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Reliability Evaluation of Prestressed Concrete Containment Structures

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Abstract

The probabilistic safety evaluation of a realistic unbonded prestressed concrete containment building subjected to combinations of static and dynamic loads is presented. Loads considered include dead load, prestressing, accidental internal pressure, tornado and earthquake loads. Pertinent load parameters are the occurrence rate, duration and intensity. These parameters are treated as random variables for most of the loads. Limit state probabilities conditional on a specific load combination are calculated using the analytical procedure developed at BNL, which makes use of the finite element method and random vibration theory. Lifetime limit state probabilities are calculated using a load coincidence formulation.

1. Introduction

The overall safety of prestressed concrete containment structures can be conveniently expresed in terms of a limit state probability. A limit state essentially represents a state of undesirable structural behavior. In general, it will depend on the characteristics of the structures and the loads that act on the structure. For a particular structural system, it is possible that more than one limit state has to be considered. For the unbonded prestressed concrete containment two limit states are considered. They are: ultimate flexural capacity of the reinforced concrete sections, and yielding in tension of the unbonded prestressing tendon. The ultimate flexural capacity of the reinforced concrete section is one of the critical failure modes for the earthquake load and tornado wind pressure. On the other hand, yielding of the unbonded prestressing tendon determines the failure probability under the membrane stresses induced by the internal pressure. The limit state probabilities conditional on the occurrence of each load combination are calculated using the procedure developed at BNL.[1] A load coincidence formulation is used to obtain the unconditional lifetime limit state probabilities under each load combination and for all possible load combinations.[1,2]

2. Containment Description

The containment consists of a cylinder with a shallow domed roof and a flat foundation slab, as seen in Fig. 1a. A plan view of the containment is shown in Fig. 1b. The containment is prestressed by an unbonded post-tensioning system. A concrete ring girder is provided at the intersection of the cylinder and dome. The wall thickness of the cylinder varies from 5.5 feet at the connection with the base mat to a thickness of 3.5 feet at 11.5 feet above the base. From this elevation to the elevation of spring line, at 164 feet above the base, the wall thickness is constant and equal to 3.5 feet. At the buttresses the cylindrical walls are 22 inches thicker to provide anchorage for the hoop tendons. The dome thickness is 2.75 feet.

The post-tensioning system consists of: (i) 216 equally-spaced vertical tendons anchored at the top surface of the ring girder and at the bottom of the base slab; (ii) a total of 555 hoop tendons anchored at six vertical buttresses and spaced 10.5 inches in the vertical direction; and (iii) three groups of 63 dome tendons oriented at 120° to each other for a total of 189 tendons anchored at the vertical face of the ring girder. Each tendon consists of ninety 0.25-inch diameter wires. The prestress in the tendons during post-tensioning is 0.80 f_{pu} where f_{pu} = 240,000 psi, is the ultimate tensile strength for the tendon wires. During the life of the structure, the effective prestress is $f_{pe} = 0.60$ f_{pu}, which accounts for the various types of losses.

Reinforcing steel is provided in the cylinder and dome to resist the strains due to shrinkage and creep. In addition, reinforcement is used at discontinuities to resist local stresses, e.g., at the intersection of the cylinder and base slab. Details of the containment reinforcement arrangement are shown in Table I for both the cylinder wall (including buttress) and the dome.

The mean value of the tendon yield strength is taken to be $\overline{f_{py}}$ = 222,500 psi. For the steel reinforcement, the mean yield strength is $\overline{f_y}$ = 72,2000 psi. For both tendons and reinforcement the modulus of elasticity is $E_s = 29 \times 10^6$ psi. The mean value of the ultimate uniaxial compressive strength for the concrete is taken to be $\overline{f_c'}$ = 6,812 psi, and the modulus of elasticity for concrete is $E_c = 4.3 \times 10^6$ psi.

A three dimensional finite element model of the containment is used for the analyses. The thin shell finite element is used for the cylinder and dome concrete shells, and the truss element is used for the unbonded prestressing tendons. These elements are described in the SAP-V computer code user's manual. The top and side views of the three dimensional finite element model for the containment are shown in Figs. 2a and 2b, respectively.

3. Containment Loads

Five loads are considered in this study. They are: the dead load (D); the prestressing load (T); the accidental internal pressure (P); the tornado load (W); and the earthquake ground acceleration (E). For the loads modeled as Poisson renewal load processes, the mean occurrence rate, duration and point in time distribution of the intensity are necessary.[1]

The dead load is the weight of the dome and the cylindrical wall. This load is obviously static and time-invariant, and it is assumed to be deterministic. The weight density of the prestressed concrete is taken to be 150 lb/ft^3.

