DYNAMIC ANALYSIS OF INFILLED R.C. PLANE FRAMES WITH OPENINGS IN THE IN FILLS

JAMAL AHMAD ALOMARI

Civil Engineering Department, Alahliya Amman University, Amman - Jordan
E-mail: j.alomari@ammanu.edu.jo

Abstract - Dynamic analysis of an 8-story reinforced concrete building frame including raft foundation and infill panels is carried out. The wall panels are composed of building stones backed by plain concrete . Door and window openings are included within the wall panels. The dynamic loading used is the well-known El Centro N-S earthquake excitation of 1949 . The raft foundation is divided into a number of finite elements and the soil below is assumed homogeneous and extending below the raft foundation. Its stiffness matrix is obtained by inverting its flexibility matrix. Wall panels with openings are represented by plane stress plate finite elements. An opening in the panel is realized as an element of very small modulus of elasticity for its material or a very small thickness or both of them. Different opening configurations and sizes were utilized. The stiffness matrix of the structure components are assembled into the overall structure stiffness matrix. A computer program in FORTRAN was written to handle the described calculations.
The structure dynamic equation is written as:
$M\, d^2y / dt^2 + C\, dy / dt + K\, y = F_{eff}(t)$
and
$F_{eff}(t) = -M\, d^2y_g / dt^2$
Where :
M is the mass matrix of the structure,
C is the damping matrix of the structure
K is the stiffness matrix of the structure
y is displacement of the structure joints, and its derivatives re[present velocity and acceleration of the structure joints.
F(t) represent the vector of forces applied to the structure.
d^2y_g / dt^2 is the ground acceleration
The described frame is analyzed under gravity loads and El-Centro excitation. The first 10 periods of vibration are obtained for different configurations of wall panels with different sizes of openings. The first five modal shapes of the building frames are obtained. The horizontal displacements at the story levels were recorded. The effects of wall panels and openings on the structure behavior under the described loading is presented.

Keywords - Dynamic; infill; opening; mode shape; period; seismic

I. INTRODUCTION

The number of studies, both theoretical and experimental, conducted on the reinforced concrete building frames with infill panels including openings is large . However, the effects on the panels and openings on the structure behavior during seismic excitation is not formulated in the codes. The effects can be summarized as follows:

1. The stiffness of the building is increased, the fundamental period of vibration is decreased, and therefore the base shear due to seismic action is increased.
2. The distribution of the lateral stiffness of the structure in plan and elevation is modified.
3. The ability of the structure to dissipate energy is substantially increased.
4. Parts of the seismic action is carried by the infills, thus relieving the structural system.
5. The properties and behavior of infill panels during earthquakes is highly variable, changes rapidly during the course of the excitation, and therefore cannot be relied on.

- On the other hand, the ACI publication (aci-sp-127), clarifies this point of view of the effect of infill panels. It states that masonry infills can attract large story shears because natural frequencies are substantially decreased with the larger stiffness that results from their presence. In this study, the effect of infill panels with openings of different configurations and sizes on the structure behavior under seismic loading will be presented.

II. LITERATURE REVIEW

The amount of research done regarding reinforced concrete frames with masonry infills having openings is large. The previous research in general confirmed that the five effects mentioned in the introduction above are still valid for infill panels with openings. However, no definite formulae were suggested to relate the window and door openings areas to those effects. The following is a summary of previous articles on the subject.

Stephen Schneider et al in their work "Lateral Strength of Steel Frames with Masonry Infills Having Large Openings" in Aug. 1998 tested steel frames with brick masonry infill with a large opening. The authors concluded that the ductility of the masonry

infills depended on the pier width and number of wyths.

