Jun 1st, 12:00 AM

Dynamic Response of Infilled Multistorey Steel Frames

Craig J. Miller

Abo-Elkhier Serag

Dynamic Response of Infilled Multistory Steel Frames

Craig J. Miller* and Abo-Elkhier Serag+

Introduction

In recent years, the dynamic response of multistory structures to wind and earthquake loading has attracted a great deal of attention among structural engineers. In particular, increased incidence of occupant comfort and perception problems due to high wind loads has led to substantial efforts to determine acceptable values of deflection for multistory frames. One method which has been suggested for controlling the drift of multistory steel frames is to use light gage steel vertical infills acting as shear diaphragms to resist wind loading.

The previous research dealing with the use of light gage panels to control drift has concentrated on the response of the structure to static loads. The objective of the work reported here was to study the effect of infills on the dynamic response of typical steel structures to both wind and earthquake loads. Assuming that a given multistory steel frame has been designed to resist the applied loads at allowable stresses, but did not satisfy deflection limitations, two alternatives for stiffening the frame were considered. One was to stiffen the frame by increasing the moment of inertia of columns and girders. The second was to leave the frame member sizes the same and add infill panels to reduce frame deflections. The intent was to tune both frames to approximately the same deflection under service wind load. The frames were then analyzed under the action of wind and earthquake loads. The relative effectiveness of the two alternative approaches to stiffening could then be compared.

*Assistant Professor of Civil Engineering, Dept. of Civil Engineering, Case Western Reserve University.

+Graduate Student, Dept. of Civil Engineering, Case Western Reserve University.

Previous Research

Until recently, it has not been possible to do any more than a very crude analysis to determine the effect of cladding or infills on the behavior of a frame. With the development of finite element techniques and rapid digital computer capabilities, more rational analyses became possible. Many investigators have attempted to develop means to predict the load-deformation response of cold-formed diaphragms when subjected to shear loads. A complete bibliography of this work can be found in Ref. 1. Until the work of Ammar and Nilson, prediction of diaphragm behavior was based on empirical equations.

Ammar and Nilson[12,13] put the prediction of inplane diaphragm behavior on a more analytical footing by use of matrix finite element techniques. They modelled the surrounding beams and purlins using standard beam elements and modelled the connectors as linear spring elements. The corrugated sheets were converted to equivalent flat sheet orthotropic members which were modelled using orthotropic plane stress elements. This analysis agrees with load tests within 15% for corrugated diaphragms which do not have a continuous flat sheet in the plane of loading. The finite element technique reduces the required experimental work to measuring the stiffness of connectors and the shear modulus and weak direction elastic modulus of the corrugated sheet.

Miller[9,10,11] used the work of Ammar and Nilson as a basis for a study of the suitability of cold formed panels for controlling drift in multistory structures. The construction assumed by Miller is shown in Fig. 1. The connection between the infill and the frame, made by means of marginal members, is assumed to meet two requirements. The first is that transfer of vertical load from girder to panel is prevented or at least minimized. The second is that the end of the panel is attached to the marginal member in such a way that the cross section profile of the panel at the ends will not distort when the panel is loaded in shear. Two details which accomplish these objectives are shown in Fig. 1b and 1c.

The finite element model of the panel used in that work is shown in Fig. 2. Based on the results of a number of single story, single bay frame analyses, Miller concludes that the infill panels can be used effectively to control drift. Because this model has a large number of degrees of freedom, Miller developed a modified model shown in Fig. 3 that is connected to the frame only at the corners of the panel. In this way, the stiffness matrix for the panel can be reduced to a 12 x 12 matrix involving only degrees of freedom at the corners. The panel stiffness can be derived independently of the frame stiffness and the same stiffness matrix used for all panels. The analysis of the multistory frame can then be done with no more degrees of freedom than are involved in this analysis of the bare frame. Using this model, a 26 story steel frame was analyzed with and without infills and the efficiency of the panels in reducing drift demonstrated.

Oppenheim[14] developed similar techniques for the analysis of infilled frames, including a corner connected model similar to that used by Miller. Although he used a different approach to determining the shear stiffness of a corrugated infill, his results lead generally to the same conclusions as Miller's results. Oppenheim also studied the dynamic behavior of infilled multistory frames. He did an elastoplastic analysis of the response of an infilled frame to a sinusoidal base motion. The frame members were assumed elastoplastic while the panels were assumed linear and elastic to an abrupt failure load. His results indicate that higher mode influences may lead to premature failure of panels in the upper stories. Recent papers[5,6] by El-Dakhakhni and reports by Bryan and Davies[2] and Davies[3] indicate continued interest in use of light gage infills as partitions and shear resisting elements in multistory buildings.

Analysis Technique and Modelling

The work to be described here follows closely the modelling and analysis techniques previously reported by the senior author. The two assumptions regarding the connection of infill to frame mentioned previously were also used here. The model of the panel used in the present work is shown in Fig. 4.

The differences between the current and previous work can be seen by comparison of Fig. 3 and 4. In the previous work, the seam connectors were assumed flexible in both horizontal and vertical directions. In the present work the seam connectors were assumed rigid horizontally and flexible vertically. The end connectors were assumed rigid vertically and flexible horizontally. In the dynamic studies each corrugated sheet was modelled by 24 orthotropic plane stress elements, while in the previously reported static work, each sheet was modelled by 54 orthotropic plane stress elements.

As in the previous work, the continuous connection between frame and infill was eliminated and the two were assumed to be connected only at the corners, thus reducing the complexity of the analysis substantially. All of the computer work was done using the ANSYS computer program[4]. The panel stiffness matrices were created using the superelement feature from ANSYS. This feature allows one to combine a large number of elements to form a superelement stiffness matrix and to then condense out degrees of freedom which are not of major interest. In the problem studied here, all of the degrees of freedom associated with nodes other than the corner nodes can be condensed out. The resulting 12 x 12 stiffness matrix is then stored on tape for use whenever an infilled frame is to be analyzed.

The elements used to model the light gage sheet are the two dimensional isoparametric quadrilateral elements available in ANSYS. The marginal members are modelled using standard beam elements and the springs are modelled using linear spring elements. Because fewer elements are used in this work than were used in the previous work, the resulting panel model is somewhat stiffer than the model used previously.

The earthquake analysis of the structure was done using the modal analysis capability of ANSYS. Included in that capability is a dynamic reduction scheme that makes it possible to condense out of the eigenvalue problem those degrees of freedom which are not felt to be significant. For the multistory frame analyses described here, the retained degrees were the horizontal displacements at certain floors. Only the first three modes were used in determining forces and displacements. The dead loads on the structure were input as lumped masses at the nodal points. The mass of the structural members including the infills was included by means of the consistent mass matrix.

Damping values were not included directly in the computer analysis. Damping was included by inputting the response spectra for a given damping value. The response spectra used are shown in Fig. 5. The spectra were developed for a maximum ground acceleration of 0.2 g. The particular response spectra used are the modified Newmark Horizontal ground response spectra which were widely used in the design of nuclear power plants until quite recently.

The choice of damping values to be used in the analysis posed a major problem. There is no available data to guide the designer in the choice of a damping value. for the cold formed infills. It seems reasonable to expect that there would be more energy dissipation in a cold formed diaphragm than in an ordinary welded or high strength bolted steel frame. The large number of connections and the likelihood of many small areas of yielding surrounding them would tend to increase the damping capability. In addition, there is likely to be some dissipation of energy due to friction as adjacent sheets move relative to one another. The report of Sexsmith[15] indicates that there is in fact frictional damping.

For the analyses to be discussed below, the assumed damping values expressed as a percentage of critical values are:

Infilled frame	-	earthquake	-	5%, 10%
	-	wind	-	2%
Bare frame	-	earthquake	-	2%
	-	wind	-	0.5%

Two values of damping are used for the earthquake analysis for the infilled frame becasue of the lack of knowledge of a reasonable value. It is likely that all of the damping values are conservative, i.e., lower than they really are.

The wind analyses of the structures were carried out using the detailed procedure given in the 1975 edition of the National Building Code of Canada[16,17] This procedure is a simplified way to account for the dynamic nature of the loading and the dynamic characteristics of the structure. Both the natural frequency and the damping are included in the calculation of the wind loading. The calculated wind load is then applied to the structure as a static load. In calculating the wind loads, it was assumed that the buildings were located in a center city exposure. Since the primary interest was in the response at service load levels, the wind loads were based on a 10 year return period for the wind.

Also included in the detailed procedures are approximate formulae for calculating maximum acceleration of the structure due to the wind load. The value of the acceleration is an important quantity in determining whether or not occupants will experience any discomfort due to wind-induced vibrations. Research indicates that an acceleration of 0.4 - 0.8% g will be felt by most people. Therefore, the occurrence of accelerations of those magnitudes should be relatively infrequent. In calculating the accelerations for the structures discussed below, it has been assumed that the cross-wind acceleration is not a problem; only the along-wind direction acceleration is calculated.

Twenty-Six Story Frame

The twenty-six story frame analyzed statically by Miller[9] shown in Fig. 6, was chosen as the first structure to be analyzed dynamically. In order to make a fair comparison between stiffening the structure by means of infills and stiffening it by means of additional moment of inertia in the columns and girders, the procedure was as follows. It was assumed that a drift index of 1/600 based on an overall height and maximum deflection would be sufficient to give adequate service load performance. The wind load used for the wind analyses was based on a 10 year return period wind speed of 50 mph (80.5 kmph). That speed converts to a basic wind pressure of 7.4 psf (354.5 N/m^2) at 30' (9.15 m) above ground.

The bare frame before stiffening is shown in Fig. 6. The drift index of the frame was 1/400 for the bare frame. Using the approximate method proposed by Fleischer[7], the frame was tuned to achieve a drift index of 1/600. The stiffened frame is shown in Fig. 7. Due to the approximate nature of Fleischer's method and the discrete nature of available moments of inertia, the frame shown in Fig. 7 has a drift index of 1/750 when analyzed for the 10 year Toronto wind. The deflected shapes for the three structures are shown in Fig. 8.

