Dynamic Analysis of Auxiliary Buildings in Nuclear Power Plants

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INTRODUCTION

All nuclear power plants have a large number of auxiliary buildings housing various services and control systems required for the operation of the plant. Illustrative examples are Turbine building, Control building, Service building etc. These buildings are seismically qualified as Class I or Class II structures. Usually, these auxiliary buildings are of low rise type with two or three floors and floor heights varying from five to eight metres and of framed construction in steel or concrete or a combination of both the materials. The floors are usually staggered with large cutouts and may not extend over the full area in plan. Some of the bays are often of double storey height with the columns continuous over a storey in order to accomodate cranes and other equipment. The structural elements supporting the roof may consist of steel roof trusses instead of beams. The seismic analysis of these structures involves the formulation of the analytical model that can simulate the physical behaviour of the structure as close as possible taking into consideration the practical aspects. The criteria adopted to formulate the mathematical model has an important bearing on the evaluated dynamic characteristics and seismic response. Hence, the development of an appropriate model needs considerable engineering judgement inorder to retain significant dynamic characteristics while at the same time optimising the effort involved in terms of input data preparation and computer costs.

CONVENTIONAL METHODS OF ANALYSIS

Conventional methods of analysis of such buildings are enumerated below:

i. Codal methods estimating the base shear approximately based on the number of floors / building height and distributing the base shear over the building height.

ii. Simplified methods of computation with one translational degree of freedom at every major floor level and assuming a shear beam type idealisation to compute stiffness. The time period is computed more accurately compared to (i) above. Using this, the base shear is estimated and distributed to the different floors.

iii. Mathematical model assuming the slabs are continuous at each floor and each floor having one translational degree of freedom. Joint rotations are considered to evaluate the dynamic stiffness matrix. This model ignores the presence of cutouts, etc. if they are present in a floor.

iv. Three dimensional framed model of the building neglecting the effect of slab inplane rigidity.

v. Three dimensional model as in (iv) above except that the slab inplane rigidity is considered indirectly by master and slave nodes.

vi. A BLOCK model of the structure which considers the effect of discontinuity of floors. One translational degree of freedom in the direction of seismic input per slab element is considered. The stiffness matrix is derived using the simplified "shear beam" idealisation. The analysis is in a two dimensional framework.

vii. A complete THREE dimensional model of the structure with the floor slabs idealised by plate elements.

Methods (i) and (ii) enumerated above are very simplified techniques and are inadequate for design of such complex auxiliary buildings. Method (iii) is inadequate as this ignores the effect of large cutouts present in the floor slabs. Method (iv) ignores the presence of floor slabs altogether though it is a three dimensional model. In method (v) the master node has to be judiciously chosen at the stiffness centre of the floor in order to avoid the effect of artificially introduced torsion if the master node is not correctly selected. Method (vi) is a very conservative model and hence not recommended. Method (vii) while being an appropriate three dimensional model representing the structural system is however very expensive both in terms of effort and computational time required for analysis. In view of frequent changes in the layouts of such buildings at the time of planning and consequent reanalysis required, a simplified yet acceptable method of analysis is necessary. The study reported herein desribes an analytical model for dynamic analysis which caters to the special features in these buildings while at the same time retains the idealisation in a two dimensional framework and hence is quite economical both in terms of effort and computational time.

DESCRIPTION OF THE ANALYTICAL MODEL

The structure is sub-divided into a series of parallel plane frames which are connected by a system of slabs. The entire building is idealised as a multiple spring mass system. Any joint in the frame can have three degrees of freedom viz. two translational degrees of freedom and one rotational degree of freedom in the plane of the frame. The mass of the structure is selectively lumped at the frame joints along the desired degrees of freedom. In addition to the lumped masses at the desired degrees of freedom of the frame, the concept of mass level is introduced which takes care of the rigidity of floor slabs connecting the frames. A common mass level number is specified to all the lumped masses in the structure (irrespective of their location in the different frames) which displaces together under seismic excitation. A common example of a mass level is a rigid slab spanning over several plane frames and connecting several dynamic degrees of freedom of each frame. The entire slab with all the joints of the frame that are interconnected by the slab moves as a rigid body in the horizontal direction. By defining the same mass level number for all the dynamic degrees of freedom in the horizontal direction, corresponding to the joints connected by the slab, the inplane rigidity of the slab is taken care of. Another example is an isolated beam such as a plinth beam which moves as a rigid element in the direction of its length. Each mass level defines one global dynamic degree of freedom in any of the three directions, viz. horizontal and vertical translations and rotation. The total number of mass levels defines size of the problem. The generation of stiffness matrix for each frame (which may or may not be amenable for geometry generation) with all the special features such as trusses, springs, hinges, walls, members of different materials, prismatic or non-prismatic sections, hinged or rigid ends, shear deformations, finite sized joints, etc. proceeds in the usual direct stiffness approach.