SurasakNiyompanitpattanaPennungWarnitchai, in Nov. 2017 tested five long-span reinforced concrete frame specimens with various configurations of masonry infill walls under lateral quasi-static cyclic loading. The specimens comprised of one bare frame and four frames with either a full infill, partial-height infill, central-opening infill or side-opening infill.
DavorinPenava, VasilisSarhosis, IvicaKožar, IvicaGuljaš developed a computational model based on the nonlinear finite element (FE) method of analysis to study the influence of differences in size and location of window and door openings on the shear resistance of infilled frames. The paper concluded that door opening causes higher shear in frame part than designed for in bare frame, and window opening causes less shear in frame part than designed for in bare frame.
Hong Guan, Carlia Cooper, Dong-Jun Lee in their work "Ultimate strength analysis of normal and high strength concrete wall panels with varying opening configurations", in May 2010, conducted a comparative study to verify the effectiveness of the method of nonlinear Layered Finite Element Method (LFEM) in predicting the failure characteristics of seven two-way normal strength concrete walls, without, and with window and door openings. Results demonstrate that increase in opening size decreases the axial strength ratio to different degrees for one-way and two-way walls. Increasing only the opening length also significantly decreases the axial strength ratio. Increasing only the opening height has little impact on the ultimate load capacity.
Made Sukrawa in his work “Earthquake Response of RC Infilled Frame with Wall Openings in Low-rise Hotel Buildings" in 2015 studied the earthquake response of RC infilled frame structures with variable wall opening. A 3-D computer models were made for three, four, and five story typical hotel buildings consisting of six frames of three bays.
The author concluded that analysis and design of the 3-D models show that the earthquake responses of RC frames infilled with walls of opening ratios 20% to 60% are significantly stiffer and stronger than that without infill wall. However, the contribution of infill walls with 80% opening in reducing story drift and frame reinforcement was much smaller. Accordingly, the infill walls with opening ratios of less than 80% should be considered in the structural modeling to obtain a more accurate analysis and efficient design.
Mosoarca Marius in his work " Seismic behavior of reinforced concrete shear walls with regular and staggered openings after the strong earthquakes between 2009 and 2011" in December 2013 theoretical and experimental studies have been conducted on three types of walls with vertical staggered openings, one with regular openings and a solid wall. The models were loaded until failure and provided information on: the forces, the horizontal displacements, and the maximum stresses and strains recorded in the concrete and in the reinforcement.
Naveed .A.G, Chandradhara .G.P in their work " Seismic Performance of Infilled Frames with and without Opening" have indicated that the frames comprising of masonry panels behave significantly stiffer as compared to bare frames. The opening is expressed in terms of various percentages. In this paper an intermediate frame of a simple building (G+3) is considered for modelling of infill frame with and without opening using the equivalent diagonal strut using the software ETABS. It is concluded that the effect of opening in the masonry decreases the lateral stiffness in the frame and reduction factor, λ can be used as a multiplication factor to calculate the reduced equivalent width of diagonal strut. Later deflection and drift do not increase considerably beyond 40-50% of opening.

A. Description of frames and loading

This paper is a continuation of previous work. A two-dimensional 8-story, 3 bay R.C. frame, shown in figure 1, built on raft foundation 0.7 meters thick is analyzed by subjecting it to the N-S El Centro excitation of 1940 using a computer program written in Fortran.

Directly on the raft foundation is constructed a 0.3 m thick reinforced concrete wall to tie the columns' stubs, and to support the exterior masonry wall, which is 0.4 m thick. The exterior wall is built of building stone, backed by plain concrete. The bond between building stones and backing concrete is usually weak, and no reinforcement is usually used within the backing concrete.

