SEISMIC ANALYSIS OF REACTOR BUILDING OF AN ATOMIC POWER PLANT ON ALLUVIAL SOIL

Anand S.Arya[I], A.R.Chandrasekaran[I], S.K.Thakkar[I], D.K.Paul[I] and
A.D. Pandey[I]

SYNOPSIS

This paper describes the determination of base-raft motion of a reactor building consisting of three sub-structures i) outer containment ii) inner containment and iii) internal structures. The building is modelled as a multimass branched system. The soil on the sides and at the base is replaced by linear springs. The representation of the internal structural system as a single mass at the end of the rigid link is considered adequate for purposes of determining motion at the raft level. A parametric study is carried out in which shear wave velocity, modulus of subgrade reaction, weight of the internal structure and height of C.G. of the internal structure above raft are varied over a practical range to determine their influence on the earthquake response.

INTRODUCTION

A nuclear reactor building in India consists of two coaxial inner and outer containment shell and reactor internals, all resting on a massive raft foundation. The external cylindrical shell is capped by a spherical dome while the internal shell carries a cellular grid floor. In the mathematical modelling for determining the raft motion caused due to free - field earthquake motion, the entire reactor building can be considered to be composed of three sub-structures A, B and C. Sub-structure A consists of the outer cylindrical shell and the spherical dome over it, sub-structure B consists of internal cylindrical shell and the cellular grid over it and substructure C includes the reactor internal structural system and the raft. The details are shown in Fig. 1a. Various elevations are marked with respect to ground whose level is taken as 100.0m. The soil below and on the sides of the outer containment shell is represented by elastic springs as shown in Fig. 1b. Transfer matrix approach has been used to determine the frequencies and mode shapes. The timewise motion at the raft level has been obtained by algebraically adding the modal responses due to the chosen free field motion. In view of many uncertainties involved in the parameter values, particularly at the preliminary design stage, the values of shear

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[I] School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee, INDIA.

modulus of soil, modulus of subgrade reaction, internal structure and component weights, etc., have been varied within a practical range to investigate the sensitiveness of the periods of the structure to such changes.

MATHEMATICAL MODEL

The mathematical model of the reactor building consists of a branched lumped mass system of the three cantilever substructures connected to the base level of the raft as shown in 1(b). Sub-structure A is divided into 25 parts above the top of raft with 26 lumped masses counting from top. The bottom mass no. 26 is connected to the base of the raft by a rigid link of height equal to thickness of raft. The stiffness of soil in the embedment portion is represented by linear springs as described later. Sub-structure B is divided into 21 parts above the top of raft. The number of masses is 22, the bottom mass being connected to the base of the raft by a rigid link as in sub-structure A. The internal structure being rigid and light as compared to the total weight of the building including raft, and its centre of gravity being close to the base, the sub - structure C is assumed to be lumped at a single point situated at the c.g. of the total mass and connected to the raft by a rigid link. The raft mass is lumped at the c.g. of the raft and is connected to the base of raft by another rigid link.

<u>Soil Stiffness</u>: For representing the soil on the sides and at base, equivalent elastic springs are introduced at appropriate levels. The stiffness of side springs is calculated on the basis that the stiffness of sandy soils varies with depth below the ground level in a linear manner represented by the coefficient of modulus of subgrade reaction n_h(4) as shown in Fig. 2a. The side soil stiffness k_{xs} at any depth x below ground level is given by

$$k_{xs} = n_h x \qquad \dots (1)$$

The stiffness of springs at various levels then consists of lumping the stiffness at appropriate mass levels as shown in Fig. 2(b).

The base spring stiffness is evaluated on the basis of the consideration of the soil as an elastic half space (Richart and Hall, 1961). The translational spring K_x and rotation spring K_θ can be obtained from the following equations (3).

$$K_x = \frac{32(1-\nu)Gr_o}{7-8\nu} \quad \text{and} \quad K_\theta = \frac{8\,G\,r_o^3}{3\,(1-\nu)} \qquad \ldots (2)$$

where G = Shear modulus of soil given by $\gamma v_s^2/g$, γ = unit weight of soil, v_s = shear wave velocity, ν = poisson's ratio for soil, r_o = radius of raft foundation at the base and g = gravity acceleration. At the base level of the raft, the total stiffness of translational spring will be equal to the sum of lateral stiffnesses obtained on the sides at this level obtained from Fig. 2(b) and that obtained from Eq. (2).