Based on past experience, 16 gage panels were chosen for use in stiffening the frame. The frame was analyzed with 16 gage infills full height in the center bay. The deflection index for the infilled frame is 1/1110. Because of the need to keep the panel shear below the buckling load of the panel, the infilled frame is quite stiff. In actual practice, the panels in the upper stories could be reduced in thickness, since the shears are lower there. Plots of deflection versus height for the original frame and the two stiffened frames are shown in Fig. 8.

In these times when the possibility of serious shortages of materials faces us, it is interesting to compare the total weight of steel added by the two alternative means of increasing the stiffness of the frame. Stiffening the frame by adding additional material to the columns and girders added 55 tons (49876 kg) to the frame weight. 14 tons (12701 kg) of sheet are required to infill the frame full height, indicating a substantial advantage to the infills. It should be noted that given today's economic conditions, the advantage in a cost comparison would rest with stiffening by increasing the moments of inertia.

Since the maximum acceleration of the structure is an important determinant of occupant reactions, the approximate procedure given in the 1975 Supplement No. 4 to the National Building Code of Canada[17] was used to calculate accelerations for the stiffened frame and the infilled frame. The calculated accelerations are 1.53% g for the infilled frame and 2.99% g for the stiffened frame. The stiffened frame is at the upper limit recommended by Supplement No. 4, while the infilled frame is toward the lower limit of the recommended range of 1 to 3%. The additional stiffness and damping provided by the infill panels lead to a substantial reduction in the acceleration to be expected.

It is interesting to note that both of these structures have fairly high predicted accelerations due to wind, even though both have drift indices well below the 1/500 or 1/600 often used as a guide to providing adequate stiffness. This is an indication of how difficult it is to limit accelerations to recommended values for steel structures which have relatively low damping and low mass, particularly when the calculation is based on structural frame behavior alone.

The two alternative frames were then analyzed for earthquake loading. The damping value assigned to the bare frame was 2%, while the analysis of the stiffened frame was done using both 5% and 10% damping. Three modes were used in developing the response of the frames. The lowest natural frequency of the infilled frame is .361 hz. The total responses of the two frames were obtained from the modal response by taking the square root of the sum of the squares of the modal responses.

Figure 9 shows the column and panel shears for the infilled frame for both damping values as well as the column shears for the stiffened frame. The figure indicates that for 5% damping the total shear at a given floor in the infilled frame is roughly the same as it is for the stiffened frame with 2% damping. The infilled frame with 10% damping has about 10% lower total shear than the others. Fig. 10 shows a plot of root mean square deflections for the infilled frame for 5 and 10% damping and for the stiffened frame. The drift-indices for the three cases are 1/555, 1/637, 1/380 respectively. These results indicate that the infills lead to a stiff frame, while at the same time because of the relatively large damping, the forces are not excessive. Recent experience in the Managua earthquake seems to indicate that greater stiffness than is present in the ductile, moment resisting space frame type of structure is desirable to help limit damage to nonstructural elements such as partitions and building contents.

40 Story Building

The 40 story frame shown in Fig. 11a was analyzed in the same way as the 26 story frame. The frame is taken from Ref. 8 . An approximate value for the drift index of the frame of 1/375 was obtained using the method given by Fleischer. The frame was then tuned to bring the drift index to 1/600. The girder sizes were increased to the values shown in Fig. 11b to accomplish the stiffening. As an alternative way of stiffening the frame, infill panels were added full height in the center bay. Because of the increased height of this frame, it was decided to use 12 ga. panels in the lower 20 stories and 16 ga. panels in the upper 20 stories.

The stiffened frame was analyzed for the wind loads resulting from a reference wind speed of 65 mph (104.7 kmph) which corresponds to a reference pressure of 11.4 psf (546.1 N/m^2). The National Building Code of Canada procedure was used to calculate wind load. The drift index actually achieved was 1/550, not 1/600 because of the approximations in Fleischer's method. The infilled frame was analyzed for loads resulting from the same wind speed. The drift index for the infilled frame is 1/790. (The deflected shapes are shown in Fig. 12.)

As with the 26 story frame, there is a substantial advantage in quantity of steel required to achieve the desired stiffness.

Again using the suggested method of the latest edition of the Canadian Code, accelerations were calculated for the infilled frame and for the stiffened frame. The infilled frame had a calculated acceleration of 2.77% of gravity when subjected to the 10 year return period wind. - The corresponding figure for the stiffened frame is 6.07% g. The infilled frame is barely within the recommended limit given by the Canadian Code, while the stiffened frame has a calculated acceleration more than double the recommended value and would clearly require additional stiffening to make it acceptable.

The two alternative frames were next analyzed for the effect of earthquake loads, using the same spectrum as for the previous frame. The fundamental frequency of the infilled frame is .391 hz, of the stiffened frame 0.362 hz. Because of the size of the frame, the horizontal displacement at every fourth floor was retained as a degree of freedom in the solution for the natural frequencies, so the solution for the 40 story frame has a higher degree of approximation than the one for the 26 story frame. The root mean square deflections (based on the first three modes) for the stiffened frame and the infilled frame with 5% and 10% assumed damping are shown in Fig. 13. The drift indices for the three cases are 1/491, 1/643, 1/760 respectively. As with the 26 story frame, for 0.2 g maximum ground acceleration, the need to provide a stiff frame for serviceability reasons leads to drift indices which are well within the recommended maximum of 1/200.

Conclusions

The two examples presented here indicate the effectiveness of cold formed infill panels in reducing the drift of steel frames subjected to lateral loads. The relative effectiveness of cold formed infills compared to stiffening the frame by adding material to the columns and/or girders is to some extent dependent on the values of damping assumed in the calculations. Some guidelines are available to assist the engineer in choosing damping values for welded steel frames. There does not seem to be comparable information available for light gage steel diaphragms. Before a definitive comparison of various methods of stiffening can be made, experimental work to determine the damping values for cold formed diaphragms must be done. The examples presented indicate some advantage can be gained using light gage panels.

Calculation of acceleration values using the method proposed in Supplement No. 4 to the National Building Code of Canada indicates that use of infills leads to lower accelerations due to greater stiffness combined with the higher damping assumed for the infilled frame. This again points up the need for experimental data on the damping to be expected in cold-formed steel diaphragms. It is difficult to get a completely fair comparison of the two methods of stiffening considered here because it is difficult to tune them both to the same deflection index. The approach taken here was to use a reasonable size panel, considering buckling loads, for the infilled frame and then tune the bare frame to a reasonable drift index. Done this way, the frames compared have quite different drift indices under the action of the wind load. However, it was felt that this is the way the comparison would be made in the design office.

Due to the high stiffness required to give satisfactory service load performance, the response of these structures to earthquake loading is well within currently recommended maximum values, even though the input ground motion was taken to have a maximum acceleration of 0.2 g, which is a relatively high value. Because of the rather high values of damping assumed for the infilled frames they exhibit relatively high stiffness with relatively low forces.

It should also be noted that in the work done here, there did not seem to be excessive shears in the upper story panels under earthquake loads, as seemed to occur in the work of Oppenheim[14]. Indications from recent earthquakes are that the ductile moment resisting space frame concept produces a structure which withstands earthquakes well itself, but which leads to significant damage to non-structural elements and contents because of the large displacements involved. If, as now seems likely, codes will require stiffer structures in the future, cold-formed infills offer a means of achieving stiffness without significantly increasing the level of force to be resisted.

References

1. Ammar, A.R., "Analysis of Light Gage Steel Shear Diaphragms, Part I," Report No. 350, Department of Structural Engineering, Cornell Univ., Ithaca, 1972.

2. Bryan, E.R., and Davies, J.M., "Diaphragm Action in Multistory Buildings," Report No. 75/70, Dept. of Civil Engineering, Univ. of Salford, Salford, England, 1976.

3. Davies, J.M., "The Design of Multistory Buildings Stiffened by Diaphragm Action," Report No. 77/95, Dept. of Civil Engineering, Univ. of Salford, Salford, England, 1977.

4. DeSalvo, G.J. and Swanson, J.A., ANSYS Engineering Analysis System User's Manual, Swanson Analysis Systems, Inc., Elizabeth, Pa., Oct. 1972.

5. El-Dakhakhni, W.M., "Shear of Light-Gage Partitions in Tall Buildings," Journal of the Structural Division, ASCE, V. 102, No. ST7, July 1976.

6. El-Dakhakhni, W.M., "Effect of Light-Gage Partitions on Multistory Buildings," Journal of the Structural Division, ASCE, V. 102, No. ST1, Jan. 1977, pp 110-132.

7. Fleischer, W.H., "Simplified High-Rise Drift Analysis and Optimized Adjustment," American Institute of Steel Construction Engineering Journal, V.9, No.2, 2nd Quarter, 1972.

8. Fleming, J.F., Podolny, W., Jr., and Rile, T.S., "Drift Study of Tapered Frameworks," Civil Engineering, Aug. 1967, pp 38-41.

9. Miller, C.J., "Analysis of Multistory Frames with Light Gauge Steel Panel Infills," Report No. 349, Dept. of Structural Engineering, Cornell Univ., Ithaca, 1972.

10. Miller, C.J., "Drift Control with Light Gage Steel Infill Panels," Proceedings of the Second Specialty Conference on Cold-Formed Steel Structures, Yu, W.W., Ed., St. Louis, 1973, pp 437-466.

11. Miller, C.J., "Light Gage Steel Infill Panels in Multistory Steel Frames," American Institute of Steel Construction Engineering Journal, V. 11, No.2, 2nd Quarter, 1974, pp 42-47.

12. Nilson, A.H., "Analysis of Light Gage Steel Shear Diaphragms," Proceedings of the Second Specialty Conference of Cold-Formed Steel Structures, Yu, W.W., Ed., St. Louis, 1973, pp 325-363.

