THE INFLUENCE OF DIAPHRAGM FLEXIBILITY ON THE SEISMIC RESPONSE OF BUILDINGS

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SUMMARY

A study is described in which the influence of floor diaphragm flexibility upon the seismic response of a variety of buildings is examined. General effects of floor diaphragm flexibility are described, a newly developed technique for modelling this flexibility is presented and recommendations are made for modelling diaphragm flexibility using this technique. The seismic response of three hypothetical structures, both with and without the effects of diaphragm flexibility, is then discussed. In each case, neglecting the effects of diaphragm flexibility leads to erroneous response. The paper closes with conclusions from the study.

INTRODUCTION

Building analyses used in support of highrise design have traditionally considered the floor diaphragms to be rigid in their own plane, implying a distribution of shear to lateral force resisting elements (columns, diagonals, walls) in proportion to the lateral stiffness of each element.

For a variety of architectural forms, the assumption of rigid floor diaphragms may be seriously in question. The seismic response of buildings that have a large aspect ratio in plan and those with several wings or separate towers, amongst other forms, is potentially strongly influenced by the effects of diaphragm flexibility.

GENERAL EFFECTS OF DIAPHRAGM FLEXIBILITY

Diaphragm flexibility has the potential of influencing the dynamic response of buildings in two predominant fashions, both of which are highlighted in the section on Example Analyses:

1. The distribution of shear among the lateral resisting elements is not necessarily in proportion to the lateral stiffness of each resisting element.

2. Diaphragm flexibility may affect the dynamic characteristics of the building. For example, in a building with several wings, local modes may occur in separate wings. These modes, which would be ignored in an analysis assuming a rigid floor diaphragm, may dominate the response of the floor system.

MODELLING OF DIAPHRAGM FLEXIBILITY

Recently, a new computer program, **COMBAT**, for the seismic analysis of buildings has been developed (Ref. 1). This program provides the designer with increased modelling capabilities over the program's predecessors, **TABS** and **ETABS**. Among the new features is a simple technique for modelling floor diaphragm

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flexibility which is easy to use, yet provides accurate results.

Previous building analysis programs condensed the total stiffness to a lateral stiffness based on three "lateral" degrees of freedom per floor (two horizontal translations and a rotation about the vertical axis) via the assumption that the floor was rigid in its own plane. This condensed system was then used to solve the eigenproblem and for subsequent dynamic analyses. Forces were calculated by back substituting to recover the full set of displacements.

The idea of condensing to a reduced set of degrees of freedom for lateral solution is retained in **COMBAT**. However, the reduced set can be expanded at each floor as required. Just as the in-plane displacements of a rigid diaphragm can be represented by three degrees of freedom at one "master node", so too can the displacements of a flexible diaphragm be represented by three degrees of freedom at each of several master nodes. These floor master nodes are linked by two-dimensional floor beams representing the in-plane stiffness of the diaphragm.

COMBAT allows two types of slaving. In the first type, a particular column is slaved directly (rigidly) to a floor node. The second type enables a column to be slaved to a floor beam member rather than a floor node, providing a rigid floor assumption. The floor displacements of a column so slaved are determined by the flexing of the floor beam to which it is slaved. This determination is made via beam functions from classical beam theory. Transformations are set up to enable element local stiffnesses to be correctly transformed to global coordinates reflecting the slaving specified by the user.

This formulation for floor diaphragm flexibility allows the user complete freedom in the modelling of a building. A single floor node with rigid slaving models a rigid diaphragm. Other configurations allow the modelling of floors which are flexible due to plan geometries such as large aspect ratios or perforated diaphragms.

Modelling Recommendations

Several recommendations regarding the modelling of floor flexibility in a program such as **COMBAT** can be made. Firstly, if more than one master node is considered, careful attention must be given to the distribution of masses at the various master nodes to ensure a reasonable mass moment of inertia about the center of mass of the floor. If the mass moment of inertia about the center of mass of the floor, calculated from the master node masses and their locations, is not consistent with the total mass moment of inertia of the floor, the torsional response of the structure can be seriously affected. Secondly, the value assigned to the moment of inertia for the floor diaphragm members is an important parameter in an analysis of this kind. If the diaphragm is a concrete slab, the stiffness of the diaphragm will change dramatically if the section cracks. An analysis should be performed assuming an uncracked section, and the moments in the diaphragm compared to the cracking moments. If these are exceeded, the analysis should be rerun with cracked section properties. Finally, an economical number of floor nodes should be used. If too many floor nodes are specified, the lateral stiffness of the structure will become large, somewhat defeating the purpose of condensing to lateral degrees of freedom. The flexible floor should be modelled with the least number of floor nodes that can capture the essence of its dynamic behavior. As demonstrated in the first example, this number would typically be at least three master nodes.