ANALYSIS PROCEDURE

The analysis procedure can be described in the following steps:

i. Formulate the mathematical model on the lines indicated above.

ii. For each frame,
   a. Define geometry, properties, restraints, masses, and mass levels.
   b. Form stiffness and mass matrix.
   c. Perform static condensation of the stiffness matrix.
   d. Assemble the frame condensed stiffness and mass matrix in the global stiffness[K] and mass [M] matrices using the the mass level number as the global dynamic degree of freedom number.
   e. Repeat (a) to (d) above for all the frames.

iii. Extract eigen values and vectors of the dynamical matrix $[M]^{-1}$ [K].This yields natural frequencies and mode shapes for the structural system.

iv. Perform a reverse condensation on the frame stiffness matrices to obtain the modeshape coefficients for the degrees of freedom at the nodes of the frame not associated with a lumped mass. Calculate weighted damping using the mode shape coefficients, material damping ratios, and member stiffness matrices. (Whitman,1972)

v. Carry out response spectrum analysis using the computed modal damping to obtain global modal displacements. The global stiffness and mass matrices, natural frequencies and mode shapes can be used for obtaining response history, which in turn is used for floor response spectra generation.

vi. For each frame, impose the same modal displacements on all the joints connected by a mass level. Calculate modal forces on the frame joints using the condensed stiffness matrices.

vii. Calculate stiffness and mass centers, eccentricities at all the mass levels. Compute the torsional corrections to the joint forces as per standard procedures.

viii. Perform static analysis for each frame using computed modal joint forces along with other static loads.

ix. Combine the modal member stress resultants in SRSS or CQC method to obtain values of member forces.The CQC method is adopted in case of closely spaced modes. (Wilson et al,1981)

x. In the SRSS or the CQC method of modal combination at the member forces level the sign of the member forces is lost. For design purposes and also in the case of design of combined foundations and rafts a set of consistent forces is desirable. Since the first mode generally has the maximum participation, the signs of the forces obtained by the SRSS/CQC method is assumed according to the signs of the member forces obtained for the forces in the first mode, and equilibrium checked. The seismic load case results are combined with results of other load cases to yield final design forces.

RESULTS

The method described above has been used to develop a computer code for the analysis of the auxiliary buildings of the 500MWe PHWR in India. The structures include Turbine building, Control building, Station and Reactor Auxiliary buildings, and Service building blocks. For the purpose of validation, some of these structures are analysed by using the general purpose program SAPIV.(Bathe et al,1974) The important dynamic characteristics are shown in Table I.

SAMPLE PROBLEM: A rectangular structure of 2 bays x 2 bays with two floors and a roof is selected for validation of the method. (fig 1). The ground floor does not have slab in the first bay and the first floor does not have a slab in the second bay. The members and lumped masses are chosen in a very arbitrary manner to represent a very distorted stiffness and mass distribution. The building has been analysed using a full FEM model and the present model, the results of which are shown in Table II. The comparision of the dynamic and static characteristics shows good conformity between the two methods.

CONTROL BUILDING: This is a rectangular building of 45 m x 64 m with two floors above ground. (fig 2). The building has been analysed with the present model and also SAPIV. Comparative results for a typical frame are shown in Table II. The total base shear as also the results of a typical frame for both the methods compare quite favourably.

TURBINE BUILDING: This is a rectangular building of 112 m x 35 m concrete external columns and steel internals. (fig 3). The roof is made of steel trusses and precast cover (non-rigid slab). There is a large central cutout in the floors spanning 6 bays to accommodate the TURBINE GENERATOR pedestal. The steel internal beams in the longitudinal direction are connected by hinges to the columns. Foundation levels are uneven to accommodate the underground pipe entry. The building is divided into two blocks with an expansion joint on axis 6a. The results of analysis of one block is shown in Table II along with a comparison of results of SAPIV. Shear beam idealisation of the building indicated the base shear higher by about 150%. The enormous difference is due to non consideration of rotation of the joints.

CONCLUSIONS

Experience in applying the present model to several of the auxiliary buildings of India's first 500 MWe PHWR plant shows that the model is highly successful in predicting the dynamic charateristics of these buildings. The major advantage is the facility to handle the input in the form of individual frame data. Any change in the structure layout affects the data of only the frames where modifications have been effected. This model,wherein the degrees of freedom are reduced from a full 3-D space model, enables the eigen solution and evaluation of response history quite economical both in terms of engineeering labour and computational time, while at the same time maintaining the desired accuracy. When compared to simpler models like the "shear beam" model, the degree of conservatism is reduced at a marginal increase in computational effort.

