SIMPLIFIED ANALYSIS OF HIGH-RISE BUILDINGS

by

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Abstract

The development and continued utilization of taller, more slender building structures has brought about an increasing need for accurate analysis techniques. These analysis techniques require the understanding of a variety of areas that range from the continuity of joints within the structure to the nature of the forces that act upon it.

Architects must consider the design of a building as a total system and cannot consider the structure as an addition that will be plugged in later by an engineer. This approach is especially essential when considering high-rise structures which require complex support systems. The safety and stability of these systems depends to a high degree on the continuity of joints which enable the building to act as a rigid, cantilevered tube.

This continuity results in forms that are highly statically indeterminate. These are structures in which the number of reactions and/or stresses exceeds the number of statical equations that are available for their determination. For such structures an indefinite number of combinations of unknowns will satisfy the laws of equilibrium, however, only one set of values will result in the distortions that are compatible with the continuity and special conditions that are unique to the structure.

Statically indeterminate structures may be analyzed by many different methods. Many approximate methods exist and often times may provide results as accurate as more time consuming exact methods.

While these approximate methods also serve as areas in estimating individual member sizes, they usually require the use of certain assumptions that may restrict their use in high-rise analysis.

Exact methods are often very lengthy and extensive; the method of moment distribution is a method that has been especially noted for its speed and accuracy. It is a method that will be extensively studied and applied throughout the research.

The magnitude of stresses and deformations in the structures are dependent on the imposed loads and many other effects. High-rise buildings, more than low buildings are affected by the instability of forces as well as secondary effects that range from changes in temperature, settlement of foundations, to the dynamic effect of lateral loads due to wind or seismic disturbances.

Another factor that must be considered in the analysis of high-rise structures is the effect that the structural geometry may have in the determination of stresses and probable deformations. The determination of the structure's shape may be a prime consideration when designing for a specific loading condition.

The objective of this research is to investigate the analysis methods of statically indeterminate frames and apply these techniques to high-rise structures. An analysis of loading conditions and structural geometry are factors that also must be considered in this investigation.

I intend to approach this objective with extensive research in engineering journals and publications along with a constant working relationship with my faculty advisor. I believe this research and program will greatly aid me in any graduate studies I may choose to endeavor in the future.

Acknowledgments

I would like to thank my parents, Larry and Gladys Garrison, for their continued support and encouragement; Professor Lee Lowry, for the help in "understanding" the computer; Head of the Building Construction Department, Carol Claycamp, for the financial help when it was needed the most; and my faculty advisor, Ron Kruhl, for the patience in answering my <u>many</u> questions and the friendship that we have developed.

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INTRODUCTION

A direct outgrowth of the urban environment, the high-rise building is continuing to be developed and utilized on a very large scale. While initially a response to the high density of population in early American cities, the development of new means of transportation and the subsequent growth of surburbia has enabled the high-rise to break free from it's inner-city dock and become a free standing skyscraper. Amidst the business centers of the world, the high-rise building has come to symbolize both corporate strength and growth to many businessmen and industries.

While the construction business has fluctuated considerably during the last few years, the market for high-rise office developments has proven to be a steady, if not growing one. One merely needs to visit Dallas or Houston, two cities which have benefitted to a great extent from the relocation of northeast and midwest industries, to get a feel for the tremendous amount of multi-story construction under progress. Also evident in those cities is the movement of business centers to suburbs such as the West Loop and SW Freeway areas of Houston and the Las Colinas development west of Dallas in Irving.

The future use of high-rise buildings is forecast by many to be a very bright one. While there may soon be a saturation in the office lease space market, the early success of high-rise condominiums promises to attract the attention of many developers and multi-story contractors. The particular growth and development of high-rise buildings in the Southwest and Sunbelt states can be contributed to many factors.

As mentioned earlier, the relocation of many established businesses and industries to the South has been brought about primarily by the freedom these states offer from tightly controlled union activity and rather burdensome corporate income taxes. With the shift of industry has come a subsequent shift in population and southern states have experienced a steady increase in net population over the last fifteen or twenty years.

Another factor that has contributed significantly to the development has been the absence of, or flexibility of, city zoning ordinances. For example, Houston, the first city in the nation to issue over $3 billion in building permits in a single year, is free of any zoning restrictions. With the extreme high cost of real estate in metropolitan areas, the opportunity for owners to take ultimate advantage of their purchased real estate is an extremely attractive one.

This steady growth and development has brought about an increasing need for accurate analysis techniques and it is the purpose of this research to examine some of the techniques that are currently being used and their application to high-rise buildings.

BACKGROUND

Paralleling the development of the high-rise has been the trend to build and design more slender structures. The safety and stability of these structures depends a great deal on the continuity or rigidity of the joints within the structure.[1] This continuity is necessary to bring about stability to the structure when under the influence of dynamic loading conditions such as wind or seismic activity. As a result of this continuity, structures are developed that are highly statically indeterminate.

Statical Indeterminancy

The key requirement for any structure is stability, the ability of any portion of the structure, when isolated as a free body, to satisfy the equations of static equilibrium. When the structure is such that the external reactions and internal stresses can be determined by the equations of equlibrium the structure is said to be statically determinate. However, when the shape of the structure makes it necessary to have a greater number reactions and for internal stresses the structure is called statically indeterminate. (See Figure 1).[2]

There are many advantages to the use of statically indeterminate structures when they are applied to high-rise buildings. Continuous construction lends itself toward the development of smaller moments and permits the use of smaller members. This can lead toward as considerable savings in the amount of material that is needed for stability. (See Figure 2) Another important consideration is the