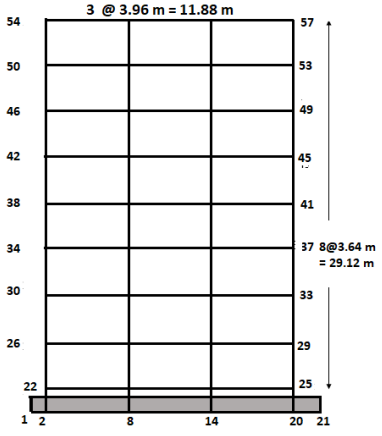
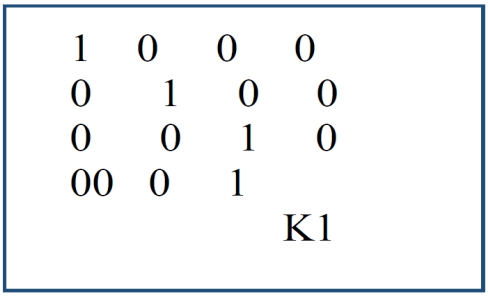


Figure 1 - Reinforced concrete bare frame on raft foundation

For the purpose of the dynamic analysis, the raft foundation joints will be considered fixed against rotation and translation. All other joints are free. Fixity of raft foundation joints is realized in the structure stiffness matrix by assigning a value of unity to the foundation member diagonal stiffness coefficients and zero to other coefficients.

Suppose matrix [K] is the stiffness matrix of the super structure, i.e., structure without the raft foundation joints. The total structure stiffness matrix, including the raft foundation will appear as follows



It was shown that, this form of the structure stiffness matrix, which includes the raft foundation members as just described, will produce the same dynamic analysis results of eigenvalues and eigenvectors, as those obtained when the basic stiffness matrix [K] of the superstructure alone is used.

Masses:

Masses of members of the plane frame under consideration are needed to form the structure consistent mass matrix. Floor masses are assumed to be distributed along floor beams. Columns are assumed to have little mass so as to avoid having an overflow in the analysis. Masses of infill panels will be added to the masses of beams upon which they rest.

It is shown in the analysis that the effect of considering the raft foundation members in the structure stiffness and mass matrices is to increase the structure fundamental period of vibration and to produce repeated eigenvalues, while the original eigenvalues of the superstructure still appear in the analysis results unchanged as if the raft elements are not included.

Damping

Damping effects, in the form of viscous damping, considered as damping ratios for all or some vibrational modes will be considered. Damping ratio in this context, ξ is defined as the ratio of real damping of structure C , to its critical damping C_{cr}.

$\xi = C / C_{cr}$

And critical damping of a one degree of freedom system is given by

$C_{cr} = 2\sqrt{kxm}$

where k and m are the stiffness and mass of the one degree system respectively. The damping matrix C would not be needed in modal analysis of earthquake response because estimated modal damping ratios are sufficient (Reference 1).

Output of dynamic analysis

The results of the dynamic analysis will include the following:

1-Eigenvalues (and periods of vibration) of the structure. The first few eigenvalues (and periods of vibration) will be tabulated.

2-Eigenvectors of the structure. There will be as many eigenvectors as there are degrees of freedom in the structure. Each eigenvector will include the coordinates of all degrees of freedom for the modal shape under consideration.

Eigenvectors for horizontal displacement degrees of freedom of some selected joints of the analyzed frames will be tabulated. Then, these selected valuesof eigenvectors will be shown in drawing to display the first five modes of vibration regarding the horizontal degrees of freedom. The response, in the form of displacements, of the structure to the applied El Centro ground excitation. The excitation up to the first second will applied. The response at some selected point of time within the excitation period will be recorded for some selected joints.

3-The member end forces in the form of shear, axial force, and moment. These forces can then be checked against member capacities to determine whether members can sustain the described excitation. The forces and moments in some selected members will be tabulated

Cases Considered:

First :

The first group of cases to be considered in the dynamic analysis studies the effect of the presence of infill panels in general, and the effect of openings in infill panels on the dynamic behavior of the studied frames. This group of cases (cases A) will not include the effect of damping. Damping ratios for all modes of vibration will be set equal to zero.

Case A.1 is the bare frame without infill panels.

Case A.2 will have 8 wall panels with openings whose area is equal to 0.36 of the wall panel area as shown in figure 2.Figure 3 shows details of finite elements comprising the wall panel and opening.