The prestressing load is static and considered time-invariant. The load is also assumed deterministic. The effective prestress during the life of the structure is $f_{pe} = 0.60$ f_{pu}. In the stress analysis, this load is applied to the containment through an equivalent variation in temperature ΔT in the truss elements modeling the prestressing tendons. For a coefficient of thermal expansion $\alpha = 6.5 \times 10^{-6}$/°F it is $\Delta T = - f_{pe}/(\alpha E_s) = - 763.9$ °F.

The internal pressure is a quasi-static load uniformly distributed on the containment wall. It is idealized as a rectangular pulse with a duration μ_{dp} = 1,200 sec, and an occurrence rate $\lambda_p = 9.4 \times 10^{-4}$/year. The point in time distribution for P is assumed to be normal. The mean value for the internal pressure is $\overline{P} = 0.89\ P_n$, where P_n is the nominal design pressure, and its coefficient of variation is 0.12. For the nominal design pressure of 47 psi, the mean and standard deviation for the pressure load are 41.8 psi and 5.02 psi, respectively.

The earthquake ground acceleration is considered to act along one horizontal direction, the global Z-direction as shown in Fig. 1b. This load is idealized as a stationary Gaussian process with zero mean. Its power spectrum is the Kanai-Tajimi spectrum with $\omega_g = 5\pi$ rad/sec and $\zeta_g = 0.060$.[1] The seismic hazard for the site is considered to follow a type II distribution $F_A(a) = \exp[-(a/u)^{-\alpha}]$ with $\alpha = 4.44$ and $u = 0.025$. The minimum significant ground acceleration at the site is $a_o = 0.05g$, which corresponds to an occurrence rate for the earthquake load $\lambda_E = 4.61 \times 10^{-2}$/year. The maximum ground acceleration at the site is $a_{max} = 0.70g$. The peak factor relating the peak ground acceleration to the root mean square ground acceleration is taken to be 3.0. Finally, the expected duration of the earthquake load is $\mu_{dE} = 10$ seconds.

The tornado load is modeled as a quasi-static load. For the tornado, the occurrence rate, λ_W, the duration, μ_{dE}, and the probability distribution of the intensity are site specific. In this study, $\lambda_W = 3.91 \times 10^{-4}$/year and $\mu_{dW} = 23.2$ seconds are used. The pressure on the external surface of the containment resulting from a tornado is $p = c_p q$, where q is the dynamic pressure and c_p is the pressure coefficient. The dynamic pressure q is given by $q = 0.00256\ W$ where W is the square of the maximum wind speed V_T. The distribution of W is assumed to be $F_W(W) = 1 - 10 \exp(-0.0338\sqrt{W})$ for $W > 75^2\ (\text{mph})^2$ and $F_W(W) = 2.77 \times 10^{-3}\sqrt{W}$ for $0 < W < 75^2 (\text{mph})^2$. The pressure coefficient c_p for the given containment shape is obtained following the recommendations in Ref. 3. The direction of wind speed is the negative Z-direction. This external wind pressure is accompanied by an internal pressure drop of 1.5 psi.[1]

4. Limit States

A limit state, which describes the state of undesirable structural behavior for the structure, will, in general, depend on the characteristics of the structures and loads that are acting on the structures. For a particular structural system, it is possible that more than one limit state has to be considered. For the present unbonded prestressed concrete containment structure two limit states have been identified. They are: yielding failure in tension of the prestressing tendon and ultimate flexural capacity of the reinforced concrete sections. The tendon is considered to yield when its tensile stress becomes equal to the mean yield strength $\overline{f_{py}} = 222{,}500$ psi. The ultimate flexural capacity of the reinforced concrete section is reached when the concrete compressive strain at the extreme fiber of the cross-section becomes $\varepsilon_u = 0.003$. Yielding of the unbonded tendon is the limit state considered for the membrane stresses produced by the internal pressure. The flexural capacity of the reinforced concrete sections is one of the critical failure modes for the earthquake load and the tornado wind pressure.