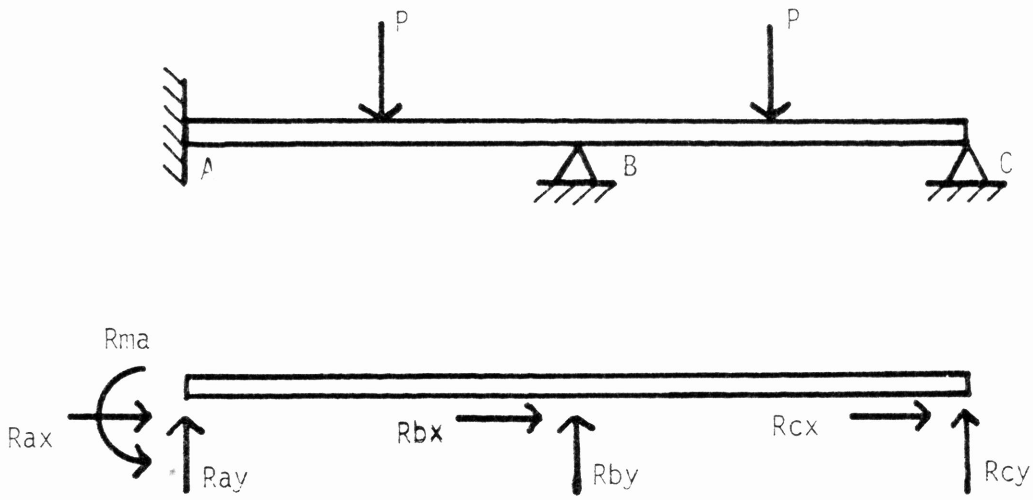


FIGURE 1. STATICALLY INDETERMINATE STRUCTURE

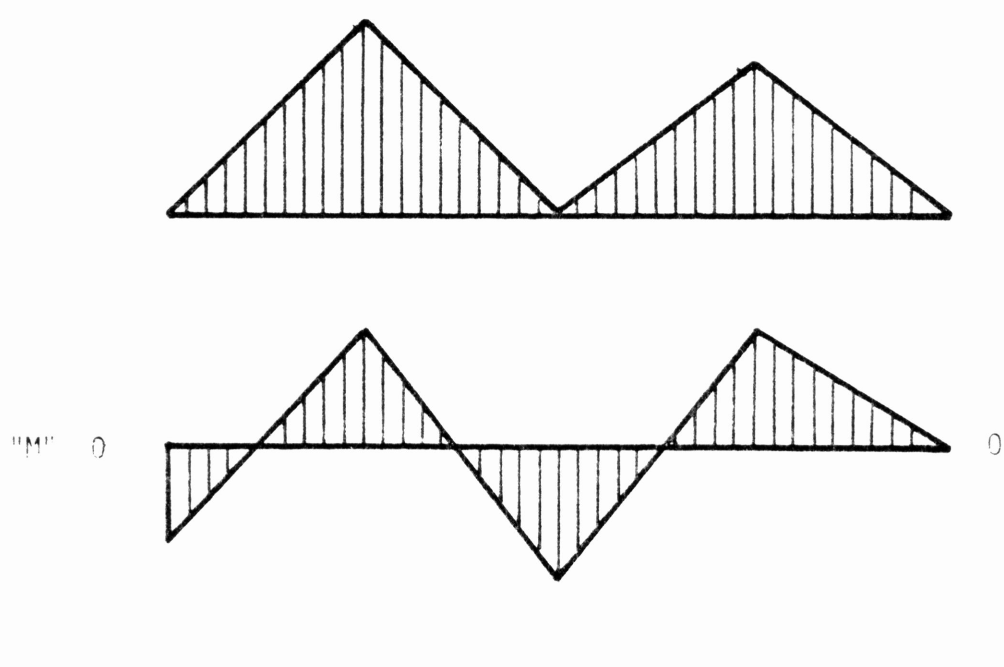


FIGURE 2. MOMENT COMPARISON--DETERMINATE vs. INDETERMINATE

difficulty of actually finding an ideal simply supported beam in a structure; the truth is that welded or riveted beam-to-column connection is not simply supported and does develop some degree of continuity. Also, members of a particular size can carry more loan if part of a continuous structure which may permit the use of fewer members and increased spacings.[3]

Considerable savings is also made when considering structures which are naturally continuous and indeterminate such a monolithic reinforced concrete structure a very popular method of construction today.

Another important advantage, and one of most importance when considering high-rise structures, is that a statically indeterminate structure is a more rigid structure. This is of particular importance when there are many moving loads. A similar statically determinate structure would require the use of diagonal cross-bracing which may hinder circulation.[4]

It is generally considered undesirable to use indeterminate structures where the foundation conditions are poor. The possible settlement of supports may cause additional moment, shears and reactions in the structure. Many analysis methods and techniques do make some consideration and compensation for possible foundation settlements.

Indeterminate structures also take considerable more time to design and analyze. It often requires the use of serveral approximations of member sizes before any analysis can be made and may take several repeated computations before final design can be determined.

There are several methods of analysis of indeterminate structures that are available to the designer.

Analysis Techniques

Moment Distribution

One method that is considerably noted for its speed and accuracy is the Moment Distribution method developed by Prof. Hardy Cross at the University of Illinois in the late 1920's. It has been considered by many to be one of the most significant developments in structural theory in the last century.[5]

The moment-distribution method follows a rather simple general procedure that can be grasped rather easily. It can only be applicable however if certain conditions are met. The first of which is that the members are of constant cross section. This is necessary because the procedure requires that each member carry a percentage of unbalanced moments based on it's relative stiffness. Another restriction on the use of this method is that there is no displacement of supports or joints.

The general procedure for moment distribution can be outlined as follows:

1. Assume all joints fixed and calculate fixed-end moments.
2. Select a joint, release the restraint, and balance all moments.
3. Temporarily fix joint again.
4. Select another joint and repeat same procedure (when all joints are done, one cycle is completed).

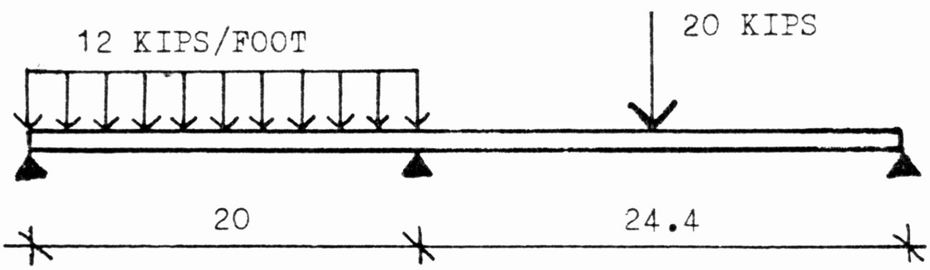
5. Carry-over one-half the induced moments to beam's opposite end and repeat the cycle.[6] (See Figure 3)

The number of cycles that the distribution is carried out is dependent on whatever degree of accuracy that one may desire or may be stopped after just several cycles to get a very good approximate answer. A common procedure is to continue cycles until the carryover moment is approximately 5% of the original fixed end moment.

The moment distribution method, though quite simple in procedure, can be a quite lengthy and extensive process when the structure may have many joints and members. With an increase in height, the relative stiffness (I/L) ratio of column to connecting beam increases and as a result, in a very tall building a comparatively light floor beam may be attached to a very heavy column. This requires the carrying out of an extended number of cycles to obtain the degree of acurracy necessary [7]

Another problem that is involved with the application of the moment distribution to tall buildings involves the conditions necessary for it's application. As stated earlier, one of the original criteria for the application was that all joints are fixed against any displacement. While this criteria may be, in theory, an applicable assumption, in practice it is known that all structures experience certain degree of sidesway. In low use buildings this effect may be neglible when determining member moments and shears, in high-rise structures a considerable amount of joint displacement may be present.

Considerable research has been published to present an answer to this problem. One of the most noted studies is one that was brought



I=CONSTANT=1

$K=\frac{I}{L}$ $DF=\frac{K}{\Sigma K}$

0	-1	-0.55	-0.45	-1	0	DF
	+400	-400	+61	-61		FEM
	-400	+186	+152	+61		BAL
	0	-214	+213	0		TOT
	+93	-200	+30	+76		C.O.
	-93	+93	+77	-76		BAL
	0	-107	+107	0		TOT
	+46.5	-46.5	-38	+38.5		C.O.
	-46.5	+46.5	+38	-38.5		BAL
	0	0	0	0		TOT
	+23.25	-23.25	-19	+19		C.O.
	-23.25	+23.25	+19	-19		BAL
	0	0	0	0		TOT
	0	-320	+320	0		Σ TOT

FIGURE 3. MOMENT DISTRIBUTION EXAMPLE

forth by Professor L. E. Grinter of Texas A&M in the early 1930's.[8] This method of handling joint translations was called by its developer "The Simplified Method of Successive Corrections."