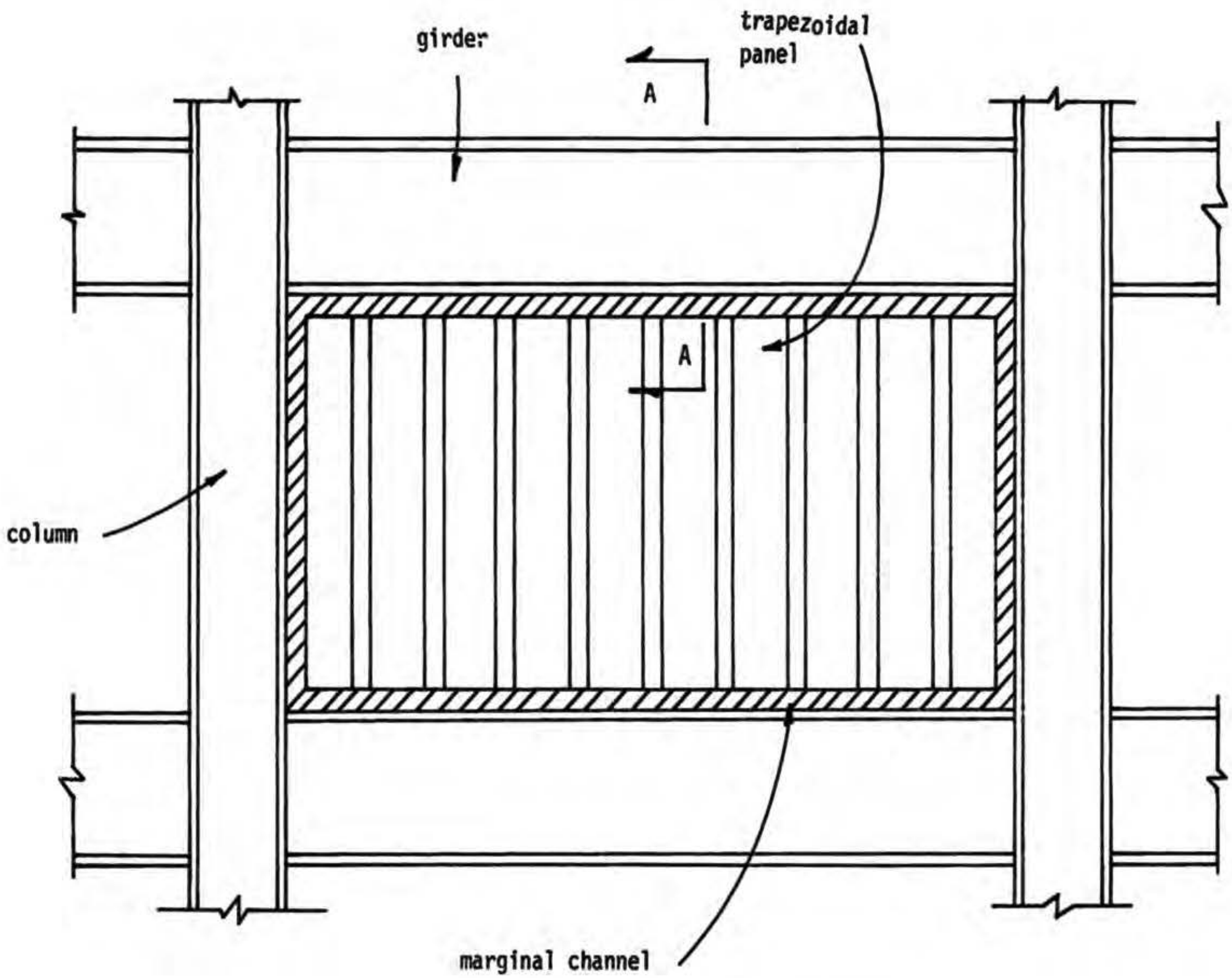
13. Nilson, A.H., and Ammar, A.R., "Finite Element Analysis of Metal Deck Shear Diaphragms," Journal of the Structural Division, ASCE, V. 100, No. ST4, April, 1974, pp 711-726.

14. Oppenheim, I.J., "Control of Lateral Deflection in Planar Frames Using Structural Partitions," Proceedings of the Institute of Civil Engineers, London, England, V. 55, June 1973, pp 435-445.

15. Sexsmith, R.G., "Behavior of a Light Gage Steel Diaphragm," Dept. of Structural Engineering Report on Project 169, Cornell Univ., Ithaca, 1975.

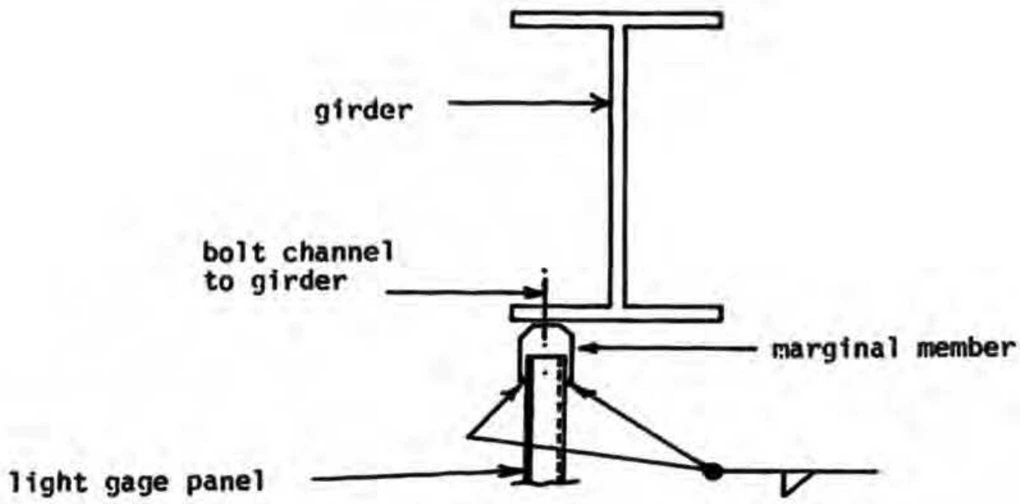
16. National Building Code of Canada, National Research Council of Canada, Ottawa, 1975.

17. Supplement No. 4 to the National Bufldıng Code of Canada, National Research Council of Canada, Ottawa, 1975.

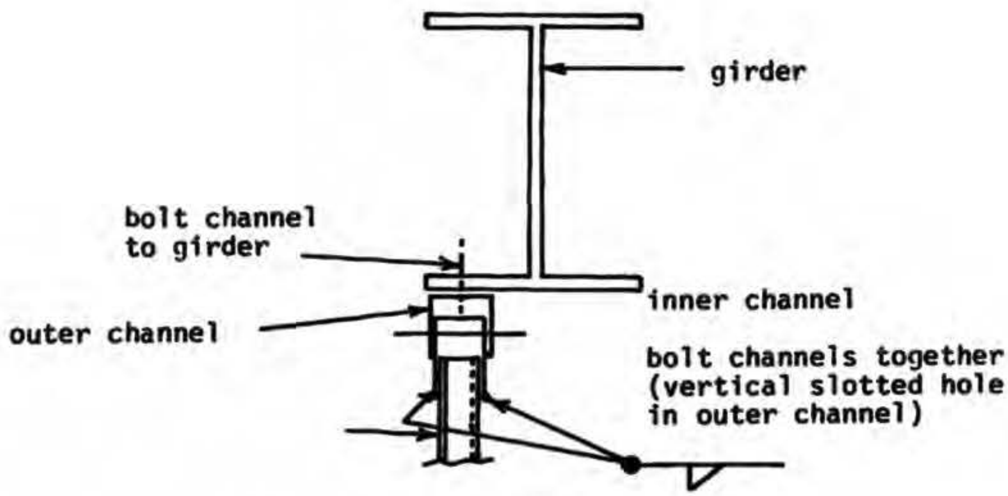


(a) Schematic of proposed construction

Fig. 1 - Proposed Construction for Infilled Frames

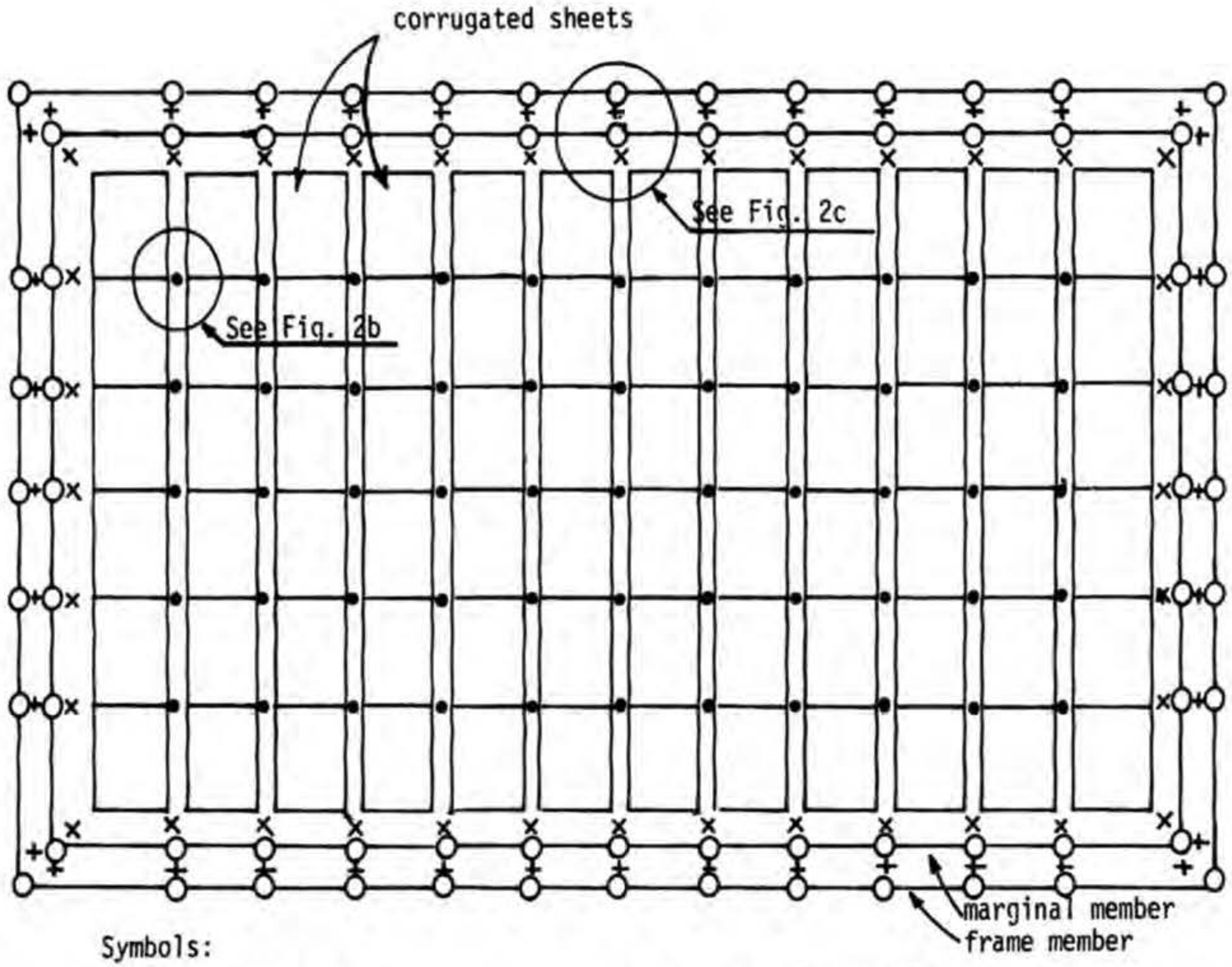


(b) Section A-A



(c) Alternate Section A-A

Fig. 1 - Proposed Construction for Infilled Frames (con't.)



