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Drift Control with Light Gauge Steel Infill Panels

Craig J. Miller

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by

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Introduction

In the design of a modern multistory structure, the contribution of cladding and interior partitions to the strength and stiffness of the structure is generally not considered, although the effect of such non-structural elements sometimes influences the choice of a suitable deflection index. Until recently, the methods required to analyze multi-story frames including cladding and partitions as structural elements have not been available. Many practicing engineers feel that the strength and stiffness of walls as structural elements is not reliable enough for use in analysis. The likelihood that partitions will be removed in the future acts as another deterrent to their use as an integral part of the structural system.

There are important reasons for including the strength and stiffness of cladding elements in the analysis of a multistory structure. Most importantly, the supposed non-structural members do have a significant effect on the behavior of a structure. Studies of the response of tall buildings under load support this statement. Almost invariably, measured deflections are smaller than computed deflections. The behavior of structures subjected to earthquake loading demonstrates the role played by cladding elements. Another reason is the possibility of obtaining a lower cost structure. Neglecting the contribution of infills leads to a more expensive frame than

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

necessary.

In the past decade, two developments have made possible the analyses required to include the effect of cladding on the response of a structure. The first is the emergence of matrix and finite element methods of analysis. The advent of matrix methods provided the theoretical basis for analyzing structures with large numbers of unknowns. The finite element method allows treatment of problems in continuum mechanics as an assemblage of discrete elements. The discrete element representation is analyzed by matrix methods. The second development is the emergence of the digital computer.

The objective of the work reported here was to study the use of light gauge steel diaphragms as infill elements to control drift of multistory frames. Single story, single bay frames of different stiffnesses and infills of different thicknesses are used to establish the suitability of the panels for reducing drift. The requirements which the connections between frame and panels must meet are determined and details proposed. An "exact" model of the light gauge infill is developed for use in studying suitability of the infill.

The behavior of a multistory frame with infill panels is investigated using panels of 12, 16 and 20 gauge. The efficiency and the possibility of buckling of the different panels is discussed.

The research reported here deals only with the structural behavior of an infilled frame at service loads. The analysis is based on linear, elastic behavior of all components. No work is done to develop means to predict the ultimate load capacity of an infilled frame, and no statement is possible regarding the effect of infilling on the mode of failure or on the level of the failure load.

The proposed construction utilizes the in-plane shear stiffness of light gauge steel diaphragms to assist in controlling the drift of multistory frames. There is very little reported work on this problem in the literature.

Reference 1 is one example of the analysis of a multistory frame utilizing corrugated sheeting as a shear resisting element.

The problem of determining the shear stiffness of light gauge steel diaphragms has been studied extensively, in particular by researchers at Cornell University and the University of Manchester. A complete bibliography of this research is included in the work of Ammar (2). The approach to the determination of the shear stiffness used in the present work is to make use of matrix finite element methods of analysis, with the properties of the elements found by experiment. The development of this approach was done by Ammar and is reported in Reference 2.

In order to do the finite element analyses required, a computer program to permit analysis of multistory frames including infills and rigid or flexible floor systems was developed. The heart of the program is a wave front equation solution routine programmed by B. Irons (3). It is anticipated that the program will be made available to designers by the American Iron and Steel Institute, which underwrote its development.

Proposed Construction

There are three basic structural requirements the panels must satisfy to be useful to control drift. The first is high resistance to in-plane shear loads. The obvious choice is a panel with a continuous plane of material in the plane of the loading such as a cellular profile deck which can be treated as a flat sheet with the cells serving as stiffeners. On practical grounds, however, the cellular deck is not a good choice for infills, since it is more expensive to manufacture than open deck because two pieces of material must be joined and more expensive to ship because it cannot be nested.

An open profile has neither of these disadvantages, but for the open section to resist shear loadings effectively, distortion of the profile at the ends of the diaphragm must be prevented, by firmly fastening the panel to the frame. The present research deals only with panels of open, trapezoidal profile. To fully utilize the panel stiffness, it is assumed fastened to the supporting members at every flat.

The second structural requirement is ability to carry transverse loads. Exterior wall panels must transmit wind load to the frame. An interior panel must resist 10 to 20 psf. The panels studied here are of minimum 20 gauge thickness and 1-1/2" depth. For the ten to twelve foot spans used here, 20 psf capacity is reasonable.

The third requirement is possession of sufficient buckling resistance. Buckling can occur due to two types of loading. The first is uniform shear loading. Resistance to shear buckling must be provided by the sheet. Buckling can also occur due to direct in-plane loads from the girders bounding the infill. Using suitable connection details, in-plane load transferred from girders to panels is minimized, eliminating this type of buckling. Gravity loads are transferred from the girders to the columns and thence to the foundations, rather than through the infills to the foundation. Excessive stress and deformation of the panel to frame connections caused by deflection of the girders is also avoided with such details.

The connection details chosen to connect the panel to the frame must transmit lateral loads from the frame member to the panel. The type of construction envisioned to accomplish this is shown in Fig. 1a. The frame member is connected to the infill by a light gauge steel channel fastened to the frame member either continuously or at closely spaced intervals. The channel is sized so the trapezoidal panels can be slipped between the flanges continuously. In the next section, a less exact model of the panel connected to the frame only at the corners is described. Fig. 3a shows the idealization of an infilled frame. The basic approach to the idealization will follow that taken by Ammar (2). The connectors are modelled as linear, elastic springs whose spring constants are obtained from tests. The design load for the connections is about 65% of the ultimate load, P_U. Connection tests by Ammar (2) indicate that welded seam connections behave linearly to about $.55P_U$ and at $.65P_U$, the stiffness is 80% of the initial. The weld tested is similar to that shown in Fig. 2a. Only a small number of the connections will reach $.65P_U$ at working loads, thus many will be at loads in the linear range. Commercial tests of large, welded diaphragms at Cornell indicate linear behavior of the system to at least 60% of the ultimate load.

Both the marginal member and the frame member are idealized by linear, elastic beam elements derived from cubic displacement functions. This element has three degrees of freedom at each end; horizontal displacement, vertical displacement and rotation. The sheets used to form the infill panel are modelled as an assemblage of orthotropic plane stress finite elements. The plane stress element chosen is rectangular with a horizontal and vertical degree of freedom at each corner. The derivation of the element stiffness matrix is given in complete detail by Maghsood (4).