EXAMPLE ANALYSES

Three example analyses are presented. The first is a six story building with a rectangular plan shape having a 5-to-1 aspect ratio. A parameter study demonstrating the various possibilities for floor flexibility modelling in terms of the number of master nodes per floor is conducted. One configuration is then used to demonstrate the effect of varying floor flexibility on the seismic response of the structure. The second example is a four story structure with three "wings" in plan. Analyses are performed assuming both rigid and flexible floors, and selected results from the analyses are compared. The third structure has two separate towers above the third floor level. One extends a further three stories, while the second extends a further eight. Again, analyses are performed assuming both rigid and flexible floors, and key results are compared.

Large Plan Aspect Ratio Example

A six story steel framed structure with plan dimensions 20' by 100' (columns on a 20' grid) and story heights of 12' was studied. All columns are WF 14x109 sections, and all beams are WF 14x61 sections. The floors are 4" concrete slabs, and two-thirds of the width was assumed effective in calculation of the floor diaphragm properties to approximately account for slab openings. The structure was analyzed with 4 different master node layouts: 1 central master node (rigid floor, model I), and then 2, 3 and 6 master nodes (models II, III and IV respectively). The nodes and their associated translational masses were positioned to give the correct rotational inertia about the center of the floor, as shown in Figure 1. An eigenanalysis was then performed for each configuration, and the results are compared in Table 1.

From inspection of Table 1, it is apparent that all models capture the "rigid floor" modes well. The "rigid floor" modes are labelled X_i, Y_i (translational modes) and R_i (rotational modes). Additional modes due to floor flexibility are identified as B_i. It is apparent that model II does not capture any floor modes in the first 18 modes. In fact the floor modes captured in this model are modes 19 through 24 with identical frequencies, one axial mode for each of the six floors. Flexural modes in the floor diaphragms are not present in model II due to zero rotational inertia at each of the two floor nodes. Model III (three floor nodes) gives the first three floor flexuaral periods as 0.1578, 0.1325 and 0.0932 seconds while model IV gives the same modes periods of 0.1142, 0.0968 and 0.0711 seconds. Model III exhibits consistently longer "floor" periods than does model IV due to the large mass at the central floor node in model III, necessitated by the constraints on the total rotational inertia of the floor. Although model IV is considered to be the most accurate representation of the actual structure due to the finer discretization of the floor diaphragm, a comparison of the bottom story column shears from a response spectrum analysis of each configuration indicates that model III is an adequate representation (column shears from III are all within 1% of the corresponding shears from IV) with significantly less computational effort. Model III was thus used to demonstrate the effect of varying floor diaphragm flexibility on the seismic response of this structure.

A set of response spectrum analyses were performed with this model. The input spectrum was that of the unscaled N-S component of the El Centro 1940 motion (5% damping). The difference in the analyses was the moment of inertia of the floor diaphragm member. A realistic value was used as the reference point, and a range of moments of inertia about this value was considered. The case with the highest moment of inertia effectively corresponds to a rigid floor assumption. The distribution of shears in the bottom story columns was examined.

Figure 2 summarizes the distribution of shears in the bottom story columns. Due to symmetry, only the three columns as identified in Figure 1 need be examined. In Figure 2 the maximum shear forces experienced in these columns are plotted as a function of the moment of inertia of the floor diaphragm member. As would be expected, when the floor member is very stiff, the shears in each column approach equal values. However, as the floor member becomes progressively more flexible, the innermost columns begin to attract more shear. The limiting case for no diaphragm is for each column to attract a shear in proportion to the tributary area associated with that column. For this particular example, at a moment of inertia corresponding to an uncracked floor slab, the maximum difference in column shear from that calculated from a rigid floor analysis is less than 2%. For a moment of inertia corresponding to a fully cracked section for the floor diaphragm, however, this difference increases to 12%.