Symbols:

0 - nodal point

+ - marginal member to frame member flexible link

x - edge or end connector

. - seam connector

Fig. 2a - Idealization of Fully Connected Model

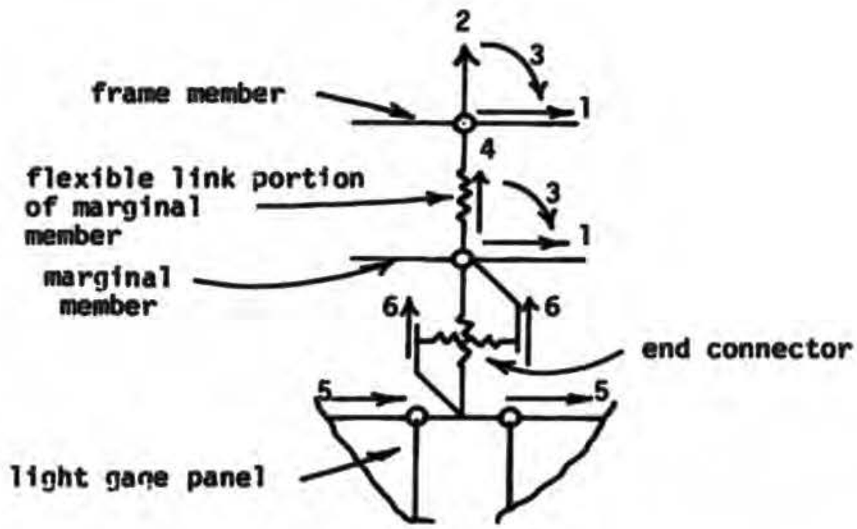


Fig. 2b - Degrees of Freedom at Connection of Frame and Panel for Fully Connected Model of the Infilled Frame

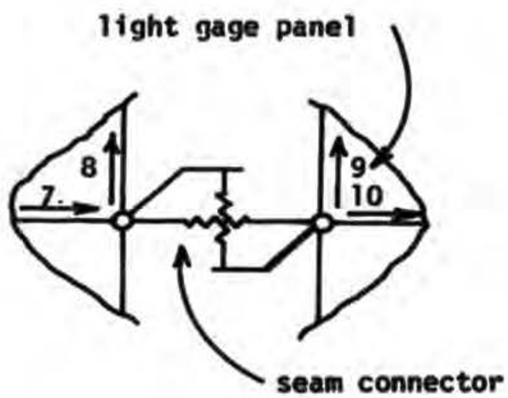


Fig. 2c - Degrees of Freedom at Seam Connection for Fully Connected Model of the Infilled Frame

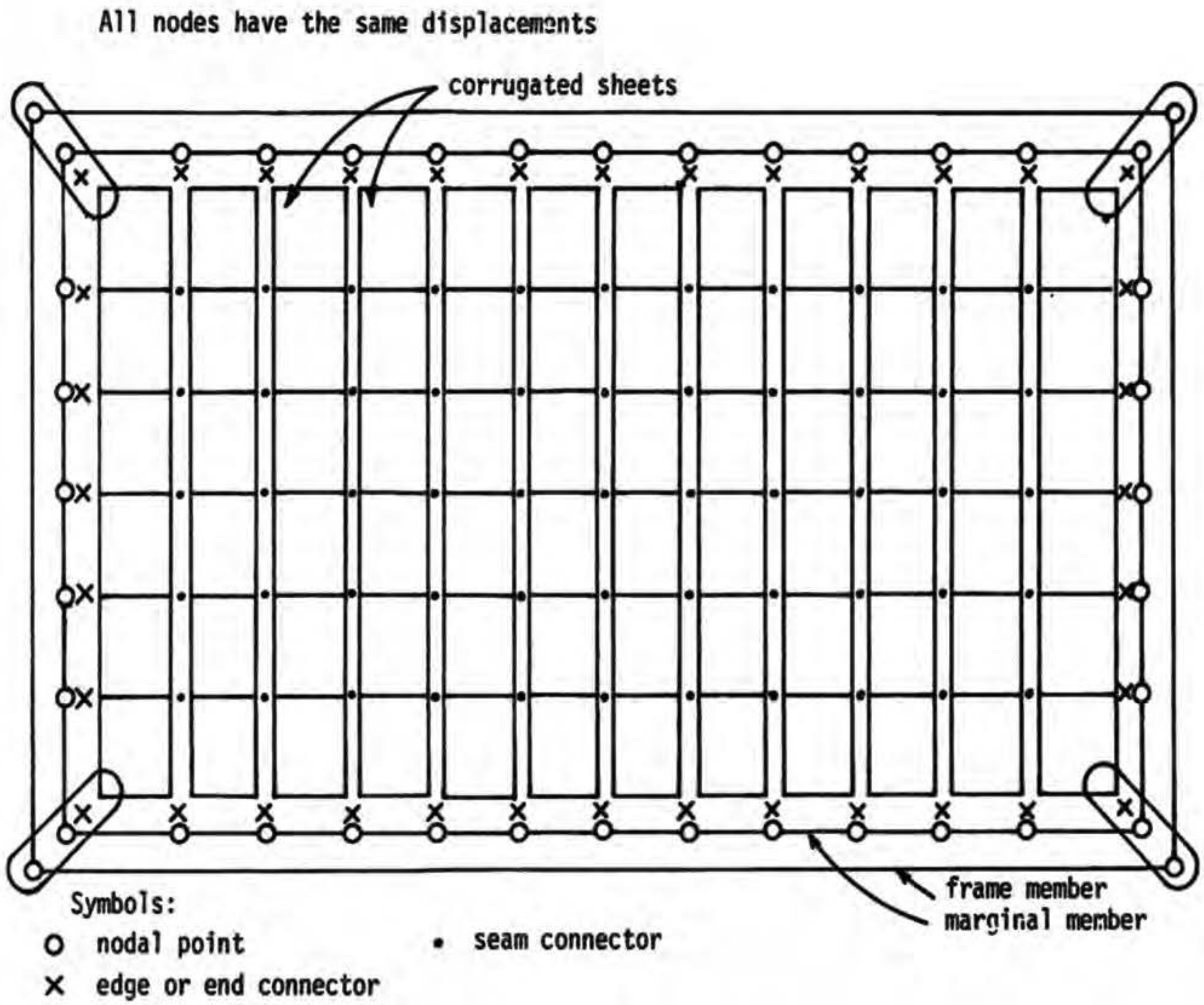
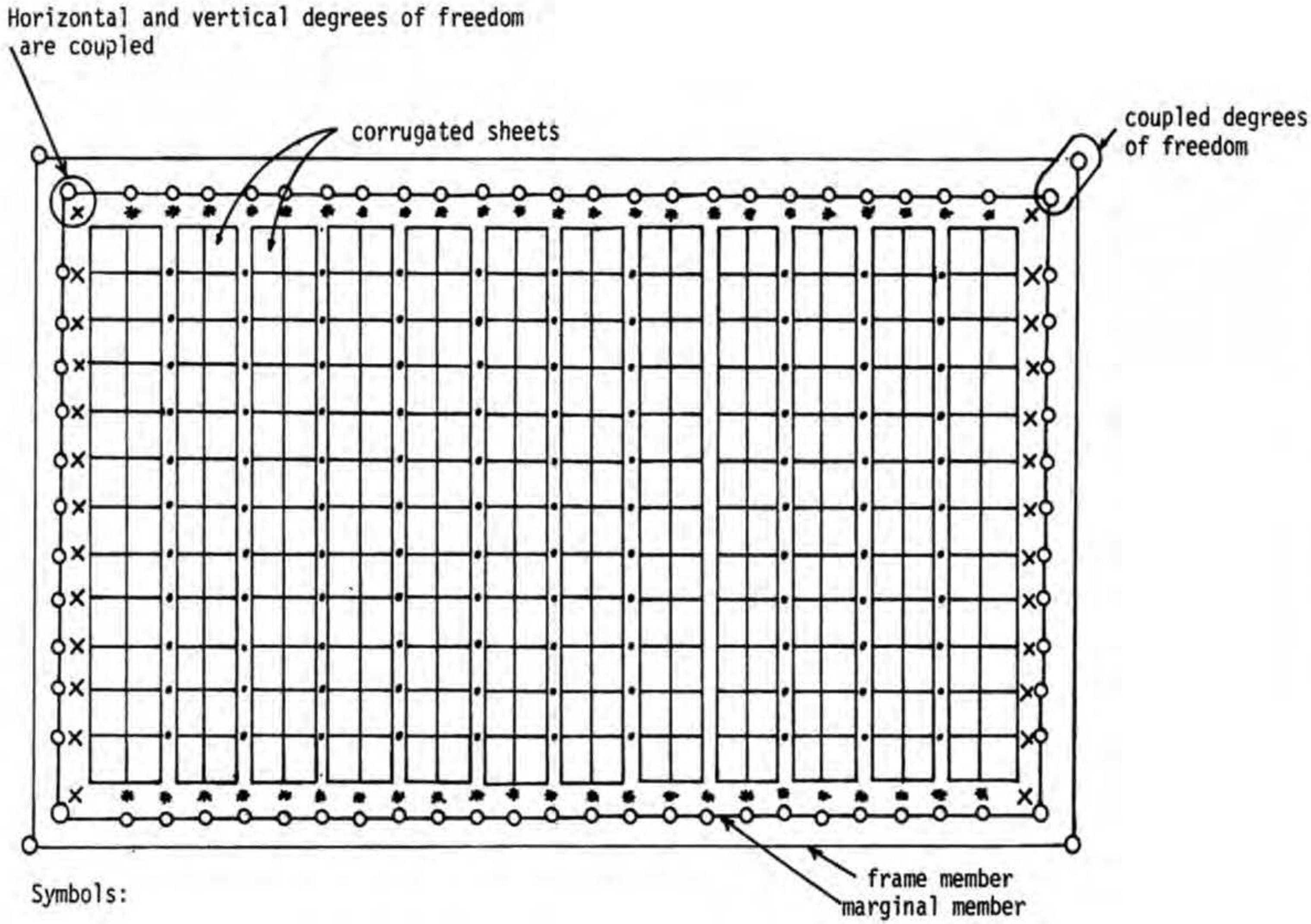