Fig. 3 shows the degrees of freedom assumed at the edge of the panel. The marginal channel and the frame member deflect the same amount vertically and rotate the same amount. The vertical displacements can have different values. The difference represents the deformation of the flexible link between frame and marginal member. Fig. 3b also indicates that the displacements of the sheet at its junction with the edge member need not be the same as those of the edge member. The difference is the deformation of the of the channel. The panels are welded to the toes of the flanges. Fig. 1b shows the panel connected to the channel member on both sides, forming a nearly continuous connection. The continuous connection prevents distortion of the deck profile at the ends. A substantial reduction in stiffness occurs if the panel can distort.

All connections dealt with in this research are welded. Use of welded connections results in a more rigid diaphragm than is possible with mechanical fasteners. The behavior of the structure will be more nearly linear with welded connections than it would be if other types of connectors were used. The connections around the perimeter of the panel are assumed to be fillet, plug or puddle welds. Since the panels are used in a vertical position, welding can be done from either side of the sheet, in contrast to floor diaphragms, where welding must be done from above. Because of this, it is possible to use the welds shown in Fig. 2a for the seam connections. This type of weld is stronger and stiffer than the type shown in Fig. 2b, which is a standard floor diaphragm seam weld.

The panel to frame connection minimizes transfer of vertical load from frame members to the panel. Figs. 1b and 1c show possible details to accomplish this. In Fig. 1b, load transfer is reduced by including in the marginal member an inclined portion which flexes as the girder is loaded. In Fig. 1c, transfer is prevented by one channel sliding within another. These details are only suggestions. Experimental work is required to develop the best details.

Description of the "Exact" Model of the Panel

The model of the panel described in this section is referred to as fully connected because the marginal member is connected to the frame connection. Fig. 3c shows the degrees of freedom at a sheet to sheet connection. For the analysis, the continuous connectors which join sheet to marginal member and marginal member to frame are lumped at the nodes of the finite elements.

Four independent material properties are necessary to specify the behavior of an orthotropic plate subject to inplane loads. There are five material constants for an orthotropic material whose principal axes coincide with the axes of orthotropy; E_x, E_y, ν_{yx}, ν_{xy}, and G_{xy}. Only four are independent because $E_x\nu_{yx} = E_y\nu_{xy}$. Referring to Fig. 4 for the directions of the coordinate axes and the direction of the corrugations, the elastic modulus of an equivalent flat sheet parallel to the corrugations is $E_y = E(\ell/s)$ where E is the elastic modulus of the base material, ℓ is the developed width and s the flat width of the sheet. The elastic modulus perpendicular to the corrugations, E_x, is found experimentally. The experimental value can be confirmed roughly by an energy analysis of one corrugation. For small deflections, E_x is on the order of 500 ksi (2). This is because it takes little load to unfold the corrugations. For shear loads, the calculated displacement of a diaphragm is not senstive to the value of E_x. The value ν_{yx} is equal to Poissons ratio for the base material. The value of ν_{xy} is found from the other constants.

The value of the shear modulus, G_{xy}, is equal to $G(s/\ell)$ where G is the shear modulus of the base material. The shear modulus of the equivalent flat plate is dependent on the conditions of restraint at the ends of the panel. If warping of the ends of the sheet is prevented, the above expression for the shear modulus is nearly true. It is not precisely true because the webs of the trapezoidal section are not restrained. If the ends of the panel can distort, the shear stiffness of the diaphragm is greatly reduced. The work of Ammar (2) indicates that G_{xy} for panels fastened at every second or third valley is approximately an order of magnitude lower than results from $G(s/\ell)$.

Description of the Approximate Model of the Panel

Because of the large number of degrees of freedom involved in the fully connected model of an infilled frame, an approximate model, called the corner only model, is necessary for multistory analysis to be practical. The degrees of freedom in the interior of the panel could be eliminated using static condensation, but the problem would still involve far more unknowns than the bare frame. The ideal situation would be to have a substitute panel which would closely approximate the behavior of the actual panel although connected to the frame only at the corners. The analysis of the frame could then be done with no increase in size compared to the analysis of the bare frame. With such an approximate model, the derivation of the stiffness matrix for the infill needs to be done only once for each type of infill.

As a beginning in the search for such a model, the panel idealization described in the last section is used, except that the marginal member is separated from the frame everywhere except at the corners, as shown in Fig. 5. An analysis of this model yields horizontal corner displacements about double the correct ones. Examination of the displacements makes it clear that the cause of the difference is folding of the profile on the windward side and opening up of the profile on the leeward side. The relative displacements in the horizontal direction between the corners is the same for the fully connected and corner only models, although the pattern of the displacements is entirely different. Because of the flexible edge member in the corner only model, transfer of load from the frame to the diaphragm cannot take place in the proper manner. In the fully connected model, the diaphragm is loaded with uniform shear loading, causing uniform compression of the panel edges and a uniform distribution of displacement from corner to corner. The light member in the corner only model is not stiff enough axially to force this behavior. Most of the load is transferred to the sheet near the point of application of the load. The panel is highly compressed near the load, causing the profile to fold up.

The above discussion suggests the possibility of obtaining better agreement between the two models by providing a greater area to the marginal members in the corner only model. Sufficiently stiff edge members will cause uniform transfer of shear from the perimeter member to the sheet. The analysis was rerun with the area of the marginal member set to its area plus the area of the frame member. The area of the frame members was set to zero. The results of this analysis showed excellent agreement with the results of the fully connected model analysis. If only the area of the horizontal members is modified, the two models agree within three percent. If the areas of all members are modified, the results agree within one percent. These results are presented in the next section.

Results of Single Story, Single Bay Analyses

The investigations described in this section are done using the structures in Fig. 5. These simulate an interior panel of a multistory, multibay frame. The frames are thirty feet wide and twelve feet high. The dimensions amd member sizes are intended to be representative of those found in a modern office structure between twenty and forty stories high. Two thicknesses of panel material are used in the analytical tests, 16 and 20 gauge. Load cases studied are lateral load applied as a concentrated load at the upper corner of the frame and gravity loads applied uniformly on the upper and lower girders with concentrated loads of 995.4 k at the upper corners to simulate load from the columns above. The uniform loads used are:

dead load: 1.5 kips/foot

live load: 2.25 kips/foot

The assumption is made that gravity loads are applied to the frame after the panels are installed.