For the analysis with the uncracked moment of inertia for the floor diaphragm, the maximum shear stress in the diaphragm is approximately 30 psi. Thus in this example, the floor diaphragm would remain uncracked at this stress level, and the influence of diaphragm flexibility is small. However, if the potential does exist for cracking of the floor diaphragm in a structure such as this, failure to include floor flexibility can lead to design forces on the non-conservative side, potentially by margins greater than those indicated in this particular example.

Three Winged Structure Example

A hypothetical structure of six stories with three wings in plan (as shown in Figure 3) is used as a second example. Each wing has an aspect ratio of three to one, and the wings are mutually oriented at 120 degrees, to form a rigid triangular zone at their intersection. Individual members have been assigned properties to give physically reasonable dynamic properties (mode shapes and frequencies). At least one structure of this general layout has recently been completed in a seismically active region of California. The floor system for this structure is modelled in two ways. It is first assumed to be rigid with a single central master node. Secondly, three additional master nodes are added, one at the extremity of each wing, giving a total of four master nodes. This modelling allows flexing of the individual wings which cannot be captured with the rigid floor assumption. Eigenanalyses are performed for each configuration, and the results for the first 10 modes are presented in Table 2.

It is apparent from examination of this table that the rigid floor assumption misses significant modes, which come in this case as early as mode 7. Note that both the translational (X_i and Y_i) and the floor modes (B_i) come in orthogonal pairs. Motion of a single wing can always be expressed as a linear combination of the two appropriate modes. The flexible floor model captures the "rigid floor" modes accurately, and the wing modes occur as additional, separated modes.

No response spectrum analyses are presented for this example. However, it is apparent that a rigid floor assumption provides the designer with no direct information on which to base the design of the floor diaphragms. Design forces for these structural elements could be deduced from the shears in the columns which each diaphragm connects, but these shears would be questionable due to the rigid floor assumption. Even if the shears were reasonable, processing is tedious and subject to human error whereas an analysis explicitly including the floor flexibility gives direct design information (moments and shears) for the floor diaphragms.

Separate Towered Structure Example

The third and final example is again a hypothetical one, but representative of an increasing number of highrises. It is a two towered structure, one tower of eleven stories, the second of six stories, connected by a central section three stories in height. Each tower is square in plan and of the same dimensions as the connecting section. Although the individual members were not fully designed, their relative sizes give physically realistic dynamic properties (mode shapes and frequencies). The structure was analyzed first with a rigid floor assumption, and then with "flexible" floors, allowing the two tower sections to respond independently. Eigenanalyses were performed for each floor configuration, and the results for the first ten modes are compared in Table 3.

It is apparent that the two models give a significantly different set of periods. The essential difference between the two sets of mode shapes corresponding to these periods is that the "flexible" floor model gives distinct modes for the two towers. A selected mode shape from the flexible floor model is plotted in Figure 4. It is clear that this mode (and others like it) is not captured with the rigid floor assumption.

To emphasize the importance of these modes, response spectrum analyses were performed on each of the two models, and the total shear at the base of the taller tower (i.e. at the third story) was examined. The input spectrum was the N-S El Centro 1940 5% spectrum. For the rigid floor assumption, the total shear at this level was 145.6 kips, while the "flexible" floor model predicted 178.7 kips. The rigid floor assumption thus underestimated the total tower base shear by 19%. Corresponding maximum displacements at the top of the taller tower were 6.4" for the rigid floor assumption and 6.6" for the flexible floor. While these maximum displacements are similar, the distribution with height (and hence the interstory drifts) are quite different for the two models.

While analyses including separate tower flexibilities have previously been possible using general purpose programs, these programs do not take advantage of the regular geometries of building systems, and are cumbersome to use. **COMBAT** allows accurate modelling of this flexibility, yet is easy to use.

CONCLUSIONS

The effects of floor diaphragm flexibility have been shown to be of potential significance in a number of building forms which are in increasingly common use today. Analyses neglecting this floor flexibility (as in the traditional building analysis assumption of rigid floor diaphragms) are potentially in error on the non-conservative side. A recently released program, **COMBAT**, allows accurate yet simple modelling of floor diaphragm flexibility, while maintaining the advantages of an analysis program designed for building-type structures.

