Paper No. 180, Bulletin, I. S. E. T. Vol. 15, No. 1 and 2, March and June, 1978, pp. 1-7

SEISMIC ANALYSIS OF A LARGE SPAN AIRCRAFT HANGAR

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SYNOPSIS

This paper presents the seismic analysis of a typical aircraft hangar structure being constructed at Bombay for Indian Airlines Corporation. The main supporting structure has a very heavy mass at the roof level and a few large size columns of height to depth ratio ranging between 2.5 to 4.0. The main objective of this investigation is to study the effect, on the magnitude of seismic forces, of (i) large variation in masses at various floor levels and (ii) inclusion of shear deformation in columns and large moment of inertia of beams in the column region. The results obtained by modal analysis on the basis of recommendations of IS; 1893-1975, are compared for the two cases (a) considering the above referred two effects and (b) neglecting the two effects.

INTRODUCTION

Hangars used for safe shelter and maintenance of aircrafts, are special type of structures. It is a usual practice to provide large column-free areas covered by suitable

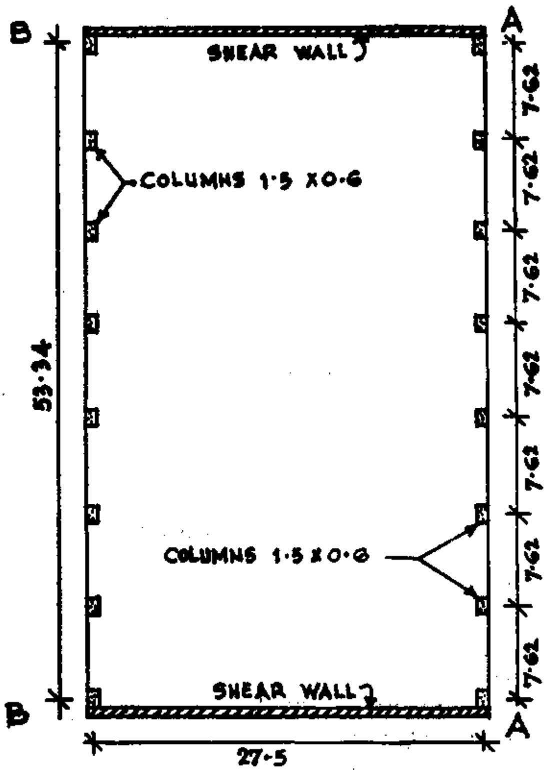

Fig. 1 Lay-out Plan

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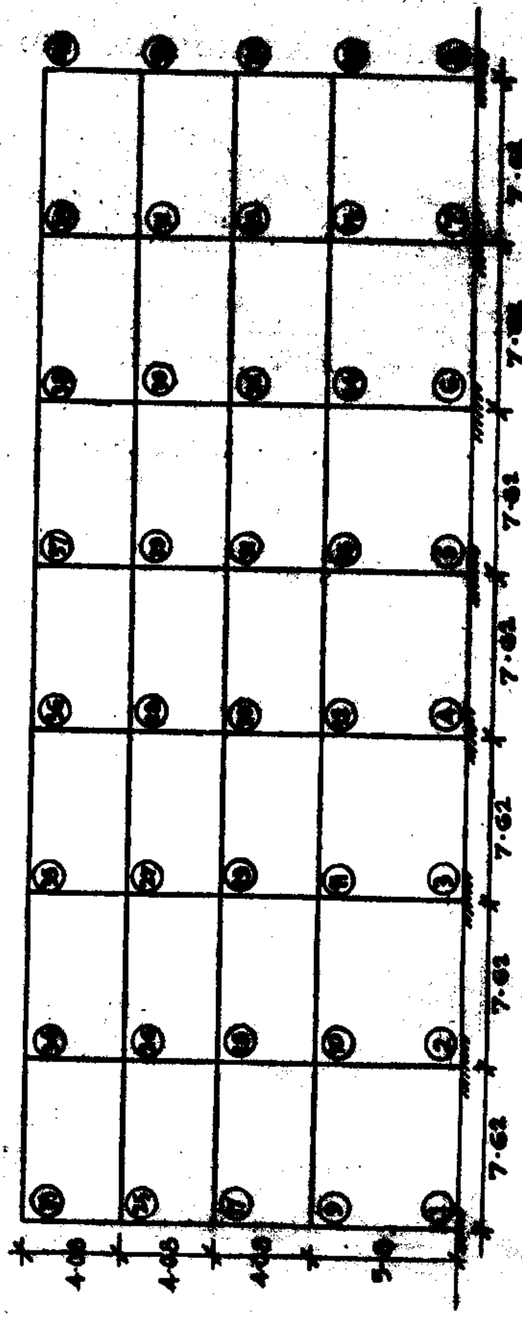