Case A.3: as case 2 but with window opening area to panel area = 0.24

Case A.4: Window opening area to panel area = 0.12

Case A.5: No openings in panels.

For these five cases, the ground acceleration of El Centro Earthquake is applied to the frame. None of these cases was able to stand the excitation as indicated by the member forces resulting from the analysis, i.e. the member forces were larger than the members' capacities. Therefore, for the purpose of this study, a ground acceleration of one tenth of that of El Centro is applied. A sample of the values of the ground acceleration applied is shown in table1.

Table1. Ground acceleration (a/ g x 10000.0) values applied to the raft foundation of the frames of cases A.1, A.2, A.3, A.4,and A.5. Values are given for a period of one second in increments of 0.02 second. These values are 1/10 of EI - Centro earthquake

recorded acceleration of the N - S component

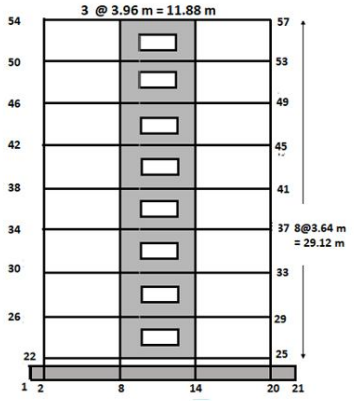


Figure 2 – Reinforced Concrete Frame With Infill Panels and window Openings. Areas of Window Opening To Panel Area = 0.36, 0.24, 0.12 For cases A.2, A.3, A.4 respectively . A5 case Panels have no openings.

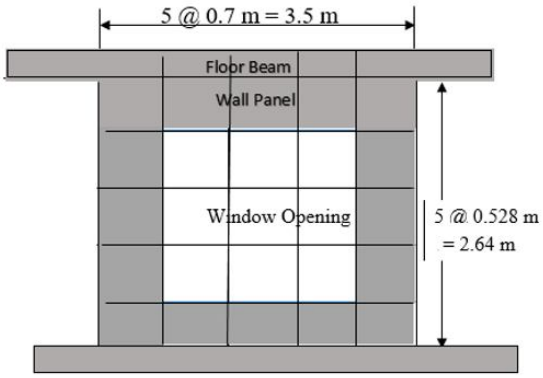


Figure 3 Window opening in the wall panel Area of opening /area of oanel = 0.36

Time	(a/g) x 10000.0
0.00	0.00
0.04	-10.80
0.08	-8.80
0.12	-12.30
0.16	-12.80
0.20	-8.50
0.24	-13.10
0.28	-19.40
0.32	-14.40
0.36	- 8.70
0.40	-6.60
0.44	-19.00
0.48	-6.60
0.52	14.10
0.56	-12.80
0.60	- 20.30
0.64	-32.50
0.68	-17.20
0.72	-16.30
0.76	-6.70
0.80	15.00
0.84	25.20
0.88	46.30
0.92	41.90

Results of the dynamic analysis performed on the 5 frames are shown in the following tables and figures.

Discussion of Results

Periods of vibration:

Table2 shows that, the fundamental period of vibration T of the frame of case A.1 , (bare frame without infill panels) is 0.889 seconds. Case A.1 is the least stiff case among the 5 cases. The fundamental period of vibration of case A.5 , (with solid infill panels), has the least fundamental period, 0.450 seconds. Case A.5 is the stiffest case.

TABLE2 . The first 10 periods of vibration in seconds for the 5 cases indicated.