METHOD OF ANALYSIS

Transfer matrix technique has been used to determine the frequencies and mode shapes. The deformations ~~due to~~ bending and shear and the effect of rotatory inertia have been included. The boundary conditions at the top and the base are satisfied to get the frequency determinant. By finding the zeros of the determinant the frequencies are obtained and then mode shapes are determined.

PARAMETERS CONSIDERED

The various parameters like the modulus of rigidity of the base soil G, modulus of sub-grade reaction n_h of side soil, weight of the internal structure W_t, height of C.G. of internal structure above base of raft h have been varied as follows.

$G = 0.5\,G_s,\ 1.0\,G_s,\ 4.0\,G_s,\ 8.0\,G_s,\ 40.0\,G_s$

where $G_s = 8155\ t/m^2$ which corresponds to a shear wave velocity of 200 m/sec, $\gamma = 2.0\ t/m^3$ and $\nu = 0.33$

n_h = 750, 1000, 1500, 2000 t/m^3

W_t = 20000, 25000, 30000 t

$\bar{h}$ = 12, 15, 18, 21 m

In addition for one case, the parameters considered are

$G = 8155\ t/m^2;\ n_h = 2000\ t/m^3;\ W_t = 24000t,\ \bar{h} = 18.0m$

RESPONSE EVALUATION

The mode superposition method of analysis is used to determine total response of the system. Since bulk of the energy is concentrated in the first few modes of vibration, the total response is approximated by the resultant response of first six modes. The modal damping in each mode is evaluated from the damping of the various materials on the basis of strain energy stored in various materials. The damping values adopted for various individual materials under Safe Shutdown Earthquake (SSE) condition is as follows

Item	Damping Percent of Critical
Inner Containment (Prestressed concrete)	5
Outer Containment (Reinforced concrete)	7
Soil (in Rocking mode)	11
Soil (in Translational mode)	35

DATA AND RESULTS

The weight lumped at various nodes, the cross sectional area and moment of inertia of various segments are given in Table 1. The fundamental period of the system is presented in Table 2 for various combination of parameters. Figure 3 shows the design basis earthquake (SSE) free field motion (1). Table 3 gives the effective damping obtained in each mode for one of the cases and also the strain energies stored in the modes indicating the effectiveness of each sub-structure or soil in each mode. Figure 4 shows the derived raft motion by timewise super-position of first four modes for this case. The results are discussed below:

Time Period: The time period T_o naturally decreases as the shear modulus of soil G increases from 4078.0 to 326,200 t/m^2. When G is very large, the time period tends to that of fixed base condition. The sensitiveness is more for softer soil with low value of G. The value of T_o is less sensitive to the changes in n_h, W_t and $\bar{h}$. Since W_t and $\bar{h}$ can be determined more precisely, small variation in them may be taken not to alter the base-raft motion to an appreciable extent. Influence of side soil on the response is also very small (see table 3).

Strain Energy and Damping: From Table 4 it is seen that the strain energy ratio in rocking soil spring is more than other springs in the first mode indicating that the first mode is predominantly rocking type. Similarly second mode is predominantly structural modes. The weighted damping factor given in the Table also point out to the above obsertation.

Raft Motion: Considering the first six modes of vibration the timewise response at the c.g. of the raft is obtained as shown in Fig. 4, the peak acceleration at raft level is seen to be 314.03 cm/s^2 as compared to 307.5 cm/s^2 of free field motion (1).

Structural Response: The displacements, forces, moments and shears have been determined in each mode of vibration and superposed by the method of quadratic super-position. The absolute peak accelerations obtained are 0.72g at top of dome (mass 1), 0.42g at grid floor level (mass 27), and 0.32g at c.g. of internal structure, that is, at mass 49.

CONCLUSIONS

The property of the base soil considerably influences the dynamic characteristics of the structure and hence these properties should be carefully investigated. If the soil is very firm, that is to say, the shear wave velocity more than 1200 m/sec, the structure may be assumed to be fixed at raft level. For the depth of embedment considered (about one-fourth the total height), the side soil has little influence on the periods. Similarly, small variations of the height and weight of the internal mass does not affect the periods significantly. The first mode is predominantly rocking type, second mode translational type and higher modes are structural modes. The peak acceleration of the raft works out to 314.03 cm/s^2 as compared to 307.5 cm/s^2 of free field motion.