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REEFERENCES

Bathe, K.J., Wilson, E.L.,Peterson, F.E..(1974). SAPIV-A Structural Analysis program for Static and Dynamic Response of linear systems. Earthquake Engineering Research Center, EERC 73-11.

Whitman, R.V.(1972). Soil Structure Interaction. Seismic design of Nuclear Power plants. Ed. Robert J Hansen. MIT Press, England.

Wilson, E.L., Der Kiureghian and Bayo, E.P.(1981).A Replacement for the SRSS method in Seismic analysis. Journal of Earthquake Engineering & Structural Dynamics, Vol 9,pp.187-194.

TABLE I
COMPARISON OF DYNAMIC CHARACTERISTICS

Characteristics	Sample problem SAPIV	Sample problem Present Model	Control Bldg SAPIV	Control Bldg Present Model	Turbine Bldg. SAPIV	Turbine Bldg. Present Model
Natural Period 1 (sec)	0.7843	0.7905	0.4054	0.3923	3.371	3.292
Part. factor 1	1.362	1.359	1.4208	1.4320	1.620	1.605
BASE SHEAR(T)	79.1	84.5	8233.0	8431.0	1075.0	946.8
CPU TIME(Apollo) (sec)	450	40	7200	900	3600	240

TABLE II
COMPARISON OF FORCES IN SOME TYPICAL MEMBERS

BEAMS

Member No	Present Model Shear(t)	Present Model Moment(t-m)	SAP IV Shear(t)	SAP IV Moment(t-m)
SAMPLE PROBLEM				
1	17.68	81.37	19.15	82.70
2	16.49	63.95	17.40	63.53
3	22.92	18.37	21.55	17.10
4	10.44	28.55	10.42	28.51
5	15.75	28.55	14.61	28.57
CONTROL BUILDING(Frame D)				
1	30.99	73.0	29.6	69.9
2	79.99	186.8	77.5	181.0
3	19.10	95.6	18.6	93.2
4	169.8	419.6	168.0	415.1
5	48.4	242.4	47.9	239.6
6	47.3	108.7	47.3	108.6
7	10.6	53.1	10.5	53.0

TABLE II (CONTD)

COLUMNS

Member No	Present Model Axial (t)	Present Model Shear (t)	Present Model Moment (t-m)	SAP IV Axial (t)	SAP IV Shear (t)	SAP IV Moment (t-m)
SAMPLE PROBLEM						
6	17.68	15.25	72.53	18.88	16.04	77.46
7	17.68	14.40	67.06	16.94	14.83	70.51
8	33.30	17.50	46.86	34.84	18.54	45.63
9	6.45	16.22	27.94	6.39	15.82	27.22
10	39.71	18.55	44.25	38.72	19.28	42.94
11	42.82	20.54	135.03	44.06	21.39	137.40
12	11.54	11.43	43.26	11.41	11.69	43.97
13	54.19	22.11	138.48	52.84	22.83	140.60
CONTROL BUILDING(Frame -D)						
8	30.9	24.0	73.0	30.0	22.8	69.9
9	9.0	35.0	104.8	17.8	28.0	77.0
10	110.6	53.6	126.6	107.7	52.0	124.3
11	6.3	83.5	204.9	6.3	80.9	200.0
12	278.0	96.7	282.0	274.0	96.8	283.0
13	22.0	104.0	313.0	22.0	104.0	314.0
14	324.0	106.0	189.0	320.0	106.0	189.0
15	26.0	98.0	197.0	26.0	98.0	197.0
TURBINE BUILDING						
1	0.18	56.08	1171.13	0.23	56.01	1086.0
3	0.0	3.76	24.32	0.04	3.64	23.87
4	0.0	4.22	26.18	6.59	4.46	27.31
6	0.15	49.85	1447.20	0.60	57.02	1575.0
7	0.18	153.41	1602.60	0.24	144.50	1578.0
12	0.15	70.07	1273.71	0.24	59.36	1148.00
13	0.18	45.49	890.0	0.24	38.54	929.80
16	0.0	0.16	1.16	2.09	0.16	1.17
23	0.0	36.31	330.36	0.0	37.75	295.70
24	0.0	40.37	367.42	0.0	38.17	293.10