Fig. 3 - Idealization of Corner Only Model

Fig. 4 - Idealization of the Panel (Superelement)



Symbols:

o nodal point

× edge connector ⟶ Horizontal degree of freedom is coupled ⟵ · seam connector

* end connector ⟶ Vertical degree of freedom is coupled

Fig. 5 - Modified Newmark Horizontal Ground Response Spectra

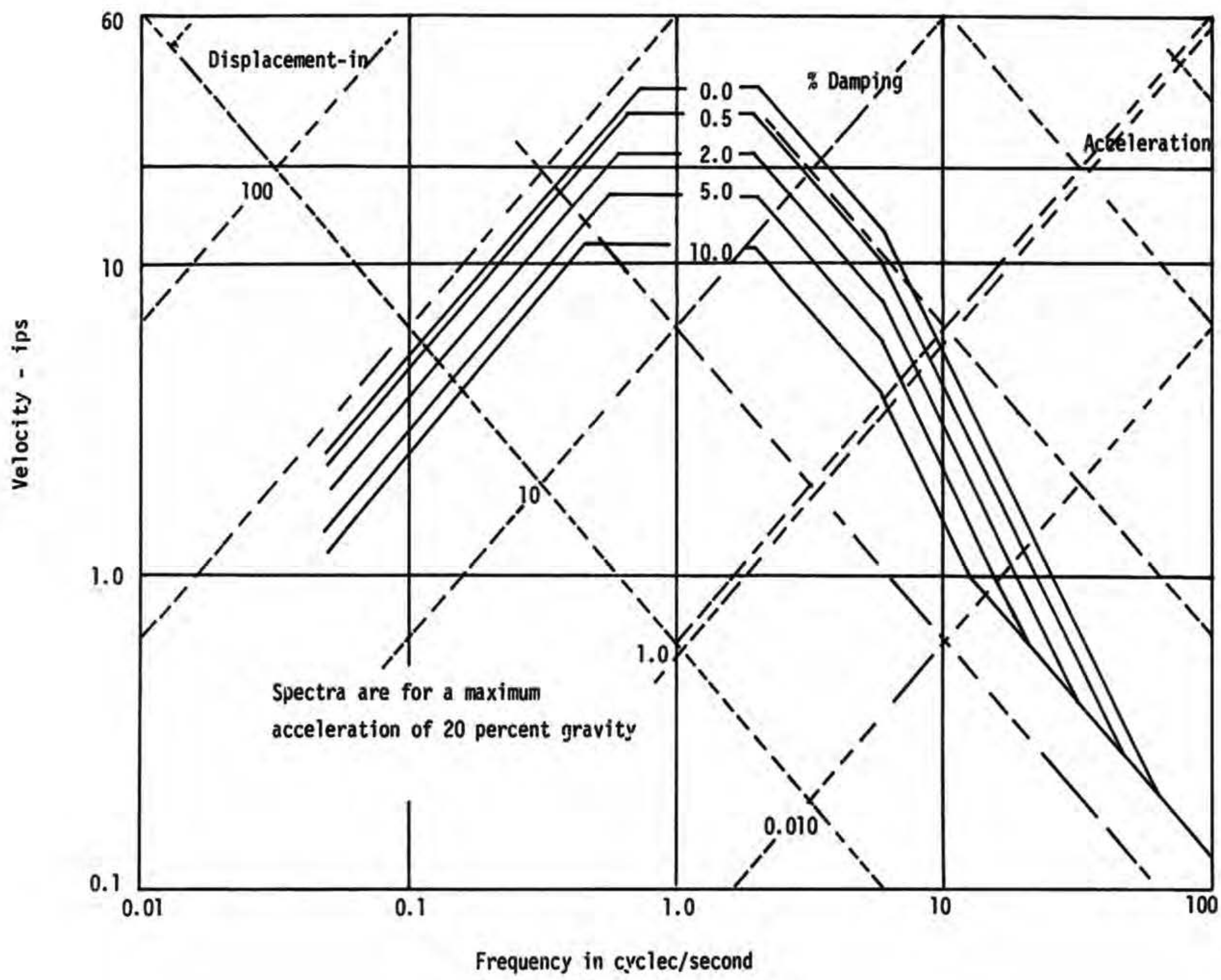
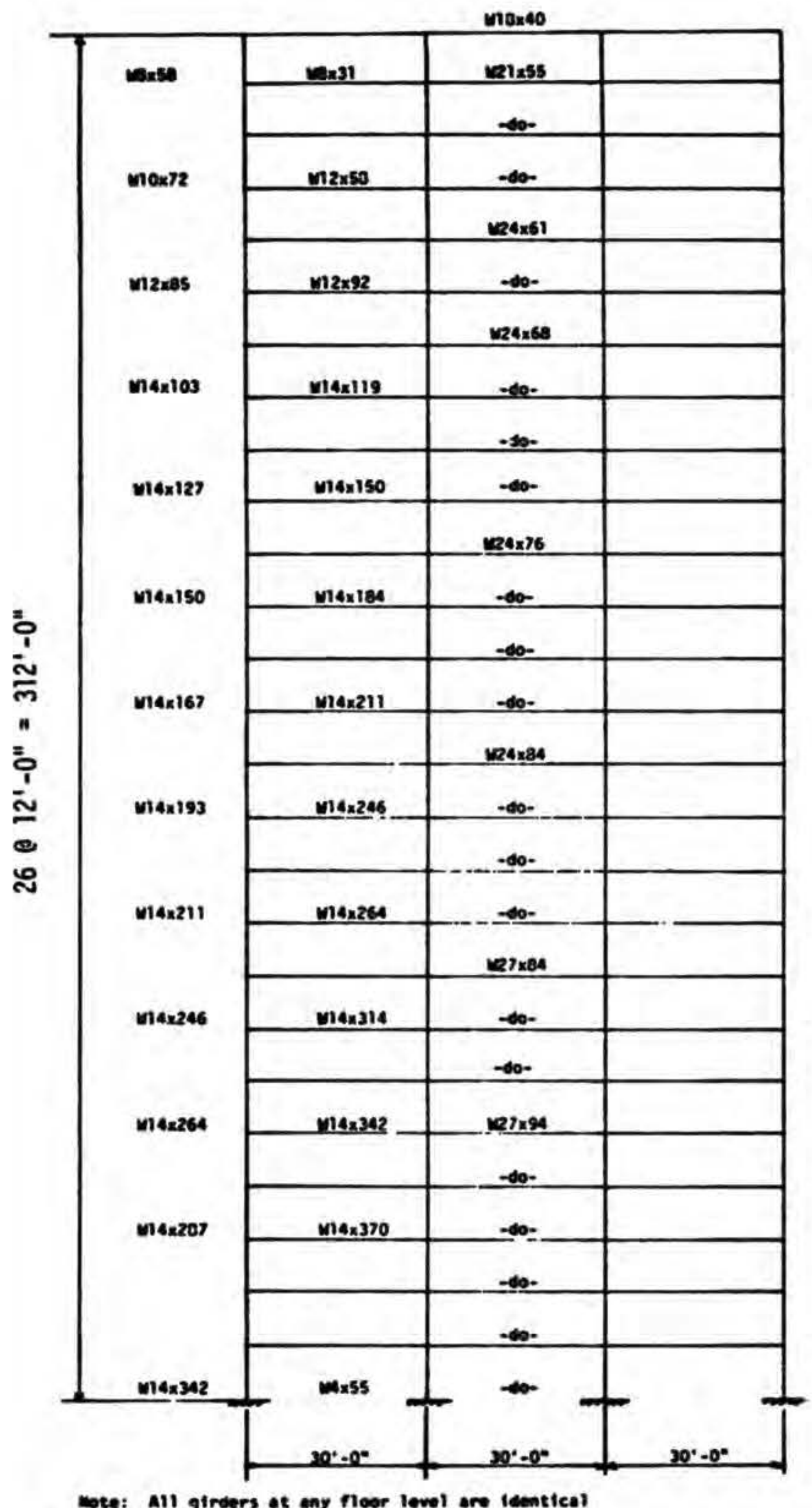


Fig. 6 - Twenty-Six Story Frame - Unstiffened



Note: All girders at any floor level are identical
All columns are two stories high
Framing is symmetrical

Note: At any floor level, all girders are identical
Framing is symmetrical, all columns are
two stories high.