5. Limit State Probabilities

The limit state probabilities for the prestressed concrete containment were obtained with the previously referred method. A summary of the results is shown in Table II. On the basis of the results shown in Table II, it may be said that the major contribution for the overall limit state probability for the containment comes from the load combination (D+T+E). For the load combination (D+T+W) both the conditional limit state probability and expected number of occurrences are much smaller than the corresponding values for (D+T+E). For the load combination (D+T+P+E), the conditional limit state probabilities are of the same order of magnitude as those for (D+T+E). However, since the occurrence rate for this load combination is small, its contribution for the overall limit state probability is not significant. The critical elements in the containment for each load combination are also shown in Table II. The limit state probabilities obtained for the prestressing tendons are virtually zero. The value of the internal pressure that causes yielding of the tendon is approximately 15 standard deviations above the mean pressure load. This, in part, explains the extremely small probability of tendon failure under this load.

6. Concluding Remarks

The probability-based method for the safety evaluation of seismic category I structures developed at BNL has been applied to the reliability assessment of a prestressed concrete reactor containment. For the example analyzed, and on the bases of the assumptions made, the most significant load combination in terms of limit state probability is the combination of dead load and prestressing with earthquake load. One advantage of the method is that it can be used to quantify safety margins for the prestressed concrete containment. The method can also be useful in the development of the load combinations for design, based on limit states and specified limit state probabilities.

7. References

/1/ Shinozuka, M., Hwang, H. and Reich, M., "Reliability Assessment of Reinforced Concrete Containment Structures", Nuclear Engineering and Design 80 (1984), 247-267.

/2/ Wen, Y.K., "Methods for Reliability of Structures Under Multiple Time Varying Loads", Nuclear Engineering and Design 60 (1980), 66-71.

/3/ Maher, F.J., "Wind Loads on Dome-Cylinder and Dome-Cone Shapes", ASCE, ST5, pp. 79-96, October, 1966.

NOTICE

This work was performed under the auspices of the U.S. Nuclear Regulatory Commission, Washington, D.C. The findings and opinions expressed in this paper are those of the authors, and do not necessarily reflect the views of the United States Nuclear Regulatory Commission or Brookhaven National Laboratory.

Table I. Reinforcement Arrangement.

Location	Elevation or Radius	Horizontal Outside Buttress	Horizontal Outside Wall	Horizontal Inside	Vertical Outside Buttress	Vertical Outside Wall	Vertical Inside
Cylinder	0'-5'	#10@7.25" 2-#11@7"	2-#11@7"	#10@12"	#10@12" #18@12" #11@24"	#18@12" #11@24"	#18@18"
	5'-11.5'	#10@7.25" #11@7"	#11@7"				
	11.5'-20'	#10@7.25" #10@12"	#10@12"	#10@18"	2-#10@12" #18@12"	#18@12" #10@12"	#10@18"
	20'-30'	#10@7.25"	#10@12"		#10@12"		
	30'-154'6"			None			None
	154'6"-164'	#10@7.25" #11@10.5"	#11@10.5"		#10@12" 2-#11@6"	2-#11@6"	2-#10@9"
	164'-175'6"			#10@12"			
Ring Girder	175'6"-184'	#10@12"			#10@12" 2-#11@6"		
	184'-194'6"				#10@12" #11@10"		#10@7"
Dome	62'10"-46' (Radius)	#10@12" (Right angle #10@12" (Meridional)			#10@12" (Right angle #11@8.0" (Meridional)		#10@7"
	46'-0' (Radius)	#10@12"		None	#10@12"		None

Table II. Limit State Probabilities.

Load Combination	Limit State	Conditional Limit State Probability	Limit State Probability	Critical Elements
D+T	--	0.	0.	--
D+T+P	Tendon	0.	0.	--
D+T+E	Flexure	3.28×10^{-4}	6.04×10^{-4}	7(Meridional Direction)
D+T+W	Flexure	8.55×10^{-10}	1.34×10^{-11}	7(Meridional Direction)
D+T+P+E	Flexure	3.84×10^{-3}	2.55×10^{-10}	31(Meridional Direction)
Overall	--	--	6.04×10^{-4}	--