Case A.l No wall panels	Case A.2 Area opening/ Area Panel = 0.36	Case A.3 Area opening/ Area Panel = 0.24	CaseA.4 Area opening/ Area Panel = 0.12	Case A.5 No Openings in panels
0.889	0.585	0.523	0.479	0.450
0.291	0.186	0.159	0.141	0.126
0.167	0.104	0.095	0.095	0.095
0.120	0.095	0.086	0.074	0.063
0.103	0.076	0.063	0.058	0.055
0.095	0.063	0.057	0.049	0.047
0.080	0.059	0.048	0.047	0.041
0.073	0.051	0.047	0.040	0.033
0.070	0.047	0.041	0.034	0.032
0.065	0.046	0.037	0.033	0.027

Therefore, the presence of 8 solid infill panels in the middle bay (case A.5) caused the fundamental period of vibration of frame A.1 to decrease from 0.889 sec. to almost ½ of this value (0.450 sec.) . Hence, the general effect of infill panels in a structural frame on its fundamental period of vibration is to decrease its value by a pronounced margin. A frame with two or more bays infilled with solid panels will be much stiffer and have a shorter period of vibration.

On the other hand, openings in infill panels modify this effect according to the size of the openings. The larger size of openings, and hence, the larger ratio of opening to panel area, will cause the effect of infills to be reduced. Larger sizes of openings i.e. larger ratios of area of opening to area of panel than 0.36, were not considered in the analysis, because they are not practical.

Moreover, Table 2 shows also that, the higher periods of vibration decrease with increasing area of opening in the infill panels. Therefore, we can conclude that, in general, openings in infills cause the frame's fundamental period of vibration, as well as the higher periods, to increase. Evaluation of this effect can be accurately obtained using the proposed method of analysis.

Lateral Displacements

Table 3 lists the lateral displacements of the 5 cases considered at each floor, taken at the left most joint in each floor. These values of lateral displacements are taken at time T = 0.6 seconds after beginning of application of ground excitation to the raft foundation. No damping is considered in these cases. Strictly speaking, it is inappropriate to consider displacements in dynamic analysis at a specific point of time to compare results of various cases, simply because periods of vibration of these cases are different, and therefore do not reach maximum values of displacements at the same time. Therefore, the information displayed in TABLE 3 for the horizontal displacements of the tabulated joints do not show any obvious relationship in the magnitude or direction of displacements. In fact, the maximum response of a structure to earthquake excitation is the most important parameter to the designer. In an earthquake analysis of a structure, which has a tremendous amount of output data, this maximum response is what should be sought.

Table 3. Lateral displacements in mm (at time T = 0.60 second after beginning of application of ground excitation) of the left hand joints of the frame for the 5 cases mentioned. Damping ratio is zero for all modes of vibration. Number of modes used to obtain the above response in the modal superposition method is 108.

Joint No.	Case A. 1 No wall panels	Case A.2 A opening/ A panel = 0.36	Case A.3 A opening/ A panel = 0.24	Case A.4 A opening/ A panel = 0.12	Case A.5 No Opening in panels
22	4.16	1.13	- 6.21	-0.31	-6.43
26	17.26	21.95	-92.07	-18.70	-59.21

30	30.03	53.03	-188.4	- 34.16	-133.6
34	42.05	86.41	- 289.1	-48.03	-213.2
48	49.74	119.3	-376.5	-60.78	-304.1
42	57.87	153.2	-458.7	-71.42	- 398.2
46	63.50	186.2	-535.4	-79.30	-488.6
50	66.86	211.3	-603.3	- 84.16	-573.0
54	69.46	228.2	- 660.8	- 85.56	-649.6

Shear Force in Columns

Tables 4 and 5 list shear forces in columns for cases A.1 to A.5 as obtained from the dynamic analysis. These shear force values are taken at time T = 0.6 seconds after beginning of application of ground excitation to the raft foundation of the structures. These values are calculated using all modes in the modal superposition method. Since the periods of vibration are different for the cases indicated, shear forces, like displacements, cannot be correctly compared to obtain concluding remarks regarding the effect of panels and openings on the response. In fact, the maximum values of structure response is the values that should be sought.

Table 4 - Shear force in KN in the columns indicated for frames of cases A.1 to A.5 These forces are obtained at time T = 0.6 second after application of ground excitation to the foundation of the frames indicated. Damping ratio is zero for all modes of vibration.