The value of the spring constant for the seam connections is taken from the work of Ammar (2). His results indicate that the stiffness parallel to the seam is 500 kips/in. Perpendicular to the seam, diaphragm tests show little movement between the two sheets. For this reason, the spring constant for this direction is taken as 10000 kips/in. These values have been used for all sheet thicknesses, since Ammar's results indicate that at low load levels ($\leq$ 40% of ultimate) the stiffness is nearly independent of sheet thickness. The stiffnesses are equal to the secant modulus at 40% of the ultimate connection load.

The spring constants for the end and edge connectors are taken as 2000 kips/in. and 1000 kips/in. respectively. These values are based on results presented in Ref. 5. The influence of the value chosen for these spring constants was investigated and found to be relatively small.

Table 1 summarizes the results of nine analyses of the frames described above. All analyses were performed using the same value of horizontal load on the frame and using the fully connected model for the infills. The addition of the light gauge diaphragm substantially reduces the deflection. The column labelled "Horiz. Deflection" gives the values from the analyses. Table 1 is evidence that the rediction in drift is large enough to indicate that light gauge infills may be practical for drift control. The drift of the frame infilled with a 16 gauge panel is only 20% less than that for the frame with 20 gauge panel, although the 16 gauge panel contains 40% more material. This happens because the seam connection stiffness is the same in either case. Thus, it is apparently advantageous to use lighter panels, if buckling is ignored. The results of Table 1 were obtained using a relatively coarse mesh, so the stiffness of the panel is overestimated somewhat.

Table 1 also shows the distribution of horizontal load between the panels and the columns. These results are based on analyses using a coarse mesh, so column shears are underestimated and panel shears overestimated. The shear distributions indicated in Table 1 demonstrate that for the cases tested, the panels and the frame each resist a substantial portion of the load. Analyses of the analytical results indicates that the forces in the connectors are within the capacity of the connections. Reference 6 contains the detailed results.

The assumption has been made that little or no gravity load is transferred from the frame member to the trapezoidal panels. To check its validity, the medium frame is analyzed with three different spring constants for the flexible portion of the marginal member. Table 2 summarizes these calculations. The stiffness of 25.8 kips/in. in the top line is calculated assuming the inclined portion of the channel member shown in Fig. 1b to be a cantilever with a concentrated load at the end. For that stiffness, the vertical load transferred from frame to diaphragm is substantial. The value in the table is the maximum that occurs. With the connection stiffness reduced to one tenth of the value above, the load transferred is substantially reduced; with one hundredth of the stiffness above, load transfer is insignificant. The distribution of horizontal loads transferred to the panel is changed by the presence of the gravity load, if the marginal member inclined portion is too stiff. The distribution of horizontal gravity shear is similar to that in a beam; maximum at the ends and zero in the middle. Superimposing that distribution on the uniform shear resulting from lateral loading increases values of horizontal force on one side and decreases them on the other side. Table 2 shows that reducing the stiffness of the spring between frame and marginal member causes the value of the maximum horizontal force in the connection to approach that of the lateral load case. These results demonstrate the importance of making the stiffness as small as possible. This is the main advantage of the detail shown in Fig. 1c. The spring constant is practically zero, since one channel is free to slide within the other.

One of the main objectives of the analyses is to assess the accuracy of the corner only model over a wide range of parameters. The results of analyses of the six panel-frame combinations under lateral load for the corner only model and the fully connected model are shown in Table 3. The answers compare favorably. The largest errors are approximately four percent in the rotations. For the horizontal and vertical displacements, the discrepancies are generally less than 1-1/2%. The corner only model gives an acceptably accurate prediction of the behavior of the panel-frame combination subjected to lateral load.

Analysis of a 26 Story Frame with Infill Panels

To demonstrate the ability of light gauge steel infill panels to control drift effectively in a multistory frame, the 26 story frame shown in Fig. 7 is analyzed in detail. The frame was designed by a research group at Lehigh University directed by Prof. J. Hartley Daniels for use in American Iron and Steel Institute Project 174; "Effective Column Length and Frame Stability". The loads and dimensions used in the analyses are given in Fig. 7.

The stiffnesses of the infill panels used in the multistory frame analysis are derived using the corner only model with the edge member sizes taken as W24 x 84 girders and W14 x 314 columns. The stiffness matrix derived on this basis is then used as the stiffness matrix for all panels in the frame. All degrees of freedom except the three at each corner of the panel are eliminated by static condensation. In this way, the number of degrees of freedom in the infilled frame remains the same as in the bare frame.

In the analysis, the areas input for the frame members are adjusted so that the total area of frame member and marginal channel is correct. In developing the panel stiffness, the marginal channel area is greatly increased, for reasons discussed in the section on the approximate model. Results of number of cases (6) indicate that no appreciable difference in deflection occurs regardless of assumed marginal channel area, as long as the total member areas are correct.

To study frame behavior with realistic infills, the 26 story frame is analyzed using 12, 16, and 20 gauge 1-1/2" deep infills full height in the middle bay. The displacements of the structure plotted versus height are shown in Fig. 8. The maximum deflection of the bare frame is about 10.1". With 20 gauge panels, the deflection is cut about 40%, to 5.9". The further reduction in deflection resulting from increased panel thickness is quite small compared to the amount of material added. For example, the change from 20 gauge to 12 gauge material increases the amount of material by 200% yet reduces the deflection of the frame only about 20%, from 5.9" to 4.7". On that basis, the thinner panel is more economical than a thicker panel. Because the seam connection stiffnesses are held constant for all thicknesses, increasing the thickness does not affect panel behavior proportionately. Similar behavior is evident in the single bay, single story results in Table 1 and in Ammar's work (2). If the seam connector stiffness were changed along with the sheet thickness, the added material would prove more effective.

The reductions in drift obtained with all infill thicknesses are substantial.

The deflection index for the bare frame is higher than most engineers consider acceptable, but with the addition of panels it is reduced to a reasonable level. Fig. 9 shows the distribution of lateral load for the structure with 16 gauge infills. The percentage of shear carried by the panels remains relatively constant throughout the height of the structure, except for the topmost and bottommost few stories. The variations occur because the column and girder stiffnesses do not change uniformly over the height of the building. In a combined frame-shear wall structure, the shear wall resists a larger share of the horizontal load at the bottom of the structure than at the top. Fig. 9 shows that this does not happen in this case of an infilled frame. In this case, the relative increase of stiffness for the infilling and the frame toward the bottom are about the same and the portion of load resisted by each remains the same.