ACKNOWLEDGEMENTS

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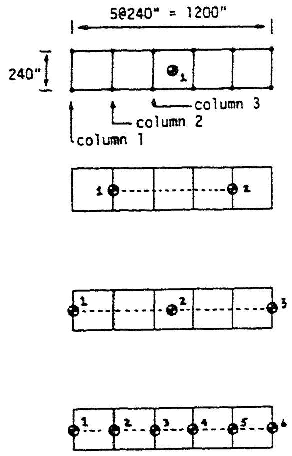
(1) "COMBAT - Comprehensive Building Analysis Tool", Computech Engineering Services, Inc., Berkeley, California. 1983.



	TRANSLATIONAL MASS	ROTATIONAL MASS
I	$M_1 = 0.4\ k.s^2/in$	$M_{1\theta} = 51{,}840\ k.s.^2\ in$
II	$M_1 = M_2 = 0.2\ ks^2/M$	$M_{1\theta} = M_{2\theta} = 0$
III	$M_1 = M_3 = 0.072\ ks^2/in$ $M_2 = 0.256\ ks^2/in$	all $M_\theta = 0$
IV	$M_1 = M_6 = 0.04\ ks^2/in$ $M_2 = M_3 = M_4 = M_5 = 0.08\ ks^2/in$	all $M_\theta = 0$