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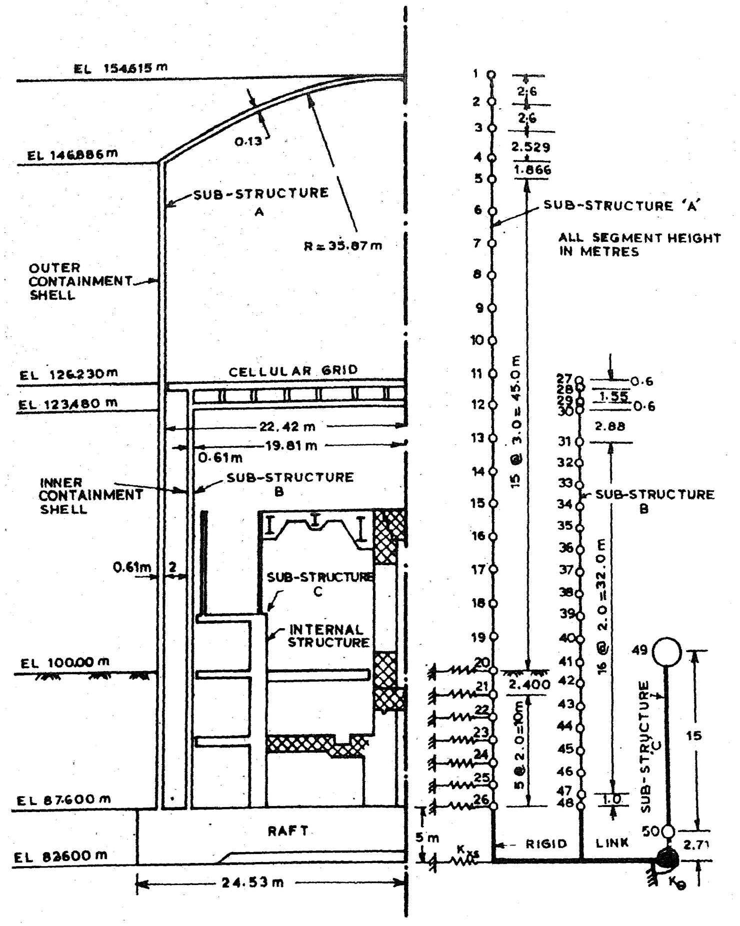
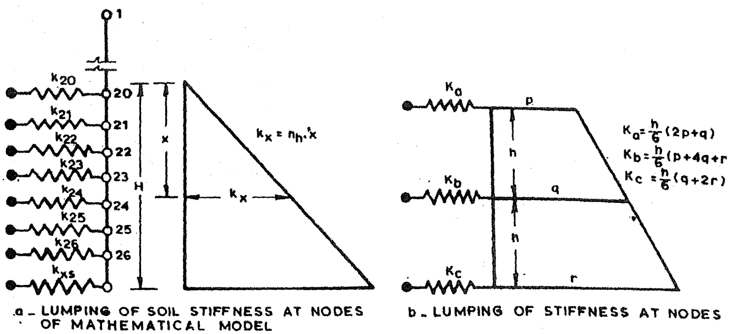


(a) Actual structure showing containments

(b) Mathematical model

FIG. 1 _ GENERAL VIEW OF REACTOR BUILDING AND MATHEMATICAL MODEL



a _ LUMPING OF SOIL STIFFNESS AT NODES OF MATHEMATICAL MODEL

b _ LUMPING OF STIFFNESS AT NODES

FIG. 2 _ DETAILS OF SOIL SPRING

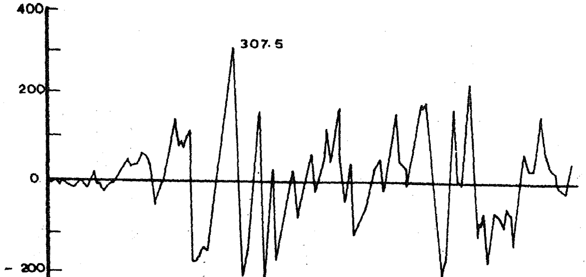


FIG. 3 _ FREE FIELD GROUND MOTION (MODIFIED EL-CENTRO)