With the shear loads on the panel known, buckling of the panels can be investigated. The buckling loads for the infills used in this analysis are calculated using the work of Easley and McFarland (7). The useful load on the panel is assumed to be the buckling load divided by 1.5. To determine if buckling is a problem, the calculated load on the panel is compared with the allowable load. For the 1-1/2" deep, 16 gauge panel the allowable buckling load is 71.4 kips. From Fig. 9. it can be seen that the second to fourth floor infills carry loads greater than 71.4 kips. The depth or thickness of these panels would have to be increased or stiffening members added to the panel to avoid buckling. Because the upper story panels are well below the allowable buckling load, it would be worthwhile to use 18 or 20 gauge panels in the upper stories. The results shows that buckling is not a problem for any of the 12 gauge panels, while the 20 gauge panels, buckling is likely from the first floor to the fifteenth, indicating that 1-1/2" deep, 20 gauge panels are not suitable for use in the lower portion of this structure.

Conclusions

The major conclusion of this investigation is that light gauge steel infill panels can be used to control drift. The drift reductions achieved by infilling a multistory frame are substantial enough to justify the extra design complexity. Practical sizes and spacings of connections provide sufficient resistance to the loads on them.

The approximate model developed to reduce the number of degrees of freedom involved does so without significant loss of accuracy. Combined with the fact that assumed edge member properties do not have a significant effect on displacement, such an approximate model makes possible use of the same panel stiffness matrix throughout a structure, if all the panels are the same. Because the cost of deriving the panel stiffness matrix will often be more than the cost of analyzing the frame, reducing the number of different panel matrices is an important aid in reducing the cost of analysis. A library of stiffness matrices for panels of different depths, thicknesses, configurations and dimensions can be compiled. The stiffnesses can be made available to designers to carry out analyses at little extra cost compared to the analysis of an unclad frame.

The analyses of the 26 story frame indicate that shear buckling can occur at the loads to be expected in multistory structures. The panels used must be chosen to have an adequate safety factor against shear buckling. If the safety factor is not adequate, the designer can increase the panel thickness or depth, change the configuration to obtain a higher moment of inertia, or add stiffening elements to increase the buckling load.

The discrete element approach is a practical and effective means of analyzing the type of structure investigated here. The important parameters, such as connection stiffness and spacing, shear modulus of the trapezoidal sheets and properties of the framing members can be varied easily. The only experimentally determined data required are the shear modulus of the trapezoidal sheets and the spring constants of the fasteners.

The approximate corner only model and the fully connected model give the same results if the connection preventing transfer of gravity loads is very flexible. The more flexible the connection is, the more nearly transfer of transverse load from frame to panel is prevented. For these reasons development work on practical ways of constructing the infills should concentrate on developing connections that are as flexible as possible in the direction perpendicular to framing members.

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TABLE 1

DEFLECTIONS AND SHEAR DISTRIBUTIONS OF SINGLE STORY, SINGLE BAY FRAMES

COMPUTED USING FULLY CONNECTED MODEL: LATERAL LOAD ONLY

Frame	Sheet Thickness	Horiz. Deflection (in.)	Shear in Columns (kips)	Shear in Panels (kips)
Light	No infill	1.41	100	--
	20 ga.	0.227	15.3	84.7
	16 ga.	0.180	11.6	88.4
Medium	No infill	0.556	100	--
	20 ga.	0.173	29.8	70.2
	16 ga.	0.140	23.2	76.8
Heavy	No infill	0.172	100	--
	20 ga.	0.101	57.0	43.0
	16 ga.	0.087	48.4	51.6

TABLE 2

FORCES ON EDGE CONNECTORS FOR 16 GAUGE PANEL, GRAVITY PLUS

LATERAL LOAD WITH DIFFERENT STIFFNESSES ASSUMED FOR

FRAME-MARGINAL MEMBER CONNECTIONS

Spring Constant (k/in)	Connector Force	
	Horizontal (kips)	Vertical (kips)
25.8	9.48	2.74
2.58	7.19	0.39
0.258	6.83	0.072