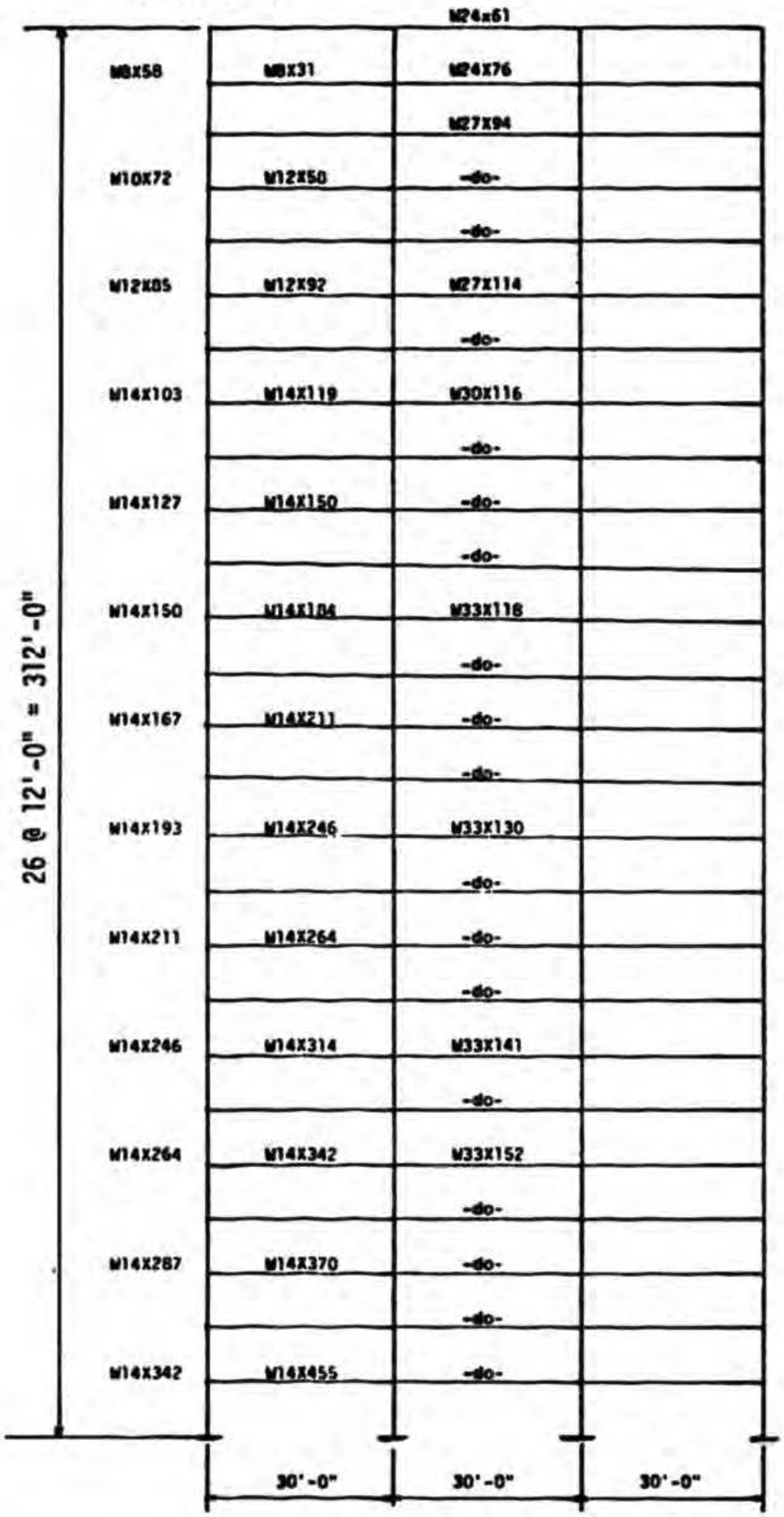


Fig. 7 - Twenty-Six Story Frame - Stiffened

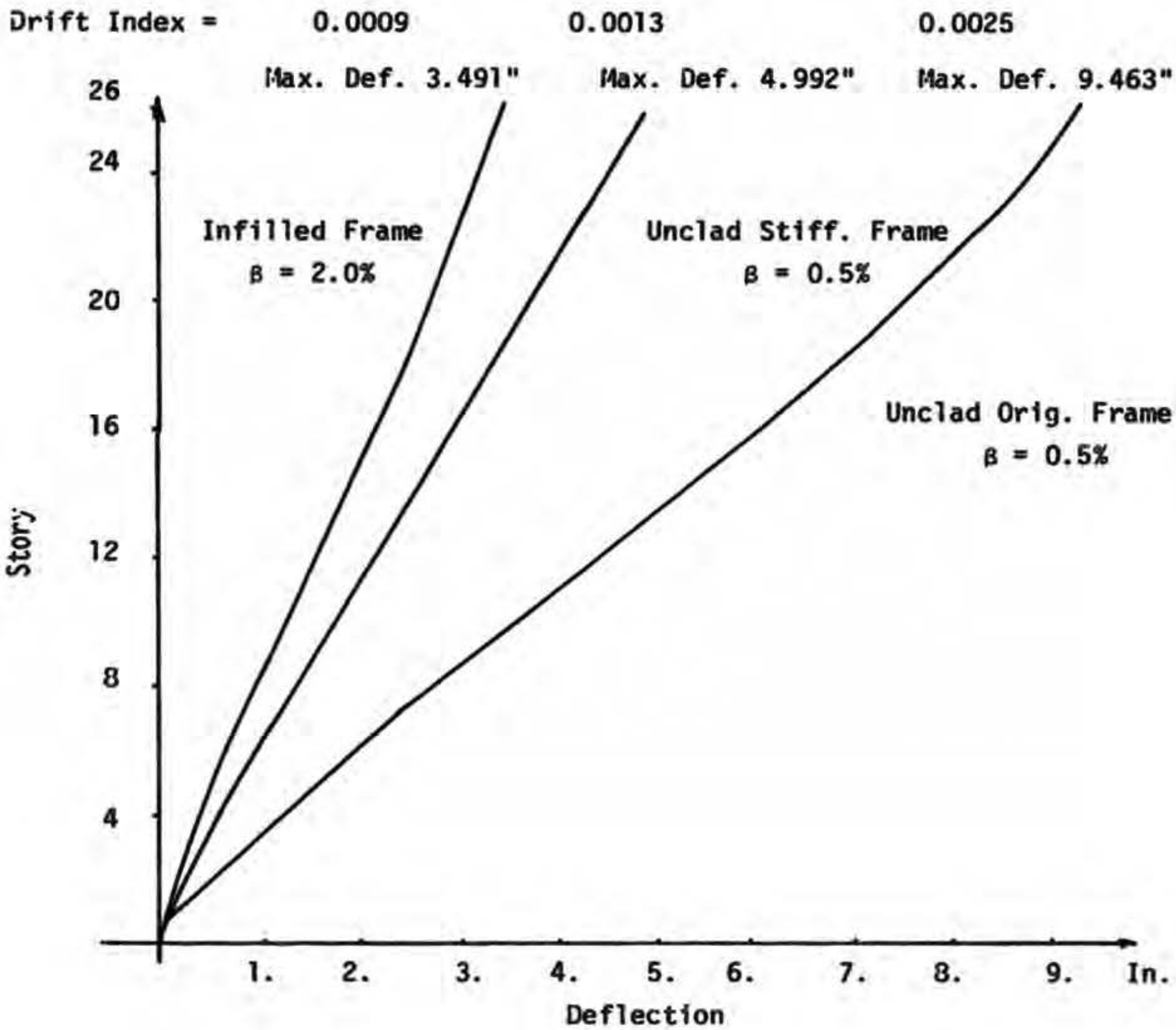


Fig. 8 - Wind Load Defl. of Infilled 26 Story Frame
Deflection of Bare Frame
and Stiff Unclad Frame

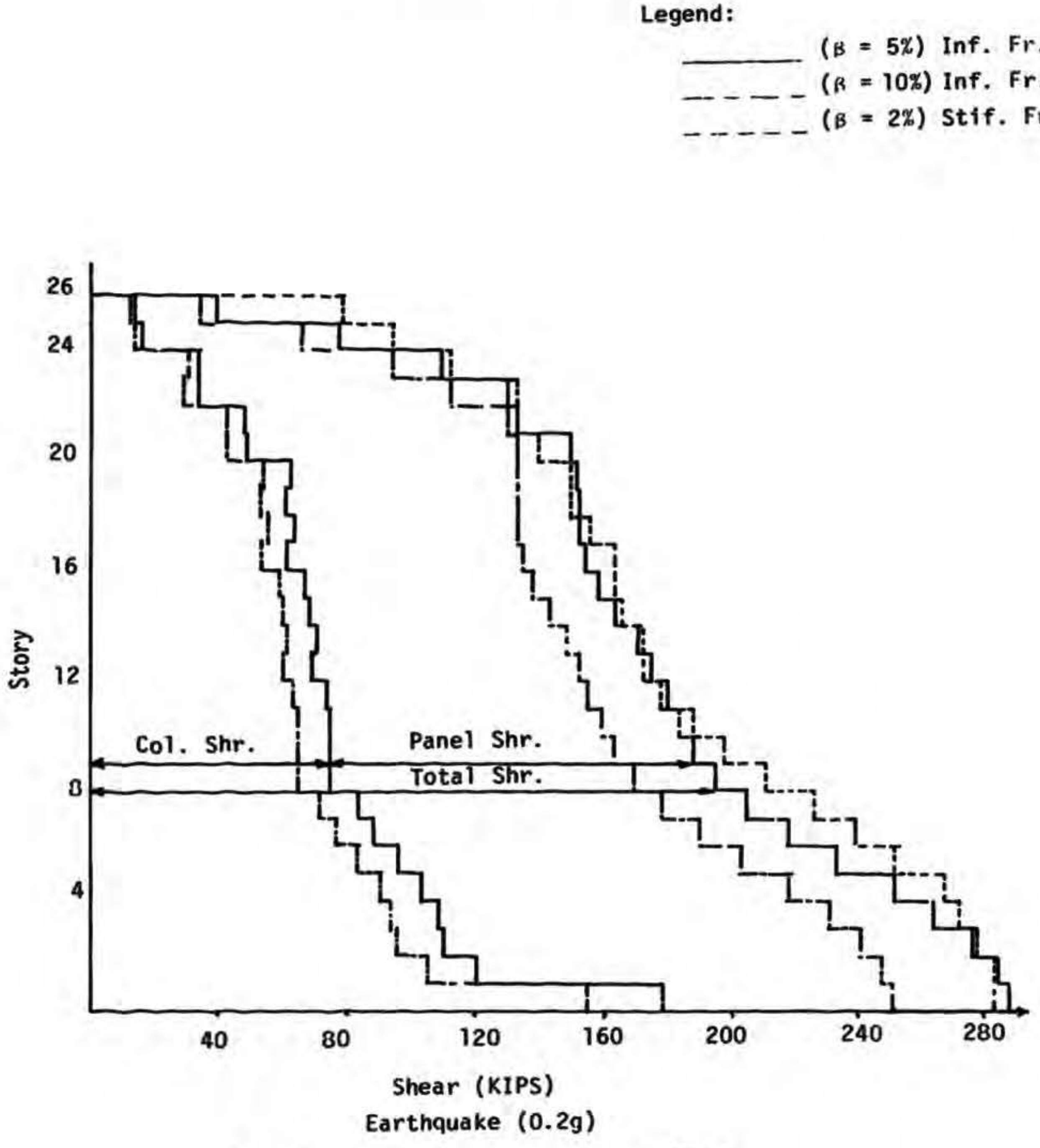


Fig. 9 - Column and Panel Shear (RMS) For Infilled Frame β = 5% & β = 10% and Column Shear (RMS) for Stiff. Frame β = 2%