The method was based on the assumption that the joint translation produced a deflection curve that was somewhat of a straight line. If this is known then an assumption can be made as to the increment of side deflection and additional moment may be applied to the frame. After balancing all additional moments, shears can then be calculated. The final moment can be calculated by multiplying calculated column moments in each story by the ratio of wind shears to calculated shears.

While Prof. Grinter's method has proved very successful in many rigid frame structures, it too requires a considerable amount of time and tedious work. It requires an initial distribution of moments due to wind loads and an additional distribution due to joint displacement.

The accuracy of moment distribution methods may depend to a great degree on the accuracy of assumptions regarding member sizes. It is often the practice to use member sizes obtained from vertical loading, but this may not be a correct assumption in cases where lateral loads are considerably larger than gravity loads.

Stiffness (Displacement) Method

Another method of analysis for statically indeterminant structures, and one that is only the subject of a general and brief discussion here, is the stiffness or displacement method of analysis. This method is primarily developed for computer applications and utilization and any extended discussion into the mathematics and programming of such a method is beyond the scope of this paper.

The stiffness method is based on the principle that the basic unknowns within the structural system are the system's displacements, whether they be joint translations or rotations. This method generates a system of linear equations which are best resolved by the use of matrix algebra. After rotation or displacement values have been determined, final reactions can be calculated.[9]

The stiffness method is the method used most extensively today for the design and analysis of high-rise buildings; but it too requires the initial assumption regarding member sizes to be considerably close for a high degree of accuracy. Another considerable drawback to the method is that it does require access and experience with computers to shorten the tedious work that is involved with matrix algebra.

Approximate Methods

In addition to the several exact methods for analyzing statically indeterminate structures, approximate methods are available and have many practical applications. Some of the more common ones are:

1. An exact analysis may be so tedious and cumbersome that time is not available for the necessary computations.
2. Approximate methods may often times produce results as accurate as many of the exact methods.
3. May serve as an aid in estimating sizes of members before a more exact method of analysis can be used.[10]

Portal Analysis

Among the most well-known methods of analysis is the portal method, which has probably been used more than any other method for determining stresses in building frames due to lateral loads. This simple method was presented in 1915 by Albert Smith.[11]

The application of the portal method requires the establishment of the following qualified assumptions:

1. Entire wind loads are assumed to be resisted by the building frames, with no assistance from floors, walls, and partitions with regards to stiffness. This is a conservative assumption because floors, walls and partitions do contribute an indeterminable amount to resistance.

2. Changes in lengths of girders and columns are assumed to be negligible.

3. Building height cannot exceed five times the least lateral dimension.

After this general criteria has been met, the basic procedure involved with portal method of analysis is a simple one. In order to reduce the degree of statical indeterminancy a frame may have, several assumptions are made in relation to the effect of lateral loads on the frame:

1. Columns and girders bend in such a manner that points of inflection occur at mid-depth or mid-spans of the member.

2. Horizontal shear is divided among the columns in the ratio of one part to exterior columns and two parts to interior columns. Another common distribution is to assume that each column takes a percentage of the wind shear based on the amount of total floor area it supports.[12]

The general procedures of the portal method is as follows:

1. Column shears, horizontal shears on each level are distributed between the columns by one of the earlier prescribed methods.

2. Column moments. Obtained by multiplying column shears by one-half the column height. This follows the original assumption of points of inflection at mid-height.

3. Girder moments. The sum of the girder moments at any joint in the frame is equal to the sum of the moments in the columns. Starting at top left hand side of the frame, moments are added or subtracted as might be needed.

4. Girder shears. Following the assumption that points of inflection occur at mid-span, girder shear are equal to the girder moment divided by one-half girder spans.

5. Column Axial Stresses. Column axial stress is equal to girder shears for exterior columns and is equal to the difference between the two girder shears for interior columns.[13] See Figure 4.

Once again it must be stressed that the application of the portal method is restricted to the satisfaction of the general criteria. If these conditions are not met, errors may occur of substantial proportions in the results.