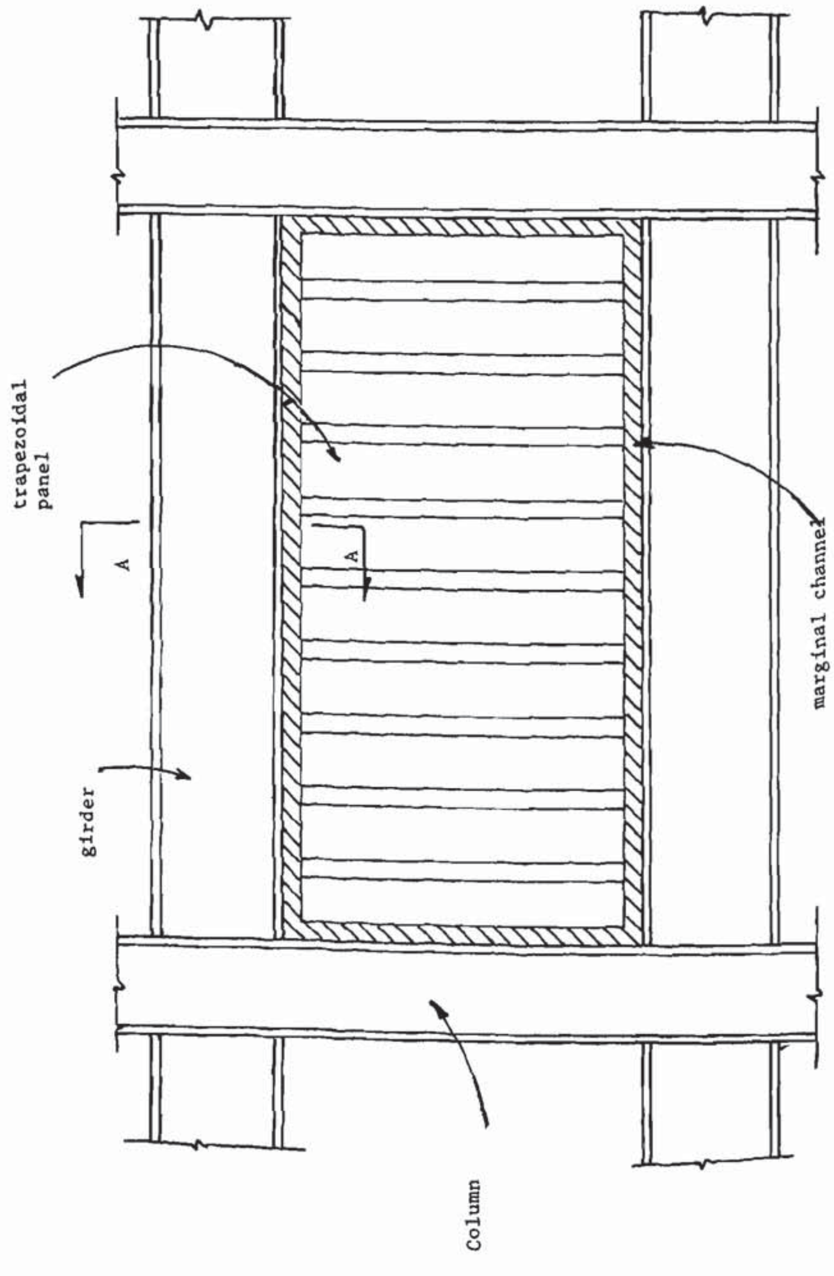
TABLE 3

COMPARISON OF RESULTS FOR EXACT MODEL AND CORNER MODEL FOR

VARIOUS FRAME PANEL COMBINATIONS (LATERAL LOAD ONLY)

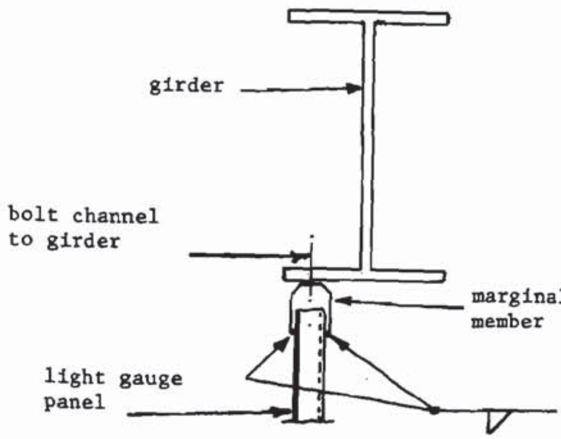
Frame	Displacement*	Exact Model	Cornel Only Model	% Error
Light 16 ga.	LCH	0.1801	0.1790	0.61
	LCV	0.00266	0.00262	1.50
	LCR	0.00100	0.00101	1.00
Light 20 ga.	LCH	0.2266	0.2268	0.18
	LCV	0.00267	0.00265	0.75
	LCR	0.00124	0.00127	2.42
Medium 16 ga.	LCH	0.1400	0.1400	--
	LCV	0.00103	0.00102	0.98
	LCR	0.00081	0.00082	1.23
Medium 20 ga.	LCH	0.1734	0.1743	0.52
	LCV	0.00103	0.00103	--
	LCR	0.00100	0.00102	2.00
Heavy 16 ga.	LCH	0.0873	0.0876	0.34
	LCV	0.00044	0.00044	--
	LCR	0.00051	0.00051	--
Heavy 20 ga.	LCH	0.1011	0.1018	0.69
	LCV	0.00044	0.00044	--
	LCR	0.00058	0.00059	1.72

*LCH = top left corner horizontal displacement (in.).
LCV = top left corner vertical displacement (in.).
LCH = top left corner rotation (radians).

