SEISMIC ANALYSIS OF REACTOR INTERNALS OF NARORA ATOMIC POWER PLANT, INDIA

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SYNOPSIS

This study describes the structural system that has been evolved for reactor internals. The internal structure of a heavy water reactor plant located in a mild seismic zone is usually kept independent of the cylindrical containment structure and is quite flexible under lateral vibrations. If such a system is adopted in a moderate or severe seismic zone, the absolute as well as inter-floor seismic displacements are found to be too large to be withstood by the equipment and piping systems. Therefore the structural system in the latter case has to be specially designed. This paper presents the results of the preliminary earthquake analysis of the original and the redesigned structural systems. Suitable assumptions have been made to represent the internal structure as a multimass system and its dynamic behaviour has been obtained for a prescribed raft motion.

INTRODUCTION

The Narora Atomic Power Plant (NAPP) is the first such station in India lying in a moderately seismic zone and having alluvial soil foundation, all earlier ones having been located in low seismic zones. Like others it will have a heavy water reactor. The main characteristics of the reactor building structural system have been

(a) a common raft foundation for the containment structures, calandria vault and internal framework which have been kept structurally independent from each other permitting lateral movement under temperature or pressure changes,

(b) the heavy pieces of equipment have been suspended through spring controlled steel hangers from floors above and restrained through bellows from below so as to have controlled movements under varying temperature environment. The resulting internal structure therefore would be rather flexible under lateral loads but it was not a problem in previous plants where the seismicity was low.

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For NAPP as a first step a similar structural system was naturally thought of as shown in Fig. 1. But it had to be changed to a stiffer system since the resulting seismic displacements could not be accommodated in the design of piping system and equipment supports. This paper describes the salient features of the structural changes and the resulting seismic displacements and forces.

SEISMIC RESPONSE OF ORIGINAL STRUCTURE

The Structure: The original proposal is shown schematically in plan and section in Fig. 1. The elevations are given taking ground level as 100.00 m. The internal structure consists of rigid walls and floors upto El. 95.50m above which it branches off into two separate sub-structures, the calandria vault and the various operating floors. For convenience of reference the floors may be named as fuelling, pump, boiler and top floors. The end of floor girders at boiler and top floors rest on the containment wall brackets through roller bearings. Besides the steel columns there are shielding walls also present in both directions below the pump floor which act as shear walls and make the framework much more rigid as compared to that above this floor. The inter-storey stiffness worked out in the N-S direction of the building are presented in Table 1. It is seen that the stiffness is very large below El 112.30 as compared to the two storeys above it. Therefore for preliminary analysis, the system could be treated as a two degree freedom system with masses lumped at El. 139.35 and 124.70.

Response: At the raft level, a modified El Centro motion was applied (See Fig. 2). The floor displacements were worked out taking damping as 2% of critical. Table 2 shows the values in the first and second modes for three conditions (a) boilers supported from top floor, (b) boilers supported at boiler floor and (c) column sizes increased to the largest rolled sections available. The absolute as well as inter-storey displacements are seen to be large in all the three cases indicating that more substantial change was required to cut down the seismic lateral displacements. Diagonal bracings or shear walls could not be provided at all the necessary places because of functional requirements of unobstructed space.

SEISMIC RESPONSE OF MODIFIED STRUCTURE

The modified structure utilizes the large lateral rigidities of the inner containment perimeter walls for providing stiffness to the internal framework. In this scheme, the top floor (El. 139.35) is deleted altogether, and the boiler floor (El 124.70) is connected to the inner containment perimeter wall as shown in Fig. 3. The boilers are supported on the boiler floor. This floor takes the vertical support from the cylindrical wall.

The seismic analysis of the structure was carried out in two steps. First the raft motion was derived from the free field motion by considering the response of the structure as a whole and taking the soil-structure interaction into account (3). In this formulation the stiffness properties of the inner and outer containments were considered through a lumped multimass model and the internal structure upto El 112.30 as well as calandria vault were taken as weights connected to the raft through a rigid link. The resulting timewise accelerations at the centre of gravity of the raft are shown in Fig. 4. In the second step, the raft motion has been taken as the base motion for the internal structure.