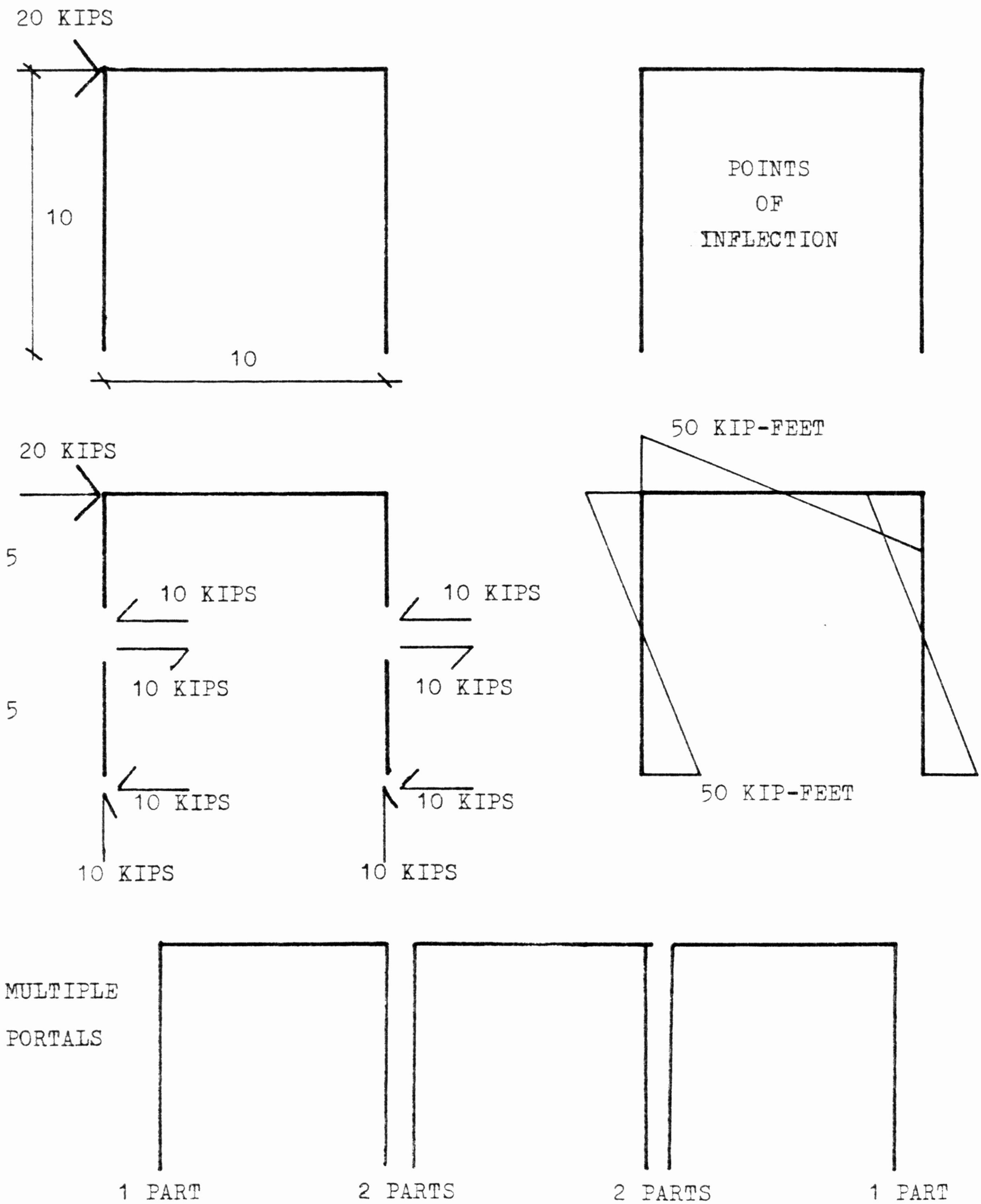


FIGURE 4. PORTAL ANALYSIS ASSUMPTIONS AND CALCULATIONS

LOADING CONDITIONS

The nature of loads that have an effect on high-rise buildings is a subject that must be given careful consideration. The forces that may have only a small effect on low-rise buildings may have a considerable effect on the stability and serviceability of a high-rise structure. An understanding of the effects of lateral loads and many other secondary conditions is necessary to get an understanding of the building behavior in its natural environment.

Loads Due to Volume Changes in Buildings Material

Many of the new design trends involve the exposure of the structural frame of the building as a response to the reduction of the building's weight and cost. The result of such design methods is the vulnerability of the building to loads due to temperature changes. The structural member is now exposed to both the controlled temperature of the interior of building and the seasonal changes on the exterior. This temperature differential may cause movement in the member due to contraction for temperature drop and expansion for an increase in temperature.[14]

The effect of such temperature induced moment is dependent a great deal on the original structural rigidity of the building and is generally proportional to the number of floors within the structure. There are a variety of different responses that may be evident in a building that is not properly designed for temperature differential. Some of them are column bending in exterior columns due to temperature differentials, differential movement between exterior and interior columns, floor cracking and resulting partition damage.

Similar to the effects caused by temperature differentials is the effect that creep shrinkage may occur in the materials.This effect is usually time-dependent based on the magnitude of the stresses and the initial strength of the concrete mix. If careful consideration is made during the design of the original mix, shrinkage may be reduced up to 40%.

Lateral loads

The most influential loading condition in today's high-rise building is the influence of lateral loads such as wind action and earthquate conditions. The early skyscrapers were not as affected by such loads due to the tremendous weight of their masonry walls. Wind forces could not overcome the gravity laods to become an influencing factor. However, as the design of high-rise buildings shifted toward the development of more open interior spaces and an overall lighter structure weight the consideration and response to lateral loads become a prime consideration. The use of longer spanning beams, non-load bearing movable interior partitions, and use of glass curtain walls have taken away from the overall rigidity of the building and the resistance to lateral sway has become a prime consideration.[15]

Considerable research and literature has been compiled on the effect of wind on buildings. A true understanding of wind and the ability to predict its behavior may be impossible in scientific terms. The dynamic effect of wind is influenced by many factors such as the roughness and form of terrain of the surrounding environment, the shape of the structure itself and the effect of arrangement with respect to other buildings.

In studies of wind velocity there has been readings indicating two different types of action: a mean wind velocity and a gust wind velocity. The mean wind velocity is representative of a static loading condition and gust velocity is the dynamic component of the lateral forces. The wind mean velocity is generally a function of the height of the building. If there is considerable ground clutter due to trees, land forms, or other buildings, the maximum mean velocity will be found at a higher height.

While extensive research is being conducted on the dynamic actions of wind, the building codes are fomulated as a static approch to this action. Wind pressure values are determined as to a maximum annual mean velocity, 30 ft. above the ground for a 50-year recurrence interval. The mean velocities were obtained by the U. S. Weather Bureau. These tablified values have been established for regional areas (See Table 1). The code has also established coefficients for building shape which reduce the value of the mean velocity. The code approach however, does not take into account the gust effects of wind action nor the effect of environmental conditions which may influence building response.[16] Research must continue in this field for designers to get a better conceptual understanding of the dynamic component of wind.

Another important factor that must be considered wnen designing for wind loads is the tolerance that the occupants have to building

Table 1. Uniform Building Code-Wind Pressure Values

TABLE NO. 23-F—WIND PRESSURES FOR VARIOUS HEIGHT ZONES ABOVE GROUND[1]

HEIGHT ZONES (in feet)	WIND-PRESSURE MAP AREAS (pounds per square foot)						
	20	25	30	35	40	45	50
Less than 30	15	20	25	25	30	35	40
30 to 49	20	25	30	35	40	45	50
50 to 99	25	30	40	45	50	55	60
100 to 499	30	40	45	55	60	70	75
500 to 1199	35	45	55	60	70	80	90
1200 and over	40	50	60	70	80	90	100

[1]See Figure No. 4. Wind pressure column in the table should be selected which is headed by a value corresponding to the minimum permissible, resultant wind pressure indicated for the particular locality.

The figures given are recommended as minimum. These requirements do not provide for tornadoes.