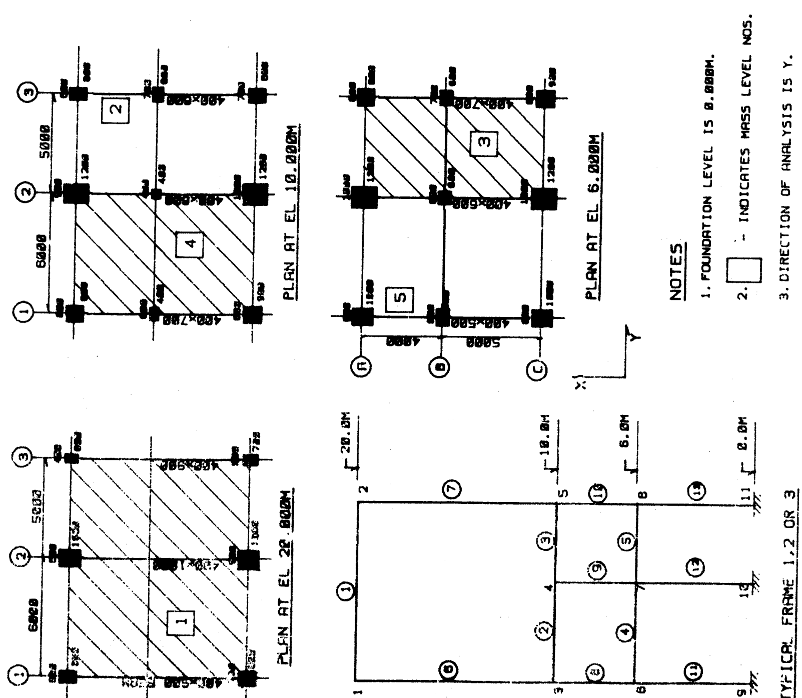
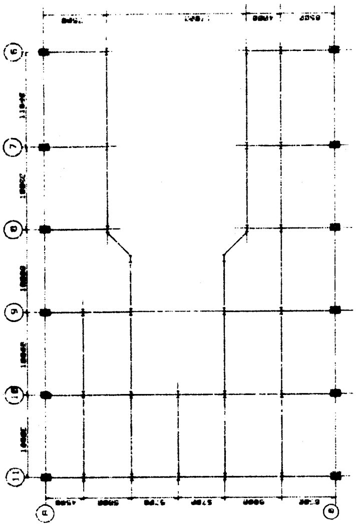


FIG-1 SAMPLE PROBLEM



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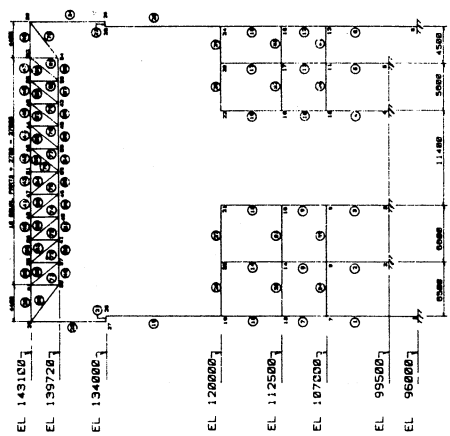
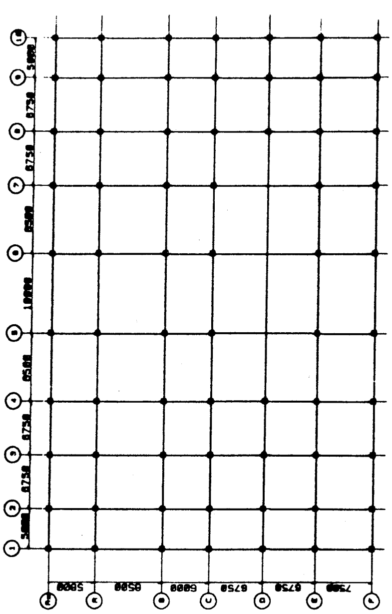


FRAME - 9

FIG-3 TURBINE BUILDING



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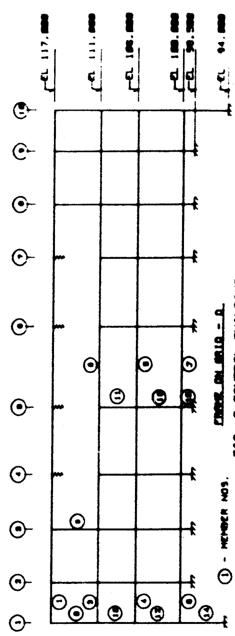

① - MEMBER NOS.

FRAME ON GRID - D

FIG. 2 CONTROL BUILDING