Column	Case A. 1 No wall panels	Case A.2 A opening/ A panel = 0.36	Case A.3 A opening/ A panel = 0.24	Case A.4 A opening/ A panel = 0.12	Case A.5 No Opening in panels
14- 24	- 287.1	-7.2	222.4	170.2	265.7
24-28	- 319.1	-424.1	1820.7	408.0	860.6
28-32	- 341.1	-575.3	1703.0	356.2	841.4
32-36	- 347.3	-607.8	1664.3	286.3	617.9
36-40	- 233.9	-565.3	1058.6	286.3	688.2
40-44	- 277.9	-457.2	1012.5	247.5	694.4
44-48	-205.8	-443.8	815.1	198.1	554.7
48-52	-141.0	-309.0	467.2	148.7	427.8
52-56	- 122.3	-93.5	229.9	77.2	148.1

Table5 Shear force in KN in the columns indicated for frames of cases A. 1 to A. 5. These forces are obtained at time T = 0.6 second after application of ground excitation to the foundation of the frames indicated. Damping ratio is zero for all modes of vibration

Column	Case A. 1 No wall panels	Case A.2 A opening/ A panel = 0.36	Case A.3 A opening/ A panel = 0.24	Case A.4 A opening/ A panel = 0.12	Case A.5 No Opening in panels
2-22	- 210.7	60.4	76.9	199.8	342.0

22-26	-193.7	- 364.9	1877.7	431.1	1273.8
26-30	-129.1	-467.4	1966.7	528.3	1798.8
30- 34	-72.6	- 546.1	2093.3	595.3	1895.9
34-38	59.0	-481.6	1678.9	735.2	2052.0
38-42	49.0	- 376.4	1531.5	677.8	2002.0
42-46	126.1	-340.6	1343.4	462.7	1766.8
46-50	153.5	- 138.8	1038.8	344.3	1491.1
50-54	158.0	-12.5	865.1	278.0	1349.7

Modes of vibration

The research program produces the eigenvalues and eigenvectors of analyzed frames simultaneously. It is capable of producing as many eigenvectors and eigenvalues as there are degrees of freedom in the analyzed frames.

Modes of vibration can be obtained from eigenvectors. Therefore, there are as many modes of vibration as there are degrees of freedom in a system. All or some modes of vibration can be considered in the analysis when the structure response to ground excitation or other dynamic forces is sought. In another case, the response in which all modes of vibration are consideredwill be compared to the response when only few modes are considered.

For the purpose of this study, the first 5 modes of vibration for each of cases A.1, A.2 , and A.3 are shown in figures 4 , 5, and 6 respectively. These modes of vibration represent the modal shapes of the horizontal displacement coordinates of the left hand joints of the frames under consideration, i. e. horizontal displacement coordinates of joints 22, 26 , 30, 34, 38, 42, 46, 50, and 54. These modal shapes are drawn based on the eigenvector values of the horizontal displacement coordinates of the mentioned joints. Eigenvectors for other cases were not tabulated, and higher modal shapes were not drawn.

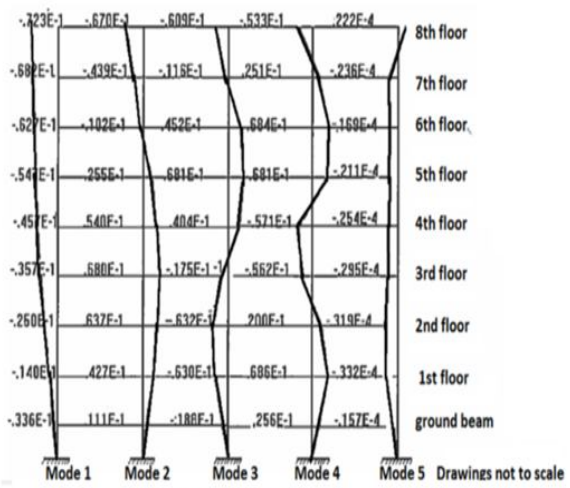


Figure 4 The first 5 modal shapes of Case A-1 (Bare Frame) and eigenvector values of horizontal degrees of freedom of the left hand joints of the frame as obtained from the dynamic analysis