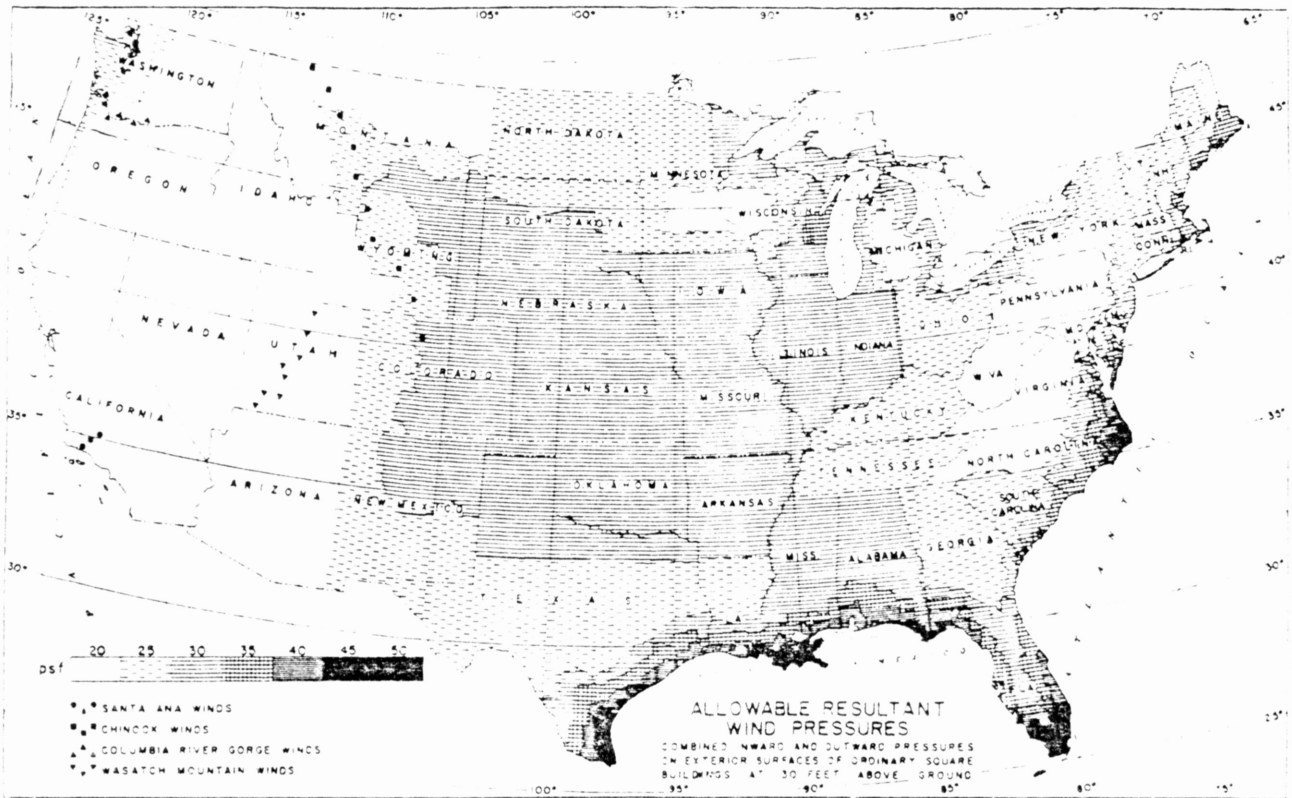


FIGURE NO. 4

1979 EDITION

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23-4

TABLE NO. 23-G—MULTIPLYING FACTORS FOR WIND PRESSURES—CHIMNEYS, TANKS AND SOLID TOWERS

HORIZONTAL CROSS SECTION	FACTOR
Square or rectangular	1.00
Hexagonal or octagonal	0.80
Round or elliptical	0.60

sway. The building structure may be able to withstand such sway but occupants may experience motion sickness. This requires a further design reduction in the lateral sway of the building.

This lateral sway in the building is called drift and is measured in relation to the building height by a factor called the drift index (Δ/h). It is common practice to design the building with sufficient lateral stiffness to keep the drift index between 0.0015 and 0.0030. This is usually based on worst mean velocity for a "ten-year storm." It is recommended that the building withstand safely "50-year storms" and "100-year storms."[17]

In addition to wind loads, buildings in some regions are also susceptible to seismic conditions. The usual practice when designing for earthquake loads is to determine an additional lateral load based on some percentage of the total weight of the building. A common range for this percentage is 5 to 10%. Some designers simply increase percentages of wind loads for seismic loads, however, this assumption is not very well justified because seismic loads are more of a function of building weight rather than exposed area.[18]

RESEARCH OBJECTIVE

During the course of the research of statically indeterminate analysis techniques, a question was raised pertaiing to the extent of the application of the approximate method of analysis--the portal method. In 1940, a comprehensive study was done by a subcommittee of American Society of Civil Engineers on the analysis methods of steel buildings under lateral loads. It was determined by this committee that the portal method produces satisfactory results in buildings up to 25 stories in height.[19]

However, in discussion with several practicing engineers it was determined that the portal method is generally not considered a very reliable method for buildings in excess of only six stories.

It was from this discrepancy in the accepted theory and practice of the portal method that the objective of this paper was formulated. In order to develop an understanding of high-rise structures under the influence of lateral loads it was determined that an examination would be made of the actual accuracy of the portal method in comparison with some more exact methods available. See Figure 5.

Methods

Because of the agreement over the accuracy of the portal method in a six-story frame, it was determined to be the starting height for comparitive analysis and the comparison would progress at a floor by floor rate until approximately 75 stories or a pattern of error was discovered.

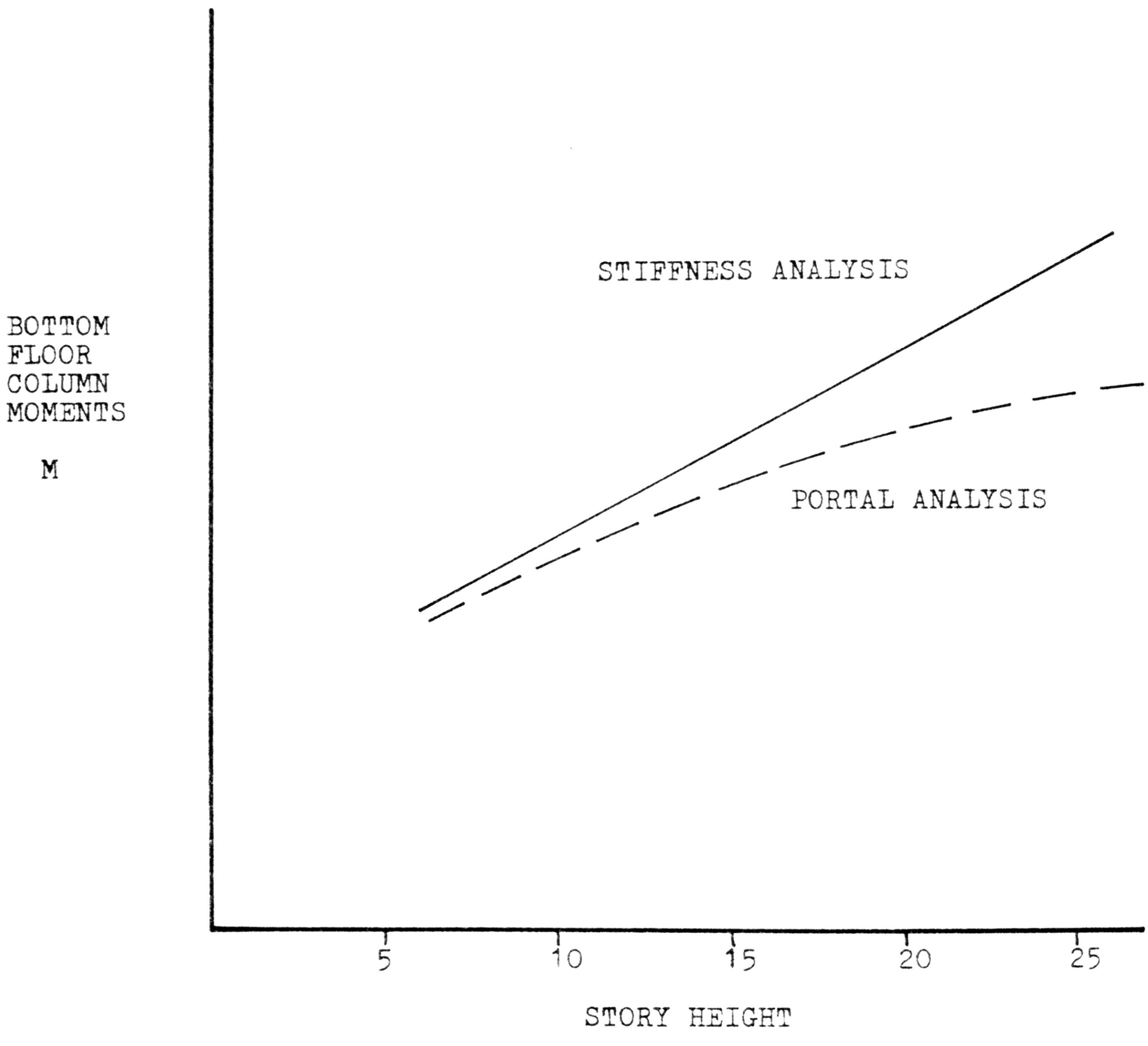

FIGURE 5. RESEARCH OBJECTIVE

The results from the portal method were obtained by hand calculations made rather easily from the procedure described earlier in this paper. The determination of a building frame size was one of the important parts of the comparison procedure. In order for the portal analysis results to be valid the building fram had to meet the general criteria that was outlined earlier in this paper. It was decided that a three-bay frame of 2-1-2 proportions would be satisfactory to simplify both the hand calculations of the portal method and the computer input for the more exact results, and yet still have the flexibility in reaching the desired story height. See Figure 6.