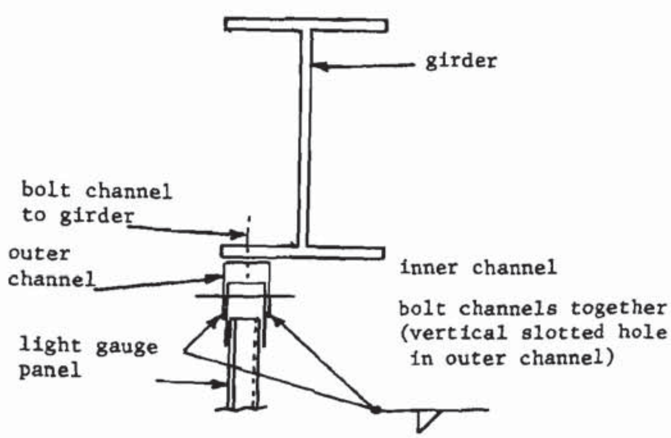


(a) Schematic of proposed construction

Fig. 1 - Proposed construction for infilled frames



(b) Section A-A

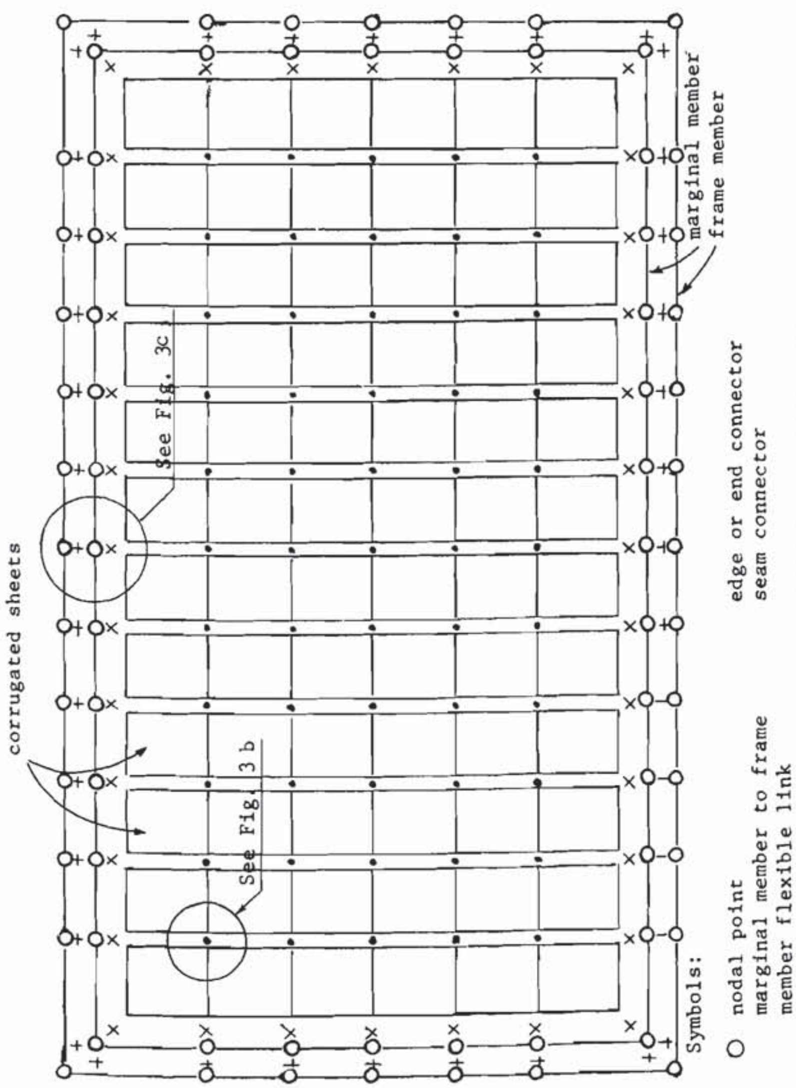


(c) Alternate Section A-A

Fig. 1 - Proposed construction for infilled frames (cont.)

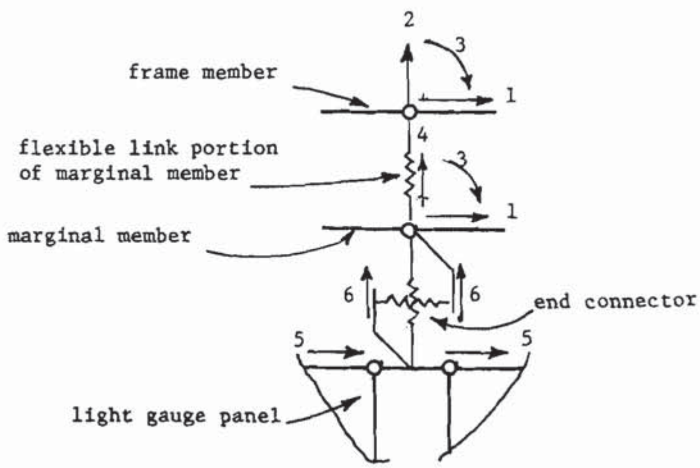
(a) Seam Fillet Weld (b) Seam Puddle Weld

Fig. 2 - Welded connections for light gauge diaphragms

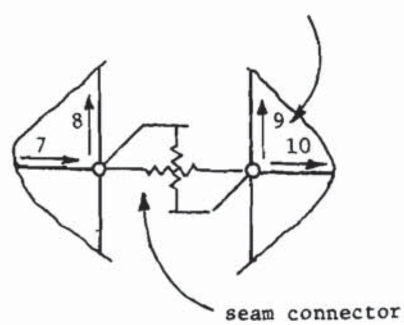


(a) panel idealization

Fig. 3 - Idealization of fully connected model



(b) Degrees of freedom at connection of frame and panel for fully connected model