The internal system is idealised as a fork type structure with one stem and two branches as shown in (Fig. 5) model-1. The stem represents the rather rigid part from the raft to the El. 95.10 where the two branches take off. One branch consists of the calandria vault and the other includes the floors, columns and walls from El 95.10 to El 112.30. The whole system is represented by lumped masses and stiffness properties at various elevations are given in Table 3. The idealisation is done separately along the two principal axes of the structure in N-S and E-W directions. In computing the stiffness some approximations have to be made due to the complicated shape of the walls, openings, relative thickness of floors and walls and the like, as follows:

(a) Walls have been considered as shear - bending cantilevers, the restraint due to relatively thin floors being neglected.

(b) The effect of cross-walls is considered like that of flanges of T-beams, that is, four times the thickness is considered as width of flange if it lies only on one side and twelve times is considered if the cross wall lies on both sides.

(c) For computing shear deformation, the gross area of wall without flanges is considered, the shape factor being taken as 1.0 for flanged sections and 1.2 for rectangular sections.

In computing masses, only 50% of the live load is considered to be present on the floors at the time of earthquake. The damping in each mode is assumed as 7% of critical damping. For the determination of total earthquake response, quadratic mode super-position method is adopted. As an alternative model, the calandria vault is treated as a single lumped mass connected to El 95.10 as shown in (Fig. 5) model - 2.

RESULTS

The time periods of the structure corresponding to two models are given in Table 4. It is seen that for vibration in E-W direction, the time periods do not change appreciably whereas in N-S direction there is considerable change of time periods. therefore, the flexibility of calandria vault should not be neglected. The mode shapes are shown in Fig. 5 for both the models in N-S direction. The peak floor accelerations considering the flexibility of calandria vault are tabulated in Table 5. It is seen that in both the directions the peak floors accelerations increase at higher floors. Using this model, the shears, moments and deflections were computed by the quadratic superposition method.

CONCLUSIONS

The significant points brought out by this study of a reactor building to be constructed in moderate seismic area are as follows:

1. The internal structural design adopted in a non-seismic area turned out to be too flexible giving rise to excessive total as well as inter-storey displacements under the design basis earthquake.

2. The whole internal structural system needed a complete conceptual change to make it laterally stiff rather than flexible. The required stiffness could not be achieved by providing diagonal bracing or filler walls due to practical limitations.

3. A branched lumped mass model incorporating the stiffnesses of the columns, walls and floors as well as that of the calandria vault is found to be appropriate for the seismic response analysis of the internal structure.

REFERENCES

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TABLE 1 - LUMPED MASS AND STIFFNESS FOR VIBRATIONS IN NS DIRECTION

Elevation	Height between floors	Boiler Supported by floor at El. 139.35		Boiler supported by floor at El. 122.70		Max. available columns section with boiler supported at El. 122.70	
	(m)	Lumped mass (Kg-s^2/cm)	Stiffness (Kg/cm)	Lumped mass (Kg-s^2/cm)	Stiffness (kg/cm)	Lumped mass (Kg-s^2/cm)	Stiffness (Kg/cm)
139.35		1180		102		102	
	16.65		2020.0		2020.0		4420
122.70		3940		4775		4775	
	10.40		25920.0		25920.0		36480
112.30		3010		3010		3010	
	12.30		743000.0		743000.0		743000.0
100.00							

TABLE 2 - TIME PERIODS AND FLOOR DISPLACEMENTS FOR VIBRATION IN NS DIRECTION

MODE	Mass point at floor Elevation	Boiler supported by floor at EL. 139.35		Bioler supported by floor at EL. 124.70		Using maximum size of columns available boiler at floor EL. 124.70	
		Time period (sec)	Displacement (cm)	Time period (sec)	Displacement (cm)	Time period (sec)	Displacement (cm)
FIRST	139.55	5.025	229.50	2.740	67.15	2.290	45.64
	124.70		19.36		53.85		37.55
SECOND	139.55	2.340	-10.82	1.400	-5.29	0.945	-0.54
	124.70		35.08		0.16		0.02