Loads to be applied to the frame were obtained from tables for wind loads in the 1979 Uniform Building Code. These wind pressure values were described earlier and are tablized in Table 1.

The results for the exact analysis were obtained through the use of a finite element program utilizing the stiffness method. The program, called STRUDL (STRUctural Design Language), was developed in the mid and late 1960's as part of the integrated Civil Engineering System (ICES), a group of engineers gathered at the Massachusetts Institute of Technology (MIT) to study computer applications. With the assistance of Professor Lee Lowry in the Dept. of Civil Engineering, an input was developed for a six-story moment resisting plane frame. It was decided that the use of printed cards would be the best means of manipulating the data file for the different story heights and also save the cost of terminal time.

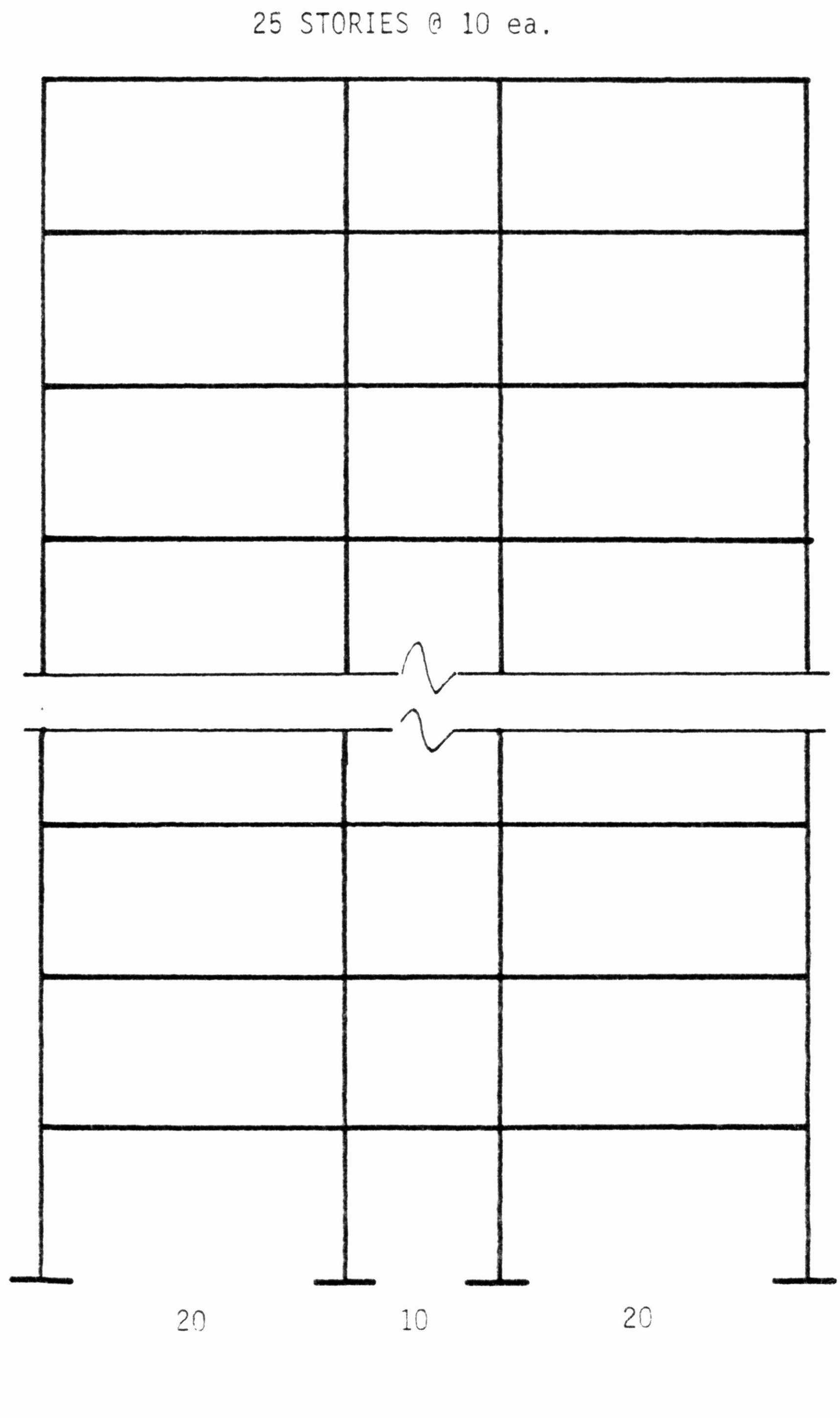

FIGURE 6. RESEARCH--FRAME DETERMINATION

Results

Using the above described procedure, a comparitive analysis was made of the frame from a six-story height to a twelve story height. At this time, it was observed that a pattern of error had been developed. While it was originally thought that as story height increased the accuracy of the portal method would decrease, it was found that the method proved to be as accurate at the six-story frame height as at the twelve-story height.

As a measure of accuracy, the percentage difference between portal moments and stiffness moments proved to be very effective. The examination of the interior column moments showed that the percentage difference of the top three floors tended to be very similar regardless of story height and that the top floor was consistently over 30% difference (See Table 2). A similar examination was made of the exterior-column moments and it was found that on the bottom floor and upper two floors the portal method produced conservative differences and the percentage difference was comparable regardless of overall story height. All other results were well within a 15% difference on the non-conservative side (See Table 3). A graphical representation of the results was made and it was discovered that the portal method did not produce answers of less accuracy as story height increased but that all of the results fell within an envelope of accuracy (See Figure 7).