(c) Degrees of freedom at seam connection for fully connected model

Fig. 3(cont) - Idealization for fully connected model of the infilled frame

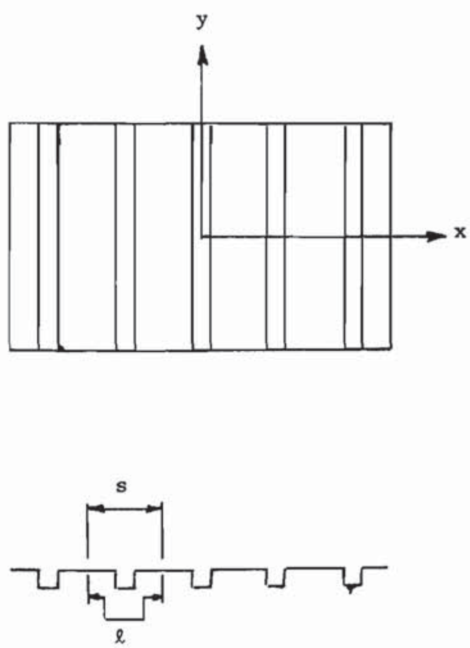


Fig. 4 - Coordinate Directions for sheet properties

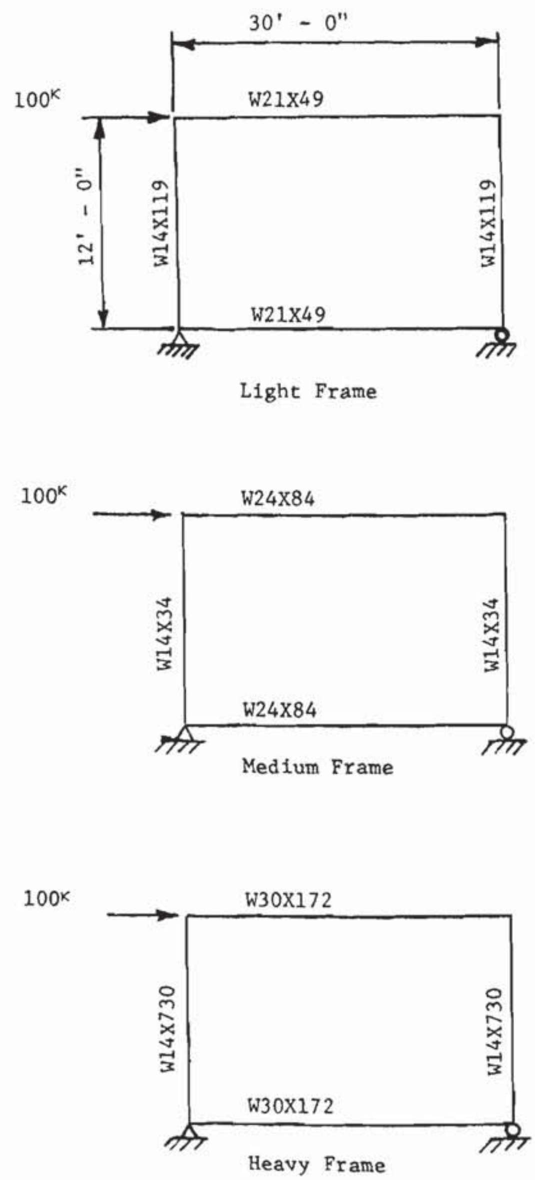


Fig. 5 - Frames used in Single Story, Single Bay Studies