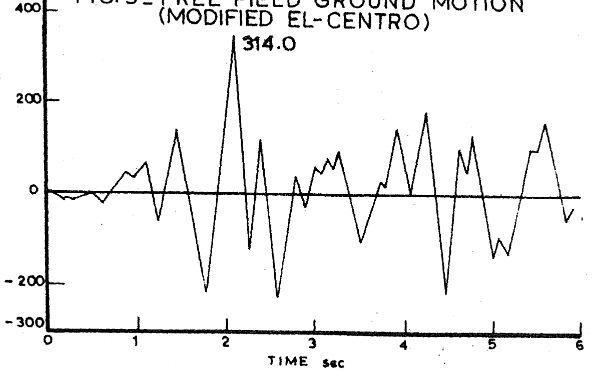


FIG. 4 _ TIMEWISE MOTION AT THE C.G. OF RAFT

TABLE 1 - WEIGHT, AREA AND INERTIA PROPERTIES

Node or Segment	Weight t	Segment Height m	X-section Area m^2	Moment of Inertia m^4
Substructure A				
1	60	2.60	30.5	1348
2	142	2.60	29.5	3880
3	210	2.58	29.7	6175
4	325	1.89	87.1	22483
5	510	3.00	87.1	22483
6	625	3.00	87.1	22483
7	625	3.00	87.1	22483
8	625	3.00	87.1	22483
9	625	3.00	87.1	22483
10	625	3.00	87.1	22483
11	625	3.00	87.1	22483
12	625	3.00	87.1	22483
13	625	3.00	87.1	22483
14	625	3.00	87.1	22483
15	625	3.00	87.1	22483
16	625	3.00	87.1	22483
17	625	3.00	87.1	22483
18	625	3.00	87.1	22483
19	625	3.00	87.1	22483
20	625	2.40	87.1	22483
21	460	2.00	87.1	22483
22	418	2.00	87.1	22483
23	418	2.00	87.1	22483
24	418	2.00	87.1	22483
25	418	2.00	87.1	22483
26	209	5.00	R	R*
Substructure B				
27	3500	0.60	1308.0	136000
28	1500	1.55	523.2	54400
29	1500	0.60	1308.0	136000
30	1777	2.88	77.1	15600
31	470	2.00	77.1	15600
32	385	2.00	77.1	15600
33	385	2.00	77.1	15600
34	385	2.00	77.1	15600
35	385	2.00	77.1	15600
36	385	2.00	77.1	15600
37	385	2.00	77.1	15600
38	385	2.00	77.1	15600
39	385	2.00	77.1	15600
40	385	2.00	77.1	15600
41	385	2.00	77.1	15600
42	385	2.00	77.1	15600
43	385	2.00	77.1	15600
44	385	2.00	77.1	15600
45	385	2.00	77.1	15600
46	385	2.00	77.1	15600
47	289	1.00	77.1	15600
48	96	5.00	R	R
Substructure C				
49	25360	15.00	R	R
50	20504	2.71	R	R

R* = Rigid Link

TABLE-2 FUNDAMENTAL TIME PERIOD FOR VARIOUS COMBINATIONS OF PARAMETERS

S.No.	G t/m^2	n_h t/m^3	W_t t	$\bar{h}$ m	Period s
1	$0.5G_s$	1000	25000	18	1.348 T_o++
2	G_s+	1000	25000	18	1.000 T_o
3	$4G_s$	1000	25000	18	0.551 T_o
4	$8G_s$	1000	25000	18	0.425 T_o
5	$40G_s$	1000	25000	18	0.297 T_o
6	G_s	750	25000	18	1.010 T_o
7	G_s	1000	25000	18	1.000 T_o
8	G_s	1500	25000	18	0.981 T_o
9	G_s	2000	25000	18	0.965 T_o
10	G_s	1000	25000	12	0.964 T_o
11	G_s	1000	25000	15	0.981 T_o
12	G_s	1000	25000	18	1.000 T_o
13	G_s	1000	20000	21	1.021 T_o
14	G_s	1000	20000	18	0.979 T_o
15	G_s	1000	25,000	18	1.000 T_o
16	G_s	1000	30,000	18	1.021 T_o

+ G_s = 8155 t/m^2 corresponding to v_s = m/s, γ_s = 2.0 t/m^3 and ν = 0.33;

++ T_o = 0.799 Sec.

TABLE - 3

TIME PERIOD (T) WEIGHTED DAMPING (D) AND SPECTRAL DISPLACEMENT (S_d) IN VARIOUS MODES

Mode No	T	D	S_d
1	0.767	0.179	4.660
2	0.305	0.280	0.773
3	0.208	0.060	0.618
4	0.091	0.068	0.092
5	0.068	0.070	0.047
6	0.053	0.053	0.026

TABLE - 4

STRAIN ENERGY RATIO IN VARIOUS MODE

Mode No	Rocking spring at base	Trans. spring at base	Trans. spring at side	Containment Structure A	Containment Structure B
1	0.6683	0.1795	0.1158	0.0226	0.0137
2	0.2165	0.6372	0.0816	0.0557	0.0090
3	0.0000	0.0046	0.0025	0.3829	0.6099
4	0.0811	0.0052	0.0002	0.5608	0.3528
5	0.0062	0.0035	0.0013	0.9197	0.0694
6	0.0062	0.0038	0.0002	0.0686	0.9212