Fig. 2 Details of Plane Frame Along A.A.
Note;—All columns of 1.5×0.6 & All Beams of 0.3×1.05

roofing which generally overhangs substantially on both sides of the main supporting structure. Such an arrangement results in a structure with few large-size columns which have to support a very heavy mass at the roof level. As the dynamic behaviour of a structure is largely influenced by both the magnitude and distribution of mass and stiffness it was decided to study the seismic response of a typical hangar structure being constructed at Bombay for Indian Airlines Corporation.

The main supporting structure is rectangular in plan with overall dimensions of 53.34×27.5 metres as shown in Fig. 1. Seven-bay, four storeyed reinforced concrete frame along each of the longer sides and large plane shear walls on the shorter sides, support the main concourse floor slabs and the overhanging roof at the top. The details of the reinforced concrete frame are shown in Fig. 2. The shear walls along AB extend upto third floor level and only columns are taken up above this level upto the roof. As is seen from Fig. 2. the storey heights are 5.8 metres for the first storey and 4.08 metres for each of the upper three storeys. The size of all the columns is 150×60 cm. Thus, the height to depth ratio of columns is in the range of 2.5 to 4.0. It is, therefore necessary to include the effect of shear deformations in columns and also the effect of large moment of inertia of beams in the column region to obtain a realistic estimate of the stiffness of the structure. Axial deformations of all members are also considered. The weights of floor-wise masses are 300, 765, 740 and 2620 tonnes at 1st, 2nd, 3rd and roof level respectively.

DETAILS OF ANALYSIS

Discussion in the paper is restricted only to the details of modal analysis of a typical longitudinal frame shown in Fig. 2 subjected to horizontal-ground motion in its own plane. In the analysis, masses are considered to be lumped at every joint. In general, each joint has three degrees of freedom viz. translations in horizontal and vertical directions and rotation as shown in Fig. 3. For the purpose of obtaining the free vibra-

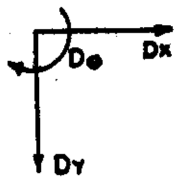

Fig. 3 Degrees of Freedom at a Typical Joint

tion characteristics, the overall stiffness matrix corresponding to three degrees of freedom per joints is reduced to that with single degree of freedom per joint in the horizontal direction by neglecting (i) the rotational inertia couples and (ii) the vertical component of inertia forces.

The equation of motion for free vibration is given by

$$[M]\{\ddot{D}_x\}+[S]\{D_x\}=0 \qquad ..(1)$$

where

$[M]$=Square diagonal matrix of size 32×32

$[S]$=reduced stiffness matrix of size 32×32

$\{D_x\}$=Lateral displacement vector of size 32×1

The reduced stiffness matrix is obtained for the following two cases:—

Case (a)—Considering the effect of shear deformations in columns and infinitely large moment of inertia of beam portions in the column region.

Case (b)—Excluding the two effects mentioned in case (a).

The expression for end moments in member including the effects of shear deformation and infinitely large moment of inertia in end portions have been developed separately by Thadani[8]. Using same concept, the stiffness coefficients have been modified to formulate the reduced stiffness matrix for case (a).