After careful analysis of the data was made, a question was brought to mind as to whether factors could be derived from the per-

Table 2. Moment Comparison--Portal vs. Stiffness

FLOOR	STIFFNESS	PORTAL	% DIFF
1	17621	15135	14% L
2	14770	12825	13% L
3	12566	10305	18% L
4	9760	7785	20% L
5	6400	4905	23% L
6	2820	1735	38% L
1	30268	25320	16% L
2	27288	23010	16% L
3	24658	20700	16% L
4	21902	18180	17% L
5	18595	15300	18% L
6	14925	11128	19% L
7	10908	8663	21% L
8	6788	5198	23% L
9	2733	1733	37% L
1	45728	38025	17% L
2	42601	35715	16% L
3	39396	33405	15% L
4	36695	30885	16% L
5	33368	28005	16% L
6	29682	24833	16% L
7	25647	21367	17% L
8	21563	17903	17% L
9	17486	14483	17% L
10	13424	10973	18% L
11	8745	6930	21% L
12	3344	2310	31% L

Table 3. Moment Comparison--P vs. S--Exterior Columns

FLOOR	STIFFNESS	PORTAL	% DIFF
1	14488	10090	30% L
2	6587	8550	23% H
3	5944	6870	13% H
4	4843	5190	7% H
5	3543	3270	8% L
6	1376	1155	16% L
1	24767	16880	32% L
2	11446	15340	25% H
3	11058	13800	20% H
4	10011	12120	17% H
5	8741	10200	14% H
6	7304	8085	10% H
7	5628	5775	3% H
8	3916	3465	12% L
9	1587	1155	27% L
1	37341	25350	32% L
2	18219	23810	23% H
3	17332	22270	22% H
4	16343	20590	21% H
5	15112	18670	19% H
6	13698	16555	17% H
7	12058	14245	15% H
8	10389	11935	13% H
9	8720	9625	9% H
10	7172	7315	2% H
11	5331	4620	13% L
12	2417	1540	36% L

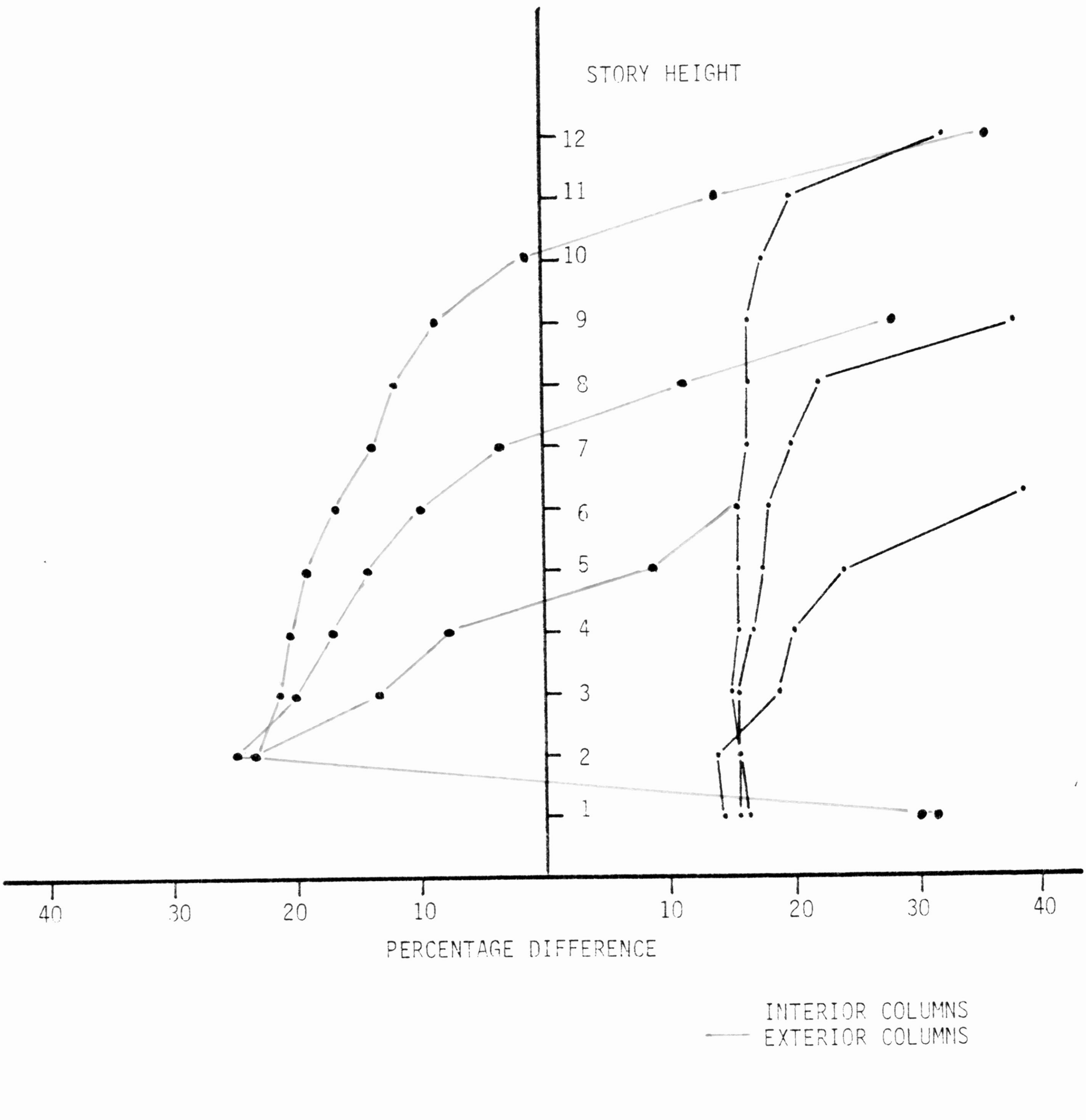

FIGURE 7. PORTAL ACCURACY ENVELOPE

cent differences and applied to a frame of similar proportions. It was decided that an application of this hypothesis would be made to a similar six-story frame. Factors were derived simply from the percentage difference in moments obtained form the earlier comparison. These factors were then applied to moments calcualted by hand utilizing the portal method and compared to results from a stiffness analysis provided by the computer. This comparison is tablized in Table 4. As can be seen the portal method with use of the adjustment factors produced results that were much more in line with the more accurate analysis results. However, the results for the top floor interior columns and the bottom floor exterior were considerably different in comparison to the other results obtained. This increase in the percentage difference for these two cases is one that is hard for us to explain and an area that requires further investigation.

One possible explanation for these differences may stem from the additional moment that may be induced in the building due to lateral sidesway. As mentioned earlier, this was one of the problems that was encountered with the moment distribution method when analyzing wind loads and was the spearhead of the research by Grinter. Since the stiffness method is based on joint translations and rotations it is likely that the induced moments from sidesway would appear in its tabulated results. It may be this induced moment that is producing the original percentage differences. The differences that appear after adjustment factors in bottom floor exterior columns and top floor interior columns may be due to distribution of moment due to a possible increase in lateral sidesway.

Table 4. Moment Comparison w/Adjustment Factors

INTERIOR COLUMNS

FLOOR	PORTAL MOMENT	ADJ. FACTOR	FINAL MOMENT	STIFFNESS	% DIFF
1	25920	1.14	29549	30604	3% L
2	21930	1.13	24781	25696	4% L
3	17940	1.18	21169	21891	3% L
4	13452	1.20	16142	16913	5% L
5	8478	1.23	10428	11083	6% L
6	2993	1.38	4130	4900	16% L

EXTERIOR COLUMNS

FLOOR	PORTAL MOMENT	ADJ. FACTOR	FINAL MOMENT	STIFFNESS	% DIFF
1	17280	1.30	22464	25142	11% L
2	14622	0.77	11259	11430	1% L
3	11958	0.87	10403	10347	0%
4	8964	0.93	8337	8355	0%
5	5652	1.08	6104	6102	0%
6	1996	1.16	2315	2344	1% L

Another possible explanation for the differences may stem from the original assumption that points of inflection in a portal frame lie at mid-height of columns and mid-depths of beams. Upon comparison to actual points of inflection it was found that this assumption was only true for the middle floors of the frame (See Figure 8). It is possible that portal analysis results based on these new points of inflection would produce better results.