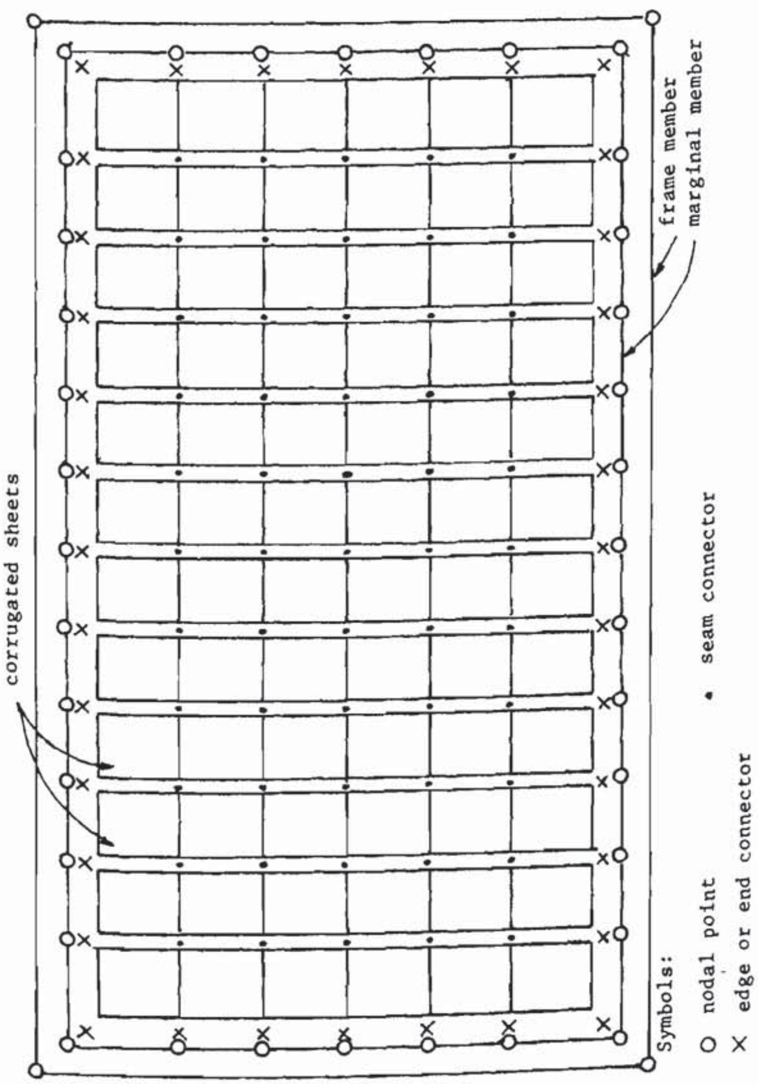


Fig. 6 - Idealization of corner only model

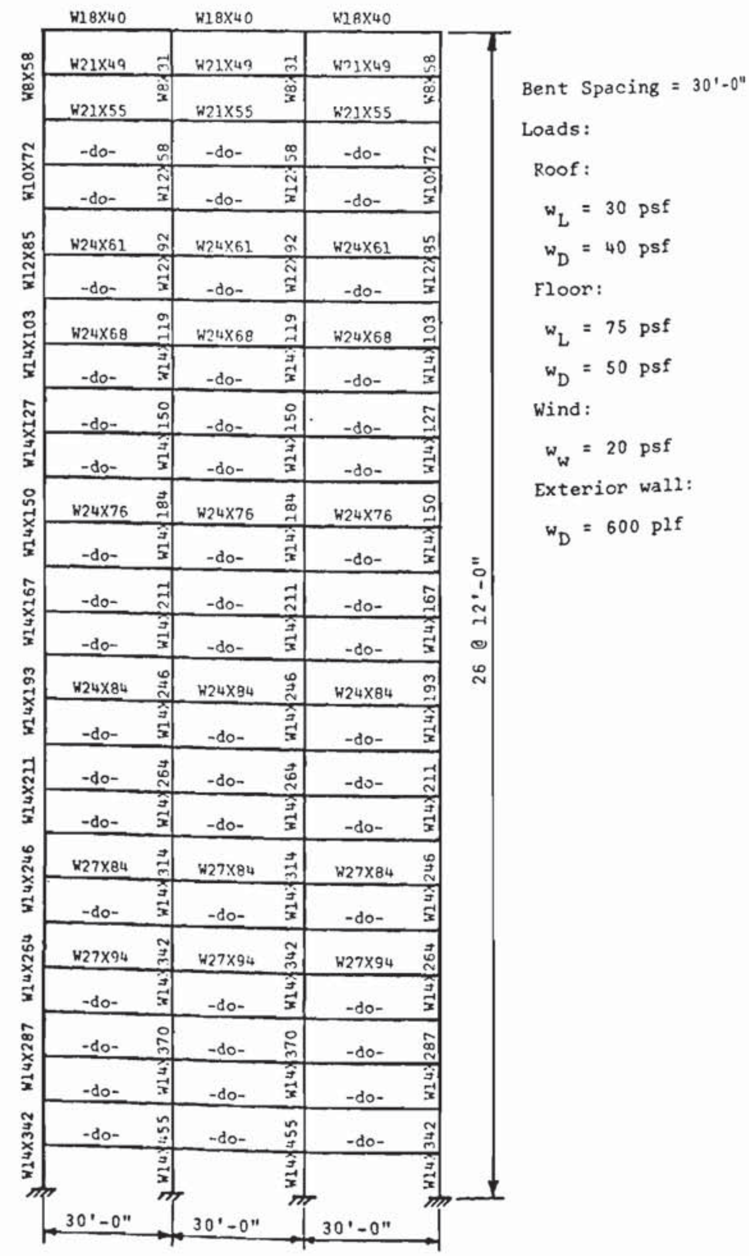


Fig. 7 - Twenty-six story frame

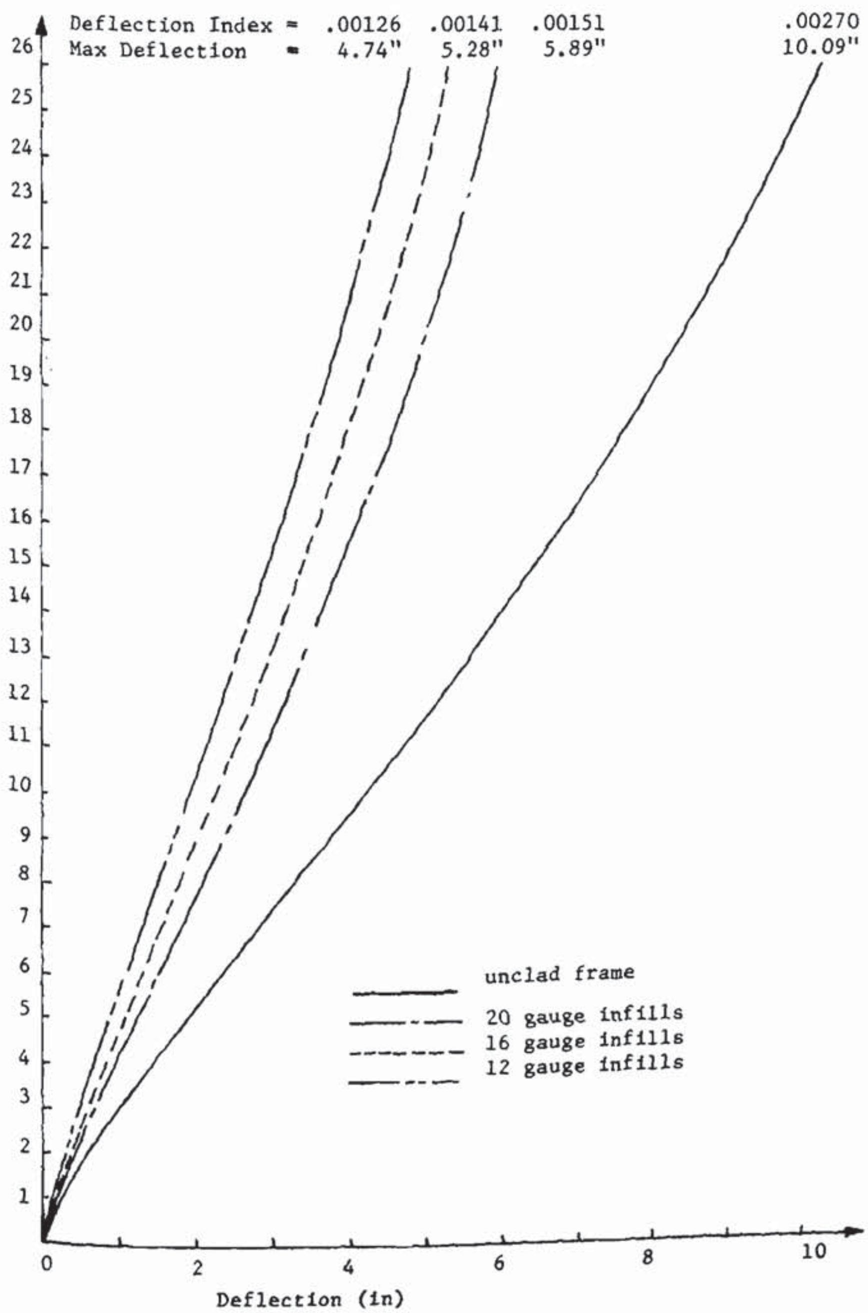


Fig. 8 - Deflected shapes for 26 story frames

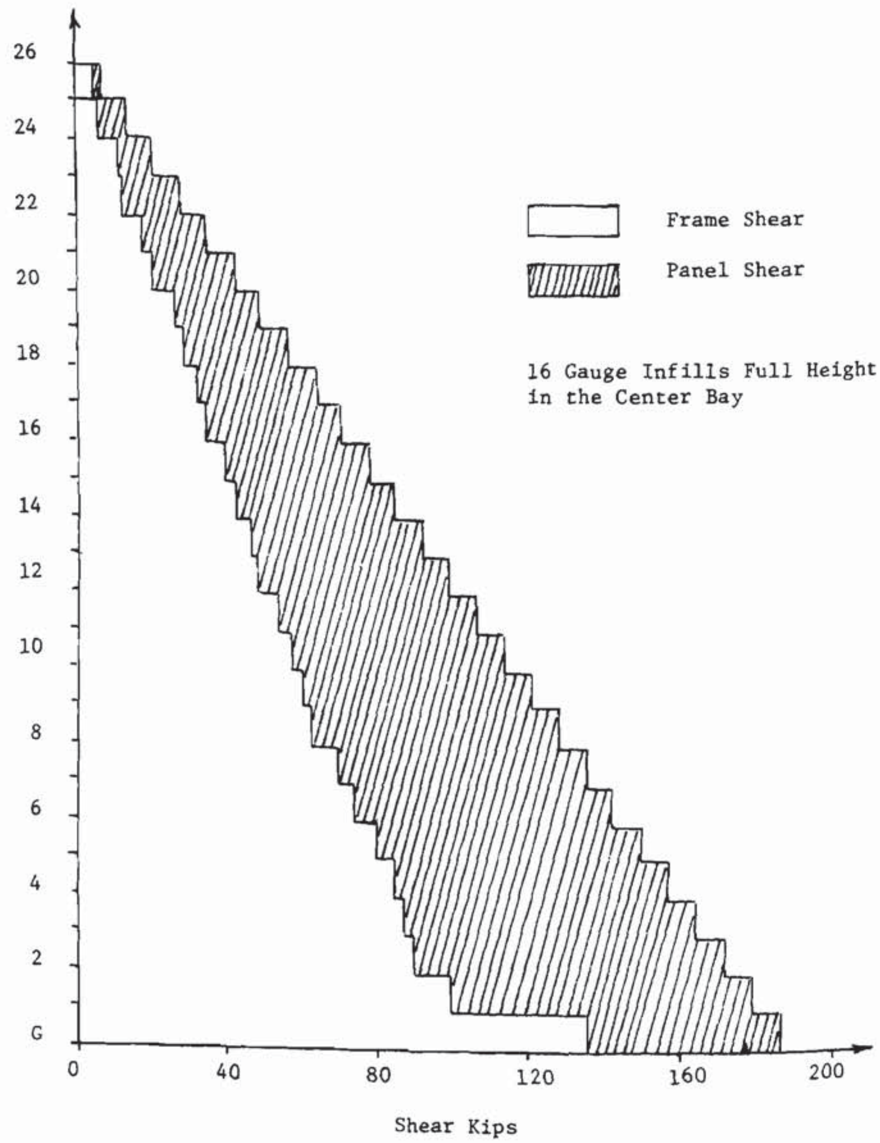


Fig. 9 - Distribution of Lateral Force Between Frame and Panels for 26 Story Building