The seismic forces are computed using the acceleration spectrum curve and modal-superposition technique as recommended by IS; 1893–1975 for the following data:—

Damping	=5%
Importance factor	=1.5
Seismic zone factor for Bombay	=0.2
Soil foundation system factor	=1.0

RESULTS AND DISCUSSION

The seismic forces at each floor obtained for the two cases are presented in Table I.

TABLE I

Floor level	Weight of floor mass (tonnes)	Seismic forces (tonnes)		Seismic forces in terms of % of weight of floor mass	
		Case (a)	Case (b)	Case (a)	Case (b)
1st	300	10.73	10.31	3.58	3.43
2nd	765	27.70	25.48	3.62	3.33
3rd	740	27.05	23.84	3.66	3.23
Roof	2620	99.63	86.50	3.81	3.30

The seismic forces in case (a) are higher than those in case (b) by 4.2%, 8.74%, 13.48% and 15 18% at 1st, 2nd, 3rd floor and roof level respectively.

Table II gives the values of storey shears for the two cases.

TABLE II

Storey No.	Weight of floor mass (tonnes)	Storey shear (tonnes)	
		Case (a)	Case (b)
1	300	165.11	146.12
2	765	154.38	135.82
3	740	126.68	110.34
4	2620	99.63	86.50

From the results presented in Table II, it is seen that the storey shears in case (a) are uniformly higher than those in case (b) by about 15%.

The natural frequency of vibration and the participation factor for various modes for case (a) are presented in Table III.

TABLE III

Mode No.	Frequency (Cycles/Sec)	Participation Factor
1	7.36	0.3580
2	23.38	0.0038
3	32.15	0.2527
4	41.08	0.0007
5	48.96	0.0284
6	57.09	0.0001
7	70.85	0.0000
8	74.22	0.0815
9	78.75	0.0082

Storey No.	Frequency (Cycles/Sec)	Participation Factor
10	81.84	−0.0003
11	86.44	0.0410
12	89.51	0.0009
13	93.34	0.0001
14	107.62	0.0133
15	108.98	0.0124
16	114.53	0.0678
17	127.70	0.0552
18	133.23	−0.0012
19	133.33	−0.0015
20	153.03	0.0033
21	153.99	0.0031
22	157.43	0.0192
23	167.55	−0.0002
24	172.48	−0.0004
25	174.73	−0.0003
26	184.76	−0.0002
27	190.70	0.0069
28	195.46	0.0078
29	228.25	0.0113
30	257.76	0.0082
31	278.82	0 0065
32	290.78	0.0137

It is noticed from the plots of various modal shapes that mode numbers 1, 3, 8 and 16 for case (a) correspond predominantly to the flexural pattern. This is also borne out from the fact that the participation factors for the modes indicated above are generally the highest in order of magnitude and have maximum influence on seismic forces developed in the structure.

CONCLUSIONS

The inclusion of axial deformations in the analysis introduces the natural frequencies corresponding to axial mode of vibration as well as those corresponding to flexural mode of vibration. It is found that the participation factors for natural vibrations in axial mode are negligibly small though some of these happen to be 2nd, 4th and 5th order modes of vibration. This suggests that while determining seismic forces, it is necessary to consider higher orders of the modes providing due weightage to the participation factors.

It is seen that in spite of large variation in masses at different floor levels, the seismic forces in terms of percentage of weight of floor mass are almost the same at all floor levels. However, the increase in seismic forces by considering the effects of shear deformations in columns and variation of moment of inertia of beams in column regions varies from 4% at 1st floor to 15% at roof level. This increase of 15% in the seismic force at roof level, where mass is relatively high, forms an important design consideration.

It is concluded that in cases of structures such as the one studied, a detailed seismic analysis including the various parameters discussed, is necessary.

ACKNOWLEDGEMENT

The authors wish to thank Mr. R.J. Dubash, Consulting Engineer, Bombay for sponsoring the above project to V.J.T.I. Thanks are also due to Shri M.G. Gadgil for the assistance rendered and to the authorities of Tata Institute of Fundamental Research where the computer analysis was carried out.

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