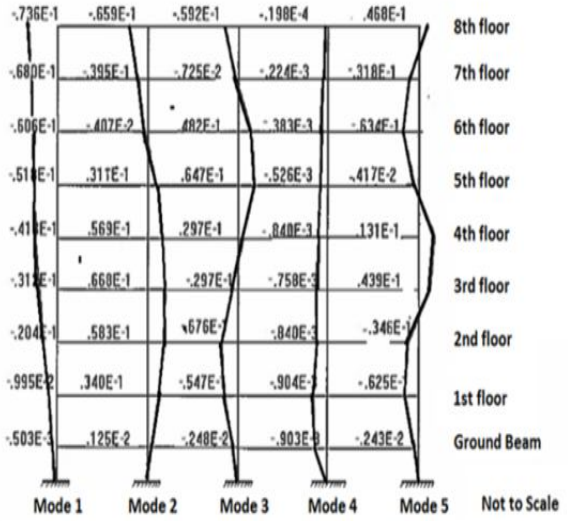


Figure 5 The first 5 modal shapes of case A.2 (Area of Opening/Area of Panel = 0.36) and the eigenvector values of the horizontal degrees of freedom of the left-hand joints of the frame as obtained from the dynamic analysis.

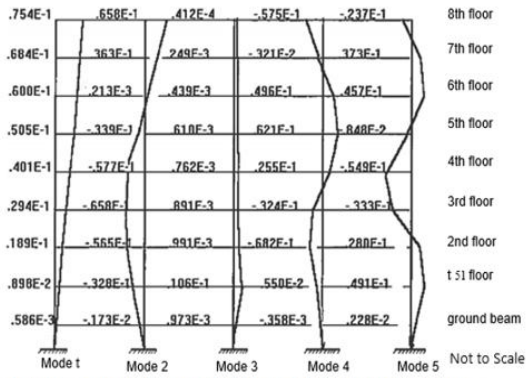


Figure 6 The first 5 modal shapes of Case A.3(opening area= 0.24 of panel area) and eigenvector values of the horizontal degrees of freedom of the left hand joints nof the frame

Studying the first 5 modal shapes shown in these figures may lead to the following observations:

The first mode of vibration is the same for all cases A. 1 to A. 5 ,with the obvious difference in the coordinate values. Note here that some cases have their first modal shape on the left side, while others have it on the right side. This should make no difference for design purposes. The other 4 modal shapes of the 5 cases have in fact the same general shape, but with different eigenvector values and different order for some of them. For example, modal shape no. 4 of case A. 2 is the same as modal shape no. 3 of case A. 4 . The 5 modal shapes of cases A. 3 and A. 5 are almost the same, with, of course, different eigenvector values.

Second:-

The second group of cases to be considered is cases B , which are identical to cases A but damping is included. Damping level in reinforced concrete structures is dependent on the level of deformation or strain in the structural members (Ref. 12) . For reinforced concrete structures with slight cracking, recommended damping values to be used in analysis is suggested by reference no. 12 to be 2% to 3% of critical damping. For structures with considerable cracking, a value of 3 % to 5 % is recommended. For the use of this study, a value of 4% of critical damping will be used for all modes of vibration. Analysis of cases B.1 to B.5 is carried out. These cases are identical to cases A.1 to A.5 respectively, except that damping in cases A is 0.0%, while it is 4.0 % for cases B. For the purpose of this study, lateral displacement of floor joints on the left hand side are chosen as a parameter for comparison. Results of analysis are shown in the following tables

DISCUSSION OF RESULTS

The effect of damping in the analyzed frames can be observed through comparison between tables 3 and 6, which list the lateral displacements of the frames floors of the cases under consideration. From TABLEs 3 and 6, if we choose the lateral displacement of joint no. 54 as a parameter then it can be seen that the lateral displacement of this joint is less in table 6 than in TABLE 3 for all cases 1 , 2 , 3 , 4 , and 5. table 7 shows a comparison between the lateral displacement of the top story joint no. 54 for the 5 damped cases (B) and undamped cases (A).

