow the use of prepackaged grout and epoxies. We will keep NRC staff advised of the progress of this issue. All other materials requirements of Article CC-2000, as augmented by Regulatory Guide 1.136, are being met.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 1(a), DATED 4/2/82

Technical basis for treating cable trays as as non-safety related structures.

SB 1 & 2

NRC - SEB DESIGN AUDIT (3/29/82 to 4/2/82)

Item 1(a), dated 4/2/82

Issue: Technical basis for treating cable trays as non-safety related structural elements.

Response:

Cable trays, like conduits and other raceway components, when used to carry safety related circuit cables are qualified as assemblies. Cable tray is purchased as a component with specific performance requirements, and the manufacturer provides substantiating test data and calculations. The manufacturer also provides a certificate of compliance to his standards for manufacture.

The balance of components in the assembly are commercial grade industry standard strut material, structural shapes, strut brackets, conduit, conduit straps, nuts, bolts, etc., whose properties are well defined by industry standards. Again, certificates of compliance are provided by the manufacturers to document the qualities of this material.

Calling for the same industry standard components in raceway systems for both Class 1E and non-Class 1E circuits precludes the change of inadvertent misapplication of an unqualified piece in the qualified system. All raceway material undergoes receipt inspection and control level "D" storage, with ongoing storage inspection.

Qualification of the conduit and cable tray raceways for the Class 1E safety related circuits has been confirmed by analysis, and calculations verify the adequacy of the system based on the properties of the raceways (including tray) and support components. This instrumentation is on file in the project records.

The above positions are reinforced by 10CFR21 whereby commercial grade items are not basic components until after dedication. Commercial grade items are those ordered on the basis of specifications set forth in the manufacturer's published product description. Dedication occurs when that item is actually installed as a basic component.

Thus the raceway system carrying safety related Class 1E circuits is indeed qualified, and the substantiating design verification calculations and quality assurance documents are required and provided.

PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE

SEABROOK STATION, UNITS 1 & 2

NRC-SEB DESIGN AUDIT (3/29/82 to 4/2/82)

at

UNITED ENGINEERS & CONSTRUCTORS INC.

RESPONSE TO ACTION ITEM NO. 1(c) & (d), DATED 4/2/82

REF.: Cable Tray Design

QUESTION (Item 1 (c), Attachment B, NRC SEB Design Audit,
Sheet 10 of 10):

Provide any available test data which would further assure the structural integrity and functionality of the trays when subjected to SSE and other applicable loads.

RESPONSE:

The above question pertains to the methodology employed by UE&C in the design and analysis of cable trays which has been addressed by the responses to Item 5, Sheet 4 of 10.

QUESTION (ITEM 1 (d), Attachment B, NRC SEB Design Audit,
Sheet 10 of 10):

Check IEEE 344 applicable provision which may require additional bases for establishing non-safety related structural elements.

RESPONSE:

The methods used by UE&C for qualifying cable tray designs satisfy the requirements of IEEE 344.