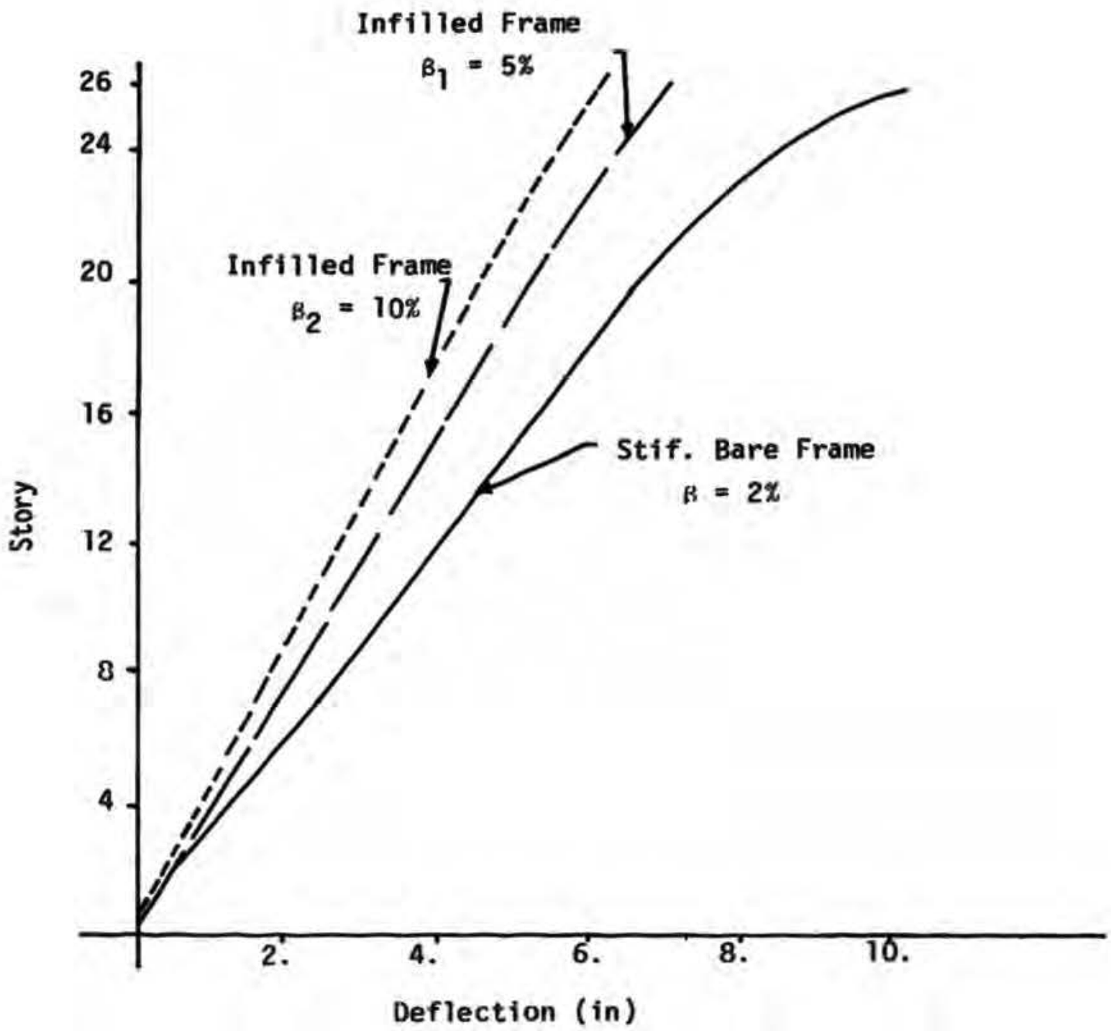
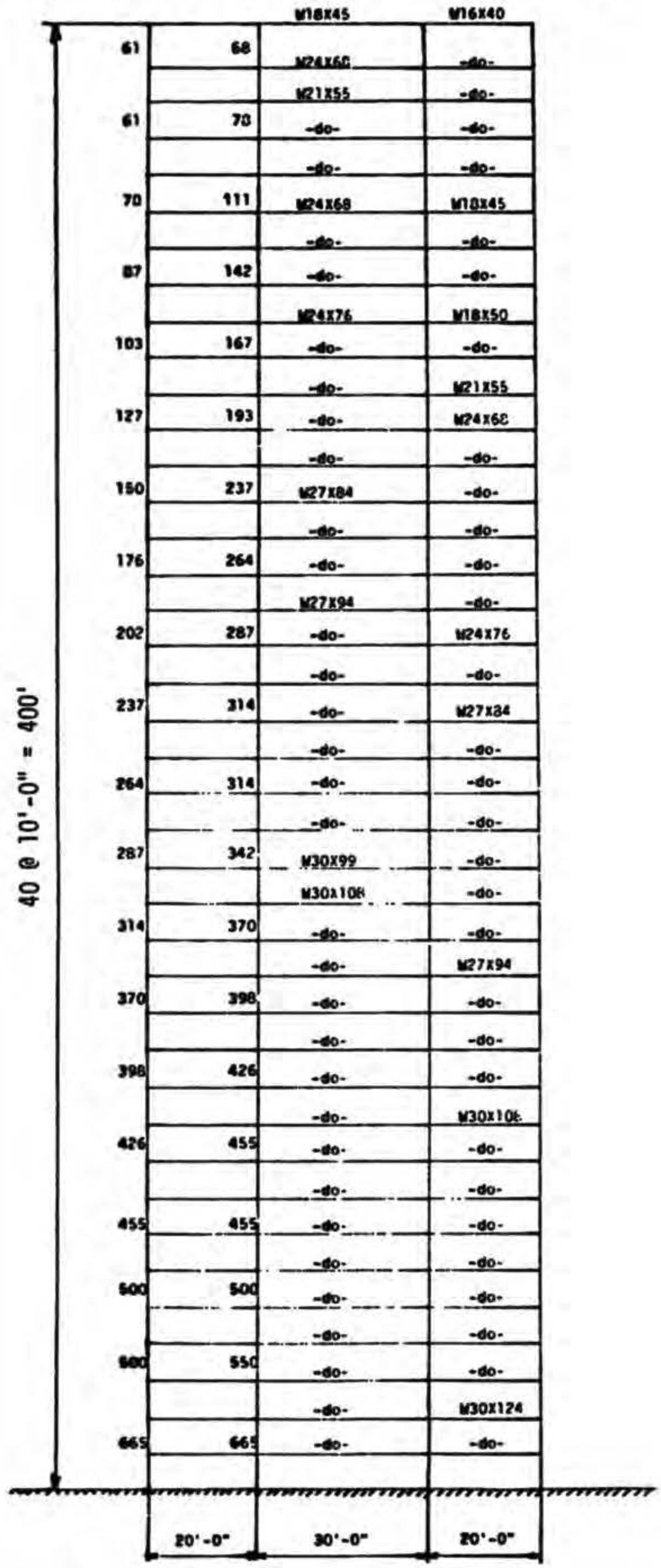


Fig. 10 - Earthquake Deflection (RMS) 26 Story Infilled Frame (β_1 = 5%, β_2 = 10%) and Stiffened Frame β = 2%