TABLE 6- Lateral displacements in mm (at time T = 0.60 second after beginning of application of ground excitation) of the joints indicated for the 5 cases mentioned. The cases are arranged in the order of increasing stiffness. Damping is 4% of critical damping for all modes of vibration.

Joint No.	Case B.l no wall panels	Case B.2 (A opening /Apanel) =0.36	Case B.3 (A opening /Apanel) =0.24	Case B.4 (A opening /Apanel) =0.12	Case B5 No Openings in panels
22	3.58	0.71	- 5.39	-2.88	-6.31
26	14.84	17.22	-79.78	-14.51	-58.09
30	26.17	42.56	-163.8	- 26.89	-127.2
34	36.91	71.59	- 250.7	- 39.44	-200.3
38	44.21	101.3	- 323.7	- 51.29	-278.5
42	52.89	129.8	- 391.5	-60.90	- 357.9
46	59.17	156.3	-453.8	-68.03	-432.6
50	63.24	178,4	-508.9	-72.60	- 501.8
54	66.18	196.1	-554.7	-75.24	-563.9

Table7 – Reduction of lateral displacement of joint 54 for damped and undamped cases.

Case No.	Undamped Cases A	Damped Cases B	Reduction of displacement due to damping
1	69.46	66.18	4.7%
2	228.2	196.1	14.1%
3	660.8	554.7	16.1%
4	-58.56	-75.24	12.1%
5	-649.6	-563.9	13.2%

Table7 shows that damping present in actual structures has the effect of reducing the frames lateral displacements. This effect is not large for bare frames (no panels) in which only 4.7 % reduction in displacement for case no. 1 is found. However, damping effect to reduce lateral displacement increases by 3 times or more when infill panels are available in the middle panels as shown in figure2 . In other words, most of damping in infilled frames is caused by the presence of infill panels, rather than by the frames proper.

CONCLUSIONS

1. Infill panels in structural frames cause its fundamental period of vibration as well as the higher periods to decreaseconsiderably.
2. Openings in infill panels cause the fundamental period of vibration, as well as the higher periods of the infilled frame to increase.
3. For a particular plane frame, a regular pattern of infill panels and openings is expected to keep the first few modal shapes unchanged but with different values of the eigenvectors.
4. Irregular patterns of infill panels lead to a sudden and unpredicted change in the modal shapes of a building frame.
5. Changes in the modal shapes due to the presence of infills and openings are followed by changes in the distribution of member forces in the frames. Therefore, for seismic design of buildings. the possible irregular configurations of infills and openings should be considered to design for maximum member forces anticipated.
6. Damping in infilled frames has a greater effect (3 - 4 times more) in reducing the lateral displacements of these frames than damping in bare frames. Hence, member forces in infilled frames are more reduced by damping than in bare frame.
7. Using the modal superposition method, It is accurate enough to consider only the first few modes of vibration when the structure response is calculated. However, it is found that shear forces in columns require more modes than lateral displacements to reach a specific level of accuracy, i .e. t to reach a specified maximum relative error.
8. Consideration of the structure foundation members in the dynamic analysis during earthquake excitation results in repeated and closely spaced periods of vibration . It also results in larger lateral displacements, and member forces than otherwise. This is due to the rotation and displacements of the foundation members, i .e. ,due to the structure - foundation interaction. It is also due to the inclusion of the repeated and longer periods of vibration in the calculation of the structure response.

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