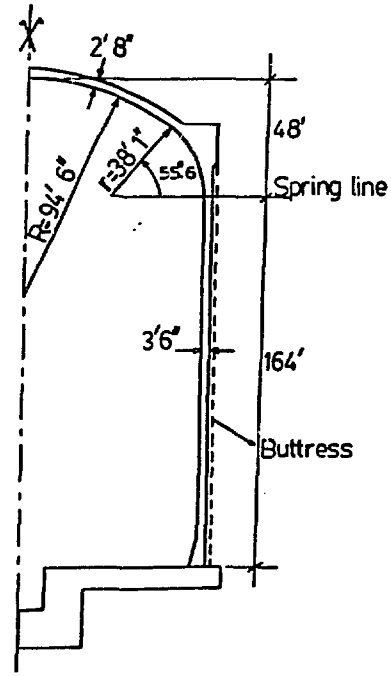

Fig. 1a. Vertical Cross Section of Containment

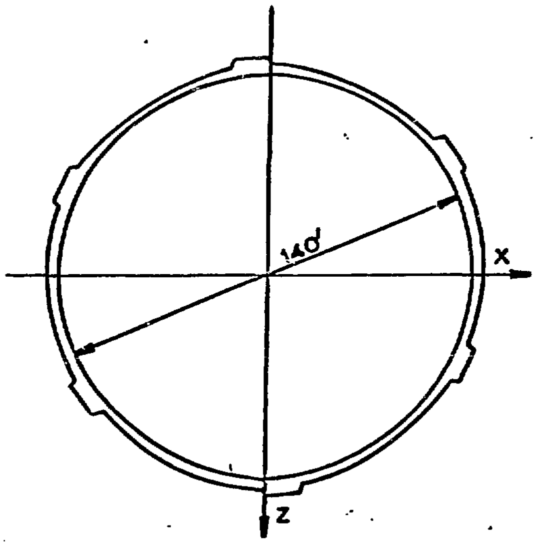

Fig. 1b. Plan Cross Section of Containment

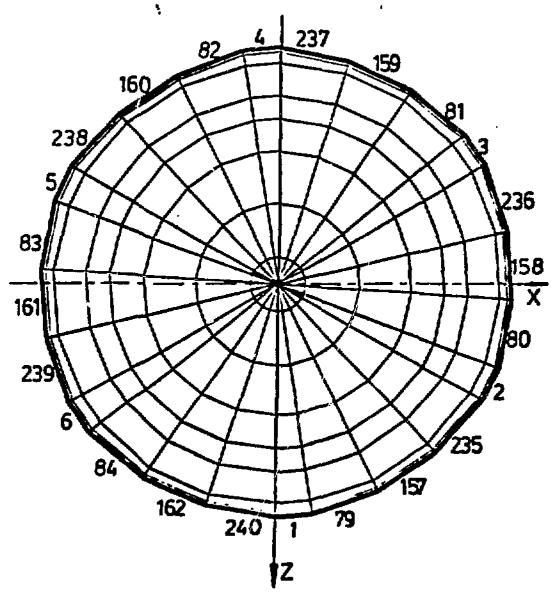

Fig. 2a. Finite Element Modal - Top View

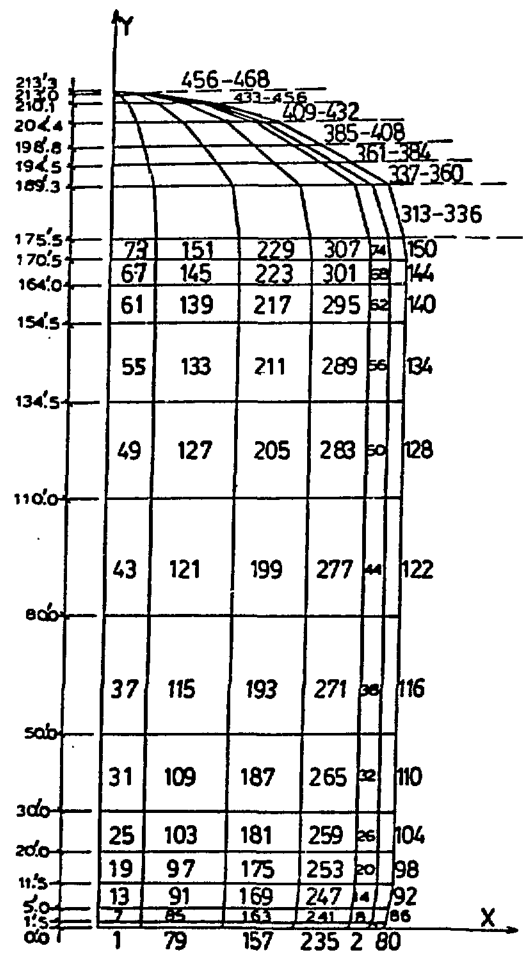

Fig. 2b. Finite Element Model - Side View