Fig. 11 - Infilled Frame -
Steel Weight = 342 tons



Frame is symmetrical
Columns are W14's

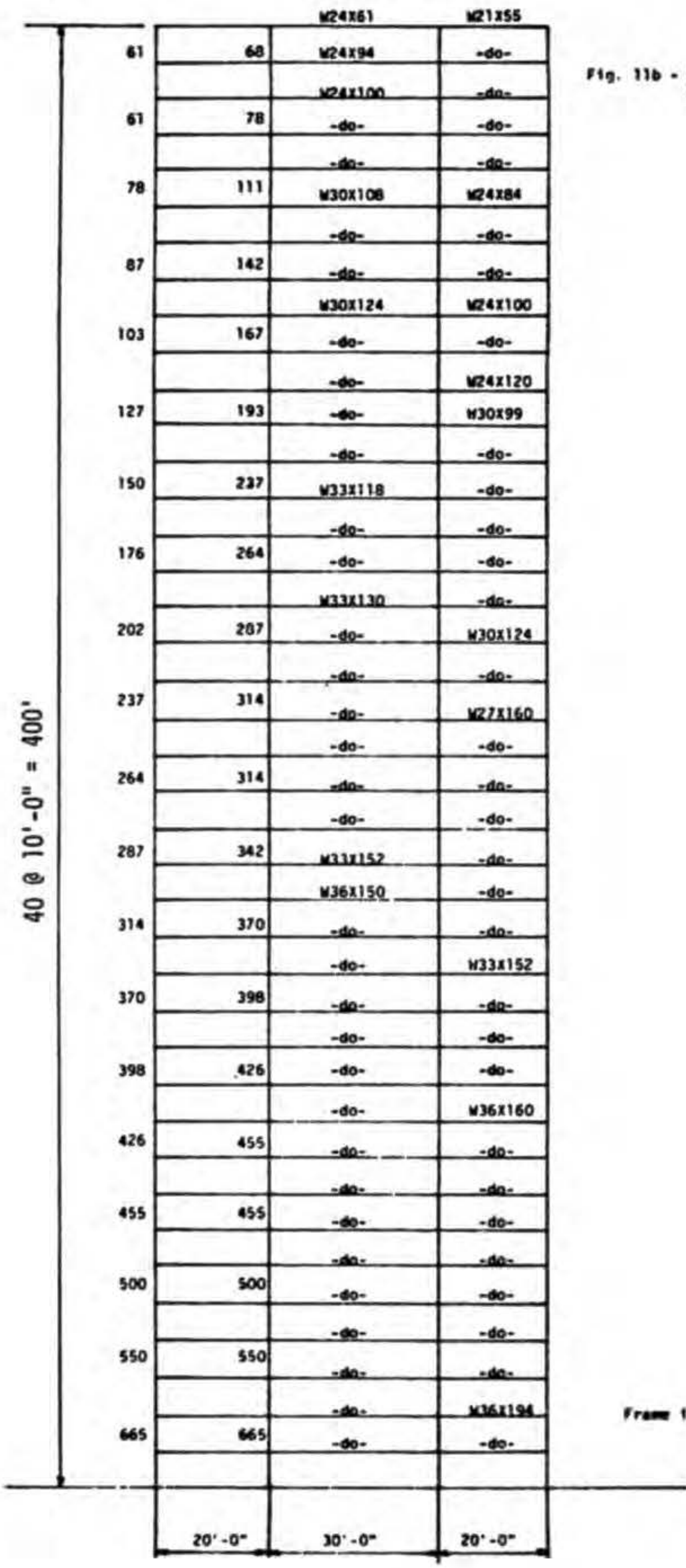


Fig. 11b - Stiffened Frame - Steel Weight = 372 tons

Frame is symmetrical, columns are W14's

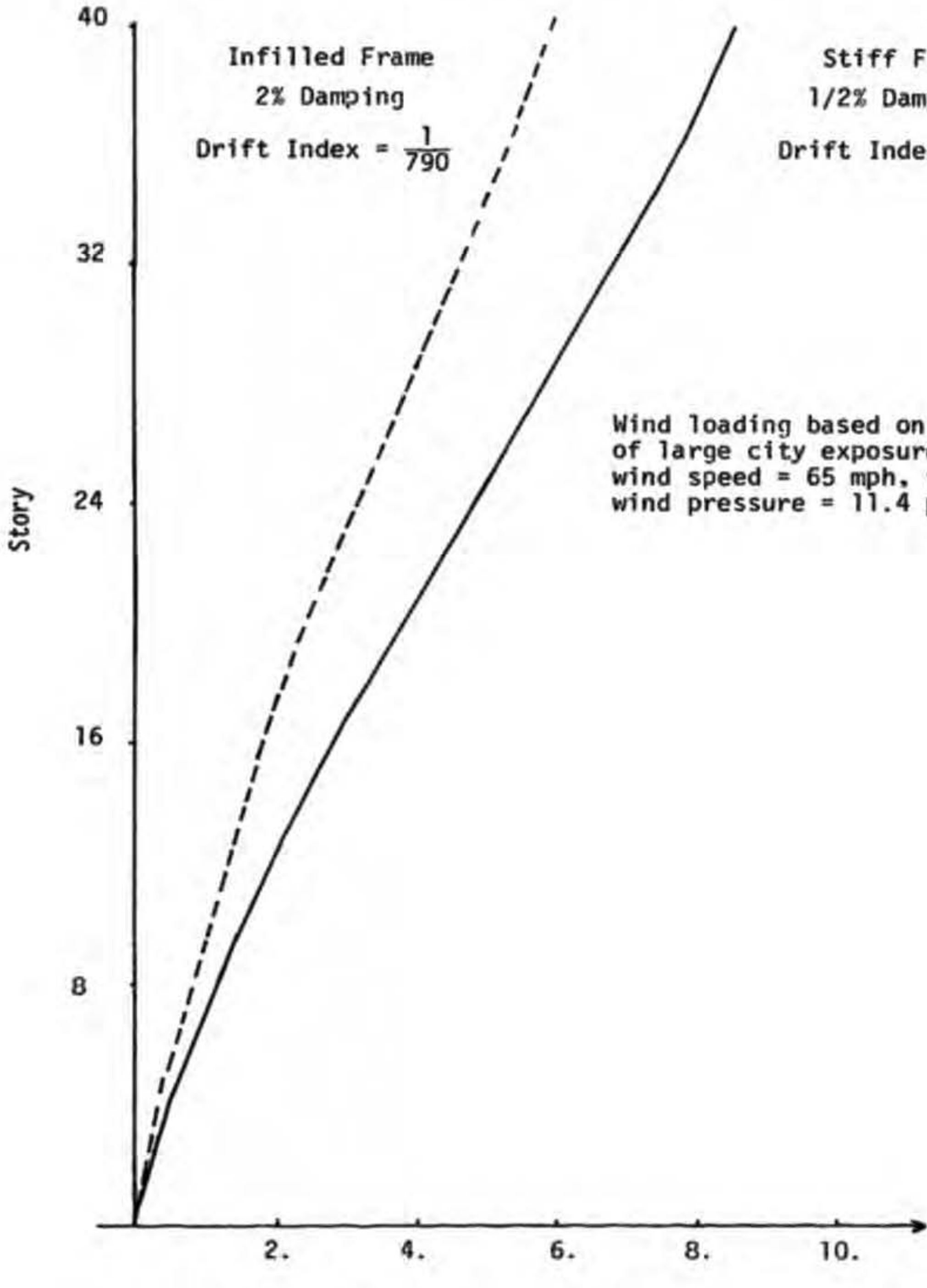


Fig. 12 - Wind Load Deflections - 40 Story Frame

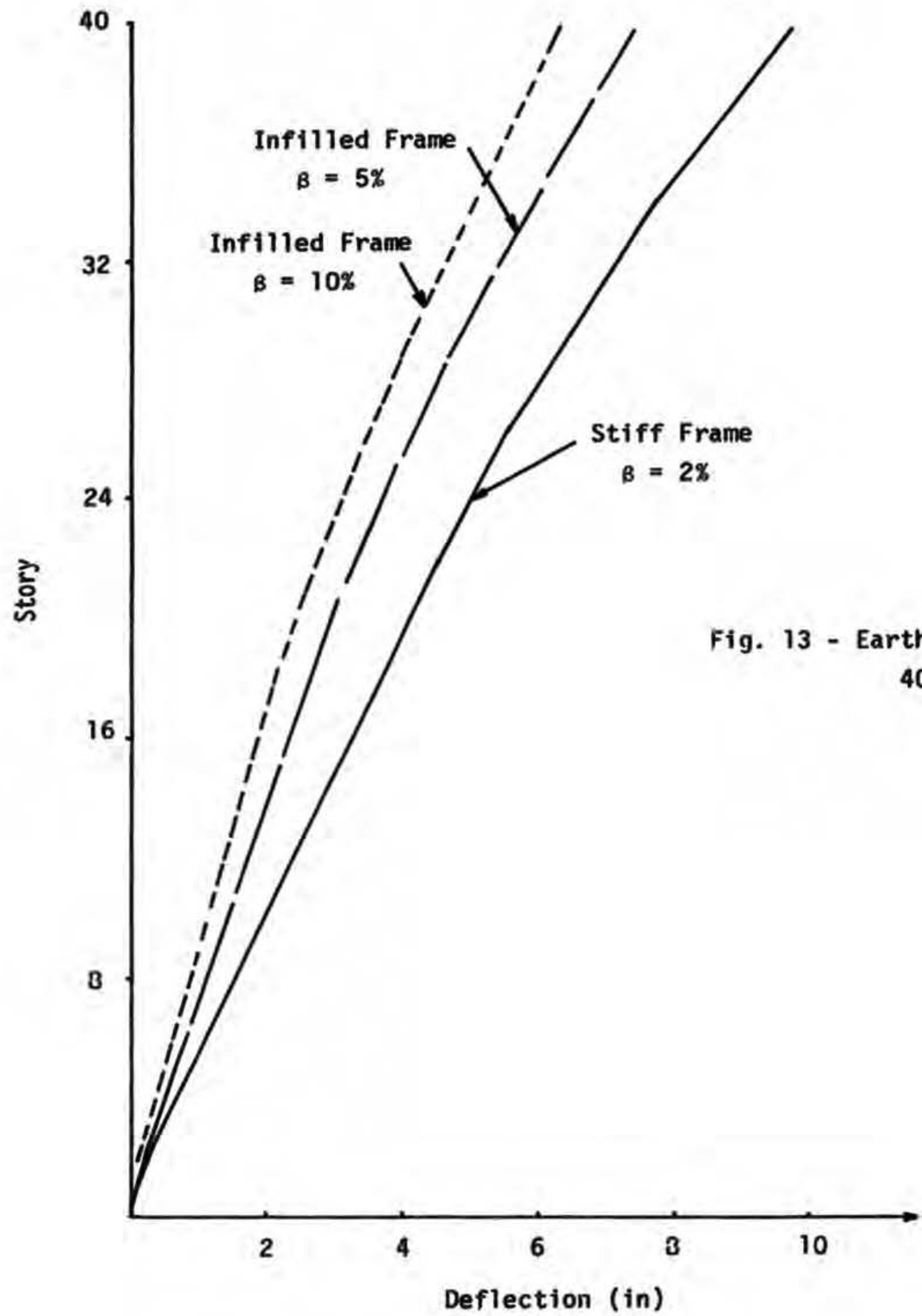


Fig. 13 - Earthquake Defl. (RMS)
40 Story Frame