TABLE 4 - TIME PERIODS

MODE NO	CALANDRIA VAULT TAKEN AS MASS ONLY		CALANDRIA VAULT TAKEN AS SUB-STRUCTURE	
	VIBRATION IN EW DIRECTION	VIBRATION IN NS DIRECTION	VIBRATION IN EW DIRECTION	VIBRATION IN NS DIRECTION
	T	T	T	T
1	0.1493	0.0906	0.1569	0.1214
2	0.0575	0.0316	0.0853	0.0923
3	0.0327	0.0196	0.0561	0.0367
4	0.0252	0.0112	0.0435	0.0304

T - Time period in seconds

TABLE 3 - PROPERTIES OF THE DYNAMIC MODEL

Sl. No.	Elev. of Node point	Weight Lumped at Node	Height of segment	EW DIRECTION Moment of inertia	EW DIRECTION Effective area	NS DIRECTION Moment of inertia	NS DIRECTION Effective area
		(Tonnes)	(m)	(m^4)	(m^4)	(m^2)	(m^2)
1	112.000	3480.0					
			4.950	1946.60	14.49	12004.00	89.05
2	107.050	2290.0					
			4.155	1946.60	14.49	12004.00	89.05
3	102.895	2090.0					
			3.505	1562.90	44.10	14610.00	70.17
4	99.390	4690.0					
			4.695	3315.00	51.76	14610.00	70.17
5	94.695	6170.0					
6	112.300	433.0					
			1.780	1232.00	28.10	357.23	30.59
7	110.520	751.0					
			3.200	1303.87	34.10	388.78	34.76
8	107.320	713.0					
			2.100	1278.36	38.71	265.85	38.71
9	105.220	548.0					
			2.720	1229.50	34.16	213.15	34.16
10	102.500	548.0					
			2.100	1278.36	38.71	265.85	38.71
11	100.400	587.0					
			2.500	1303.87	34.10	388.78	34.76
12	97.900	663.0					
			1.780	1575.40	84.80	472.70	84.80
13	96.120	411.0					
			1.425	1303.87	34.10	388.78	34.76
14	94.695	6170.0					
			7.095	5870.00	116.72	15336.00	116.64
15	87.600						

TABLE 5 - PEAK ACCELERATIONS

Floor Elevation	PEAK ABSOLUTE ACCELERATION cm/s^2	
	Vibration in NS Direction	Vibration in EW Direction
112.300 (Top of calandria)	402.6	361.0
112.000	336.1	444.8
107.050	332.6	375.8
102.095	329.0	352.9
99.390	325.6	341.6
94.695	320.1	325.4
C.G. of raft	314.0	314.0