FIGURE 1 - FLOOR DIAPHRAGM MODELLING

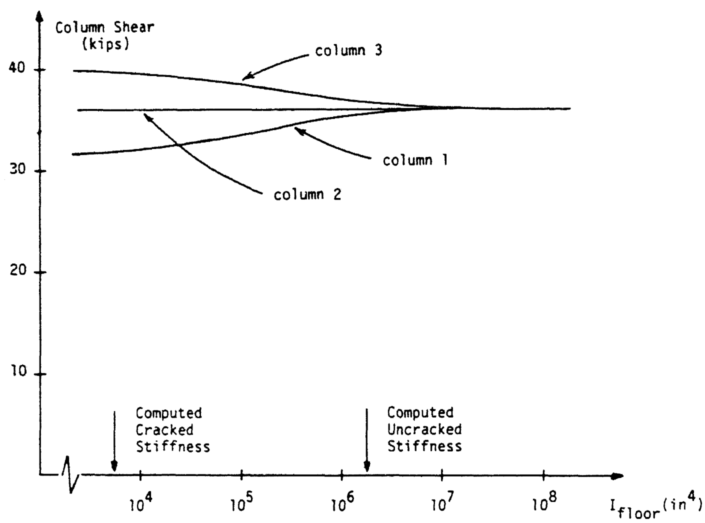


FIGURE 2 - COLUMN SHEAR DISTRIBUTION

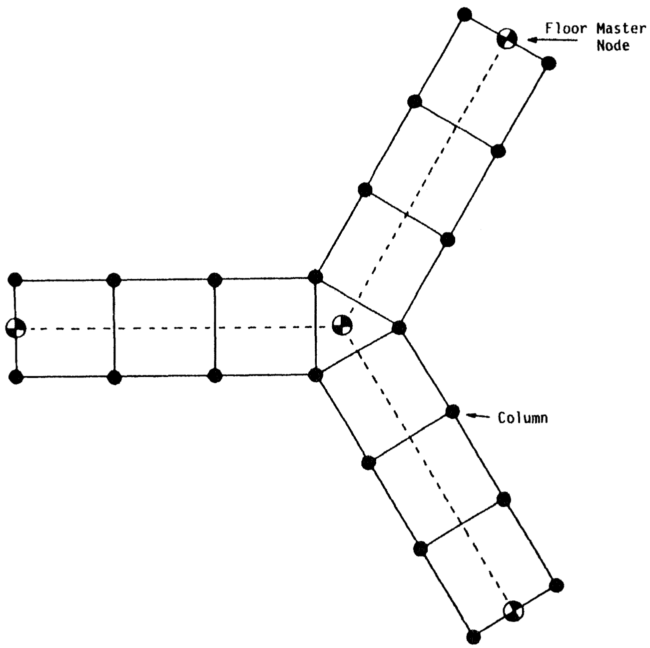


FIGURE 3 - PLAN OF THREE WINGED STRUCTURE

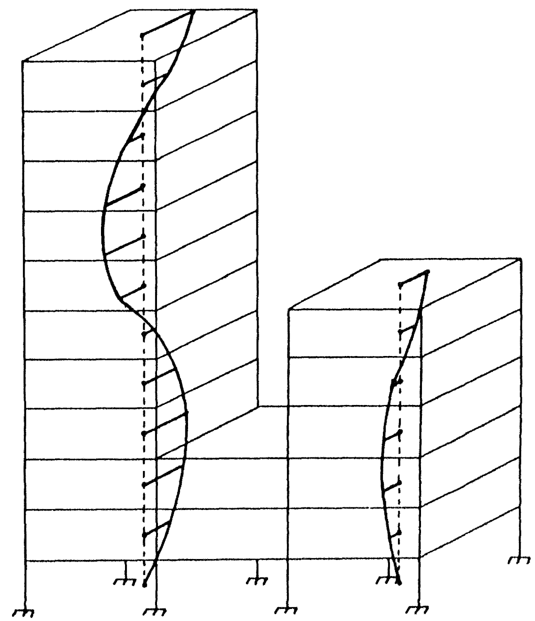


FIGURE 4 - SEPARATE TOWERED STRUCTURE - MODE 7

MODE	MODEL							
	I		II		III		IV	
1	0.764	Y_1	0.764	Y_1	0.765	Y_1	0.765	Y_1
2	0.763	X_1	0.763	X_1	0.763	X_1	0.763	X_1
3	0.641	R_1	0.642	R_1	0.641	R_1	0.641	R_1
4	0.247	X_2	0.247	X_2	0.247	X_2	0.247	X_2
5	0.228	Y_2	0.229	Y_2	0.234	Y_2	0.231	Y_2
6	0.193	R_2	0.198	R_2	0.194	R_2	0.194	R_2
7	0.141	X_3	0.141	X_3	0.157	B_1 *	0.142	X_3
8	0.115	Y_3	0.116	Y_3	0.141	X_3	0.119	Y_3
9	0.098	R_3	0.107	R_3	0.132	B_2 *	0.114	B_1 *
10	0.096	X_4	0.096	X_4	0.127	Y_3	0.100	R_3
11	0.074	X_5	0.074	X_5	0.100	R_3	0.097	B_2 *
12	0.070	Y_4	0.071	R_4	0.097	X_4	0.097	X_4
13	0.062	X_6	0.070	Y_4	0.093	B_3 *	0.074	X_5
14	0.060	R_4	0.062	X_6	0.084	B_4 *	0.073	Y_4
15	0.048	Y_5	0.053	R_5	0.074	X_5	0.071	B_3 *
16	0.041	R_5	0.049	Y_5	0.064	X_6	0.063	X_6
17	0.038	Y_6	0.044	R_6	0.062	R_4	0.062	R_4
18	0.033	R_6	0.038	Y_6	0.061	B_5 *	0.051	Y_5

Table 1 : Comparison of Periods (Seconds) - Example 1

MODE	MODEL			
	RIGID		FLEXIBLE	
1	0.433	R_1	0.434	R_1
2	0.356	X_1	0.359	X_1
3	0.356	Y_1	0.359	Y_1
4	0.126	R_2	0.127	R_2
5	0.111	X_2	0.113	X_2
6	0.111	Y_2	0.113	Y_2
7	0.065	R_3	0.093	B_1 *
8	0.060	X_3	0.093	B_1 *
9	0.060	Y_3	0.076	B_2 *
10	0.044	R_4	0.076	B_2 *

Table 2 : Comparison of Periods (Seconds) - Example 2

MODE	MODEL	
	RIGID	FLEXIBLE
1	0.830	0.876
2	0.667	0.771
3	0.355	0.378
4	0.278	0.316
5	0.246	0.250
6	0.141	0.191
7	0.139	0.142
8	0.108	0.138
9	0.094	0.119
10	0.085	0.092

Table 3 : Comparison of Periods (Seconds) - Example 3