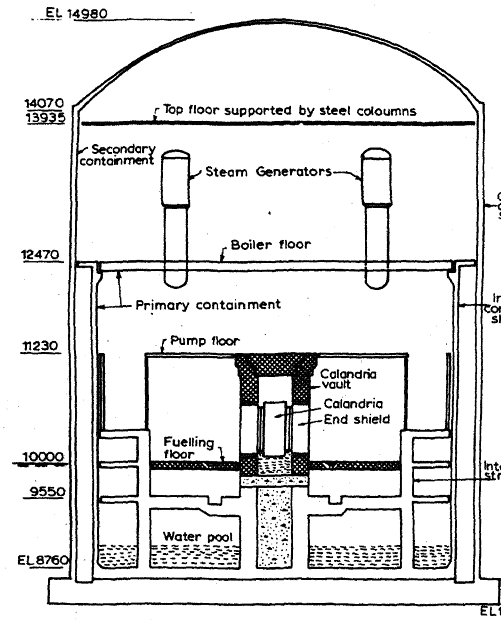

FIG. 1a _ SECTIONAL ELEVATION OF ORIGINAL STRUCTURE ABOUT XX

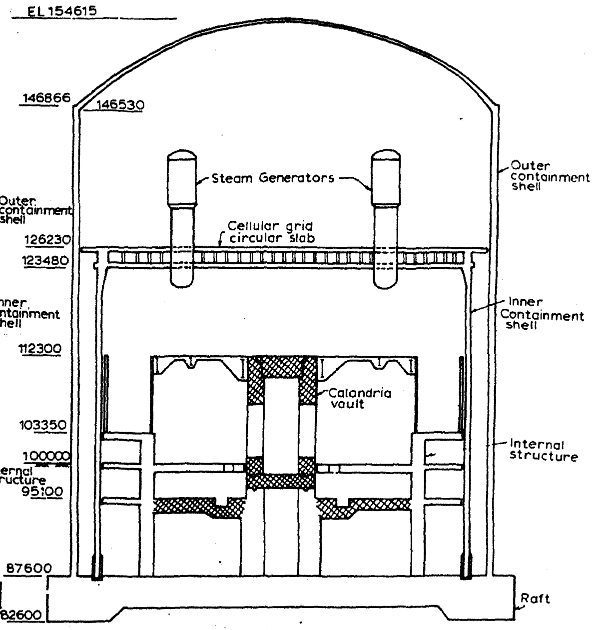

FIG. 3 _ SECTIONAL ELEVATION OF EVOLVED STRUCTURE ABOUT XX

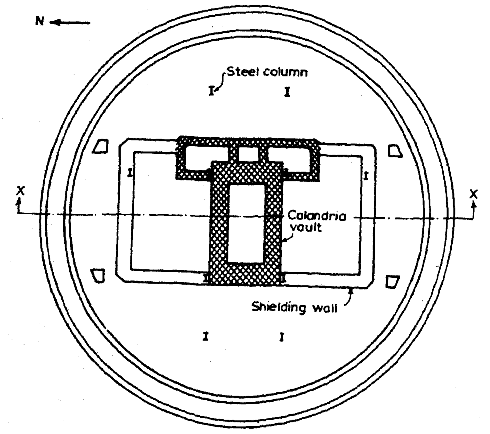

FIG. 1b _ PLAN AT EL 9550 OF ORIGINAL STRUCTURE

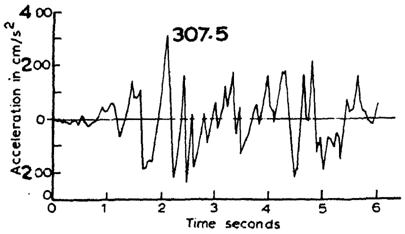

FIG. 2 _ FREE FIELD GROUND MOTION (MODIFIED EL CENTRO)

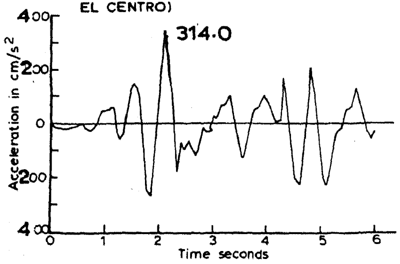

FIG. 4 _ TIMEWISE MOTION AT RAFT

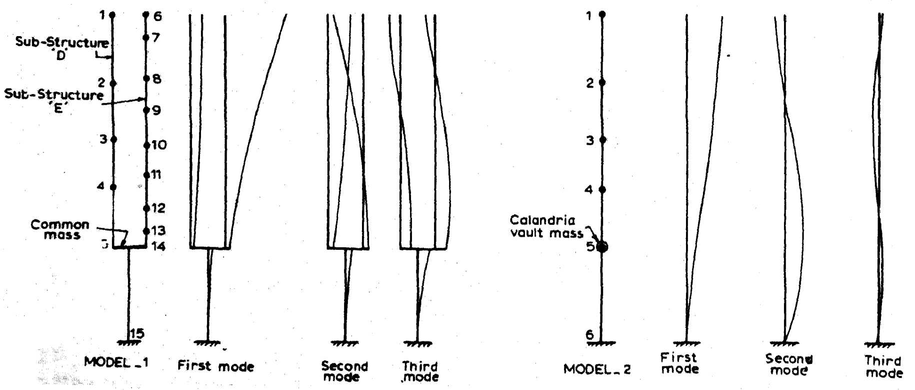

FIG. 5 _ MODE SHAPES MULTIPLIED BY MODE PARTICIPATION FACTOR FOR VIBRATION IN N-S DIRECTION