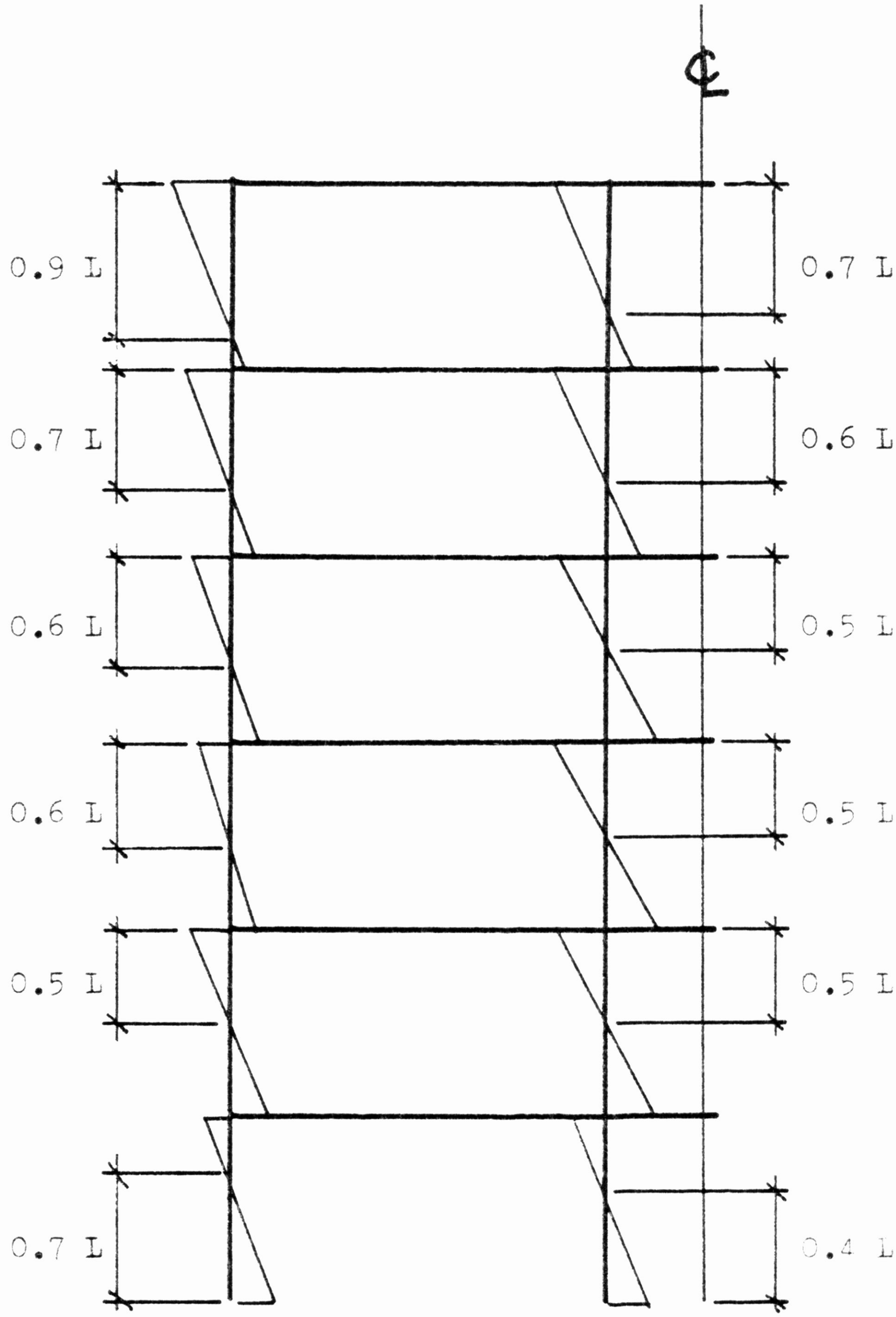

FIGURE 8. ACTUAL POINTS OF INFLECTION

CONCLUSION

It was the purpose of this investigation to examine some of the simplified methods and possibly bring them closer in line to true values when their accuracy drops off. It was originally intended to carry the comparison procedure up to the 24 story height. Problems were encountered due to the considerable high cost of running a STRUDL analysis program. Considerable time and effort was spent in an effort to obtain a similar program through another system that would be free for my use. However, after running into problems with access to this program it was felt that in an effort to save time the STRUDL analysis would be continued. After running programs through the 12-story height a pattern was discivered and the analysis stopped there.

As stated earlier, the need for accurate analysis techniques is continually growing due to the increasing utilization of the high-rise building. The use of simplified methods with the application of adjustment factors may produce very good estimates for the architect, structural engineer, and contractor to determine overall building structure size and weight. And with further refinement and investigation it is felt that these could be applicable.

While our investigation involved only one particular building frame it is hoped that this research would continue with the investigation of other frames.

References

[1]M. Smolina, Analysis of Tall Buildings by the force displacement (New York: John Wiley & Sons, 1975), p. 1.

[2]John I. Parcel and Robert B. B. Moorman, Analysis of Statically Indeterminate Structures (New York: John Wiley & Sons, 1955), p. 1.

[3]Jack C. McCormac, Structural Analysis (Scranton, Pa.: International Textbook Co. 1960), p. 172.

[4]Parcel, p. 8.

[5]Crawley Dillion, Steel Buildings: Analysis and Design, 2nd Edition (New York: John Wiley & Sons, 1970, 1977), p. 306.

[6]Dillion, p. 313.

[7]Robins Fleming, "Wind Stress Analysis and Moment Distribution", Engineering News Record, 9 February 1933, p. 194.

[8]L. E. Grinter, "Wind Stress Analysis Simplified," American Society of Civil Engineers, January 1933, p. 3-27.

[9]Dillon, p. 382.

[10]McCormac, p. 177.

[11]McCormac, p. 188.

[12]Jack C. McCormac, Steel Building Design, 3rd Edition. (New York: Harper Row, 1981) p. 582.

[13]McCormac, p. 190.

[14]Wolfgang Schueller, High Rise Building Structures, (New York: John Wiley and Sons, 1977) p. 39.

[15]Schueller, p. 11.

[16]Schueller, p. 11.

[17]McCormac, Steel Building Design, p. 573.

[18]McCormac, Steel Building Design, p. 574.

[19]Lambert Fall, Ed-in-Chief, Structural Steel Design (New York: Ronald Press Co., 1964) p. 712.

Bibliography

1. American Concrete Institute. Response of Multistory Concrete Structures to Lateral Forces. Detrpot: ACD Publication, 1973.

2. Crawley, Stanley W., Robert M. Dillon. Steel Building: Analysis and Design. 2nd. ed. New York: John Wiley and Sons, 1977.

3. Fleming, Robins. "Wind Stress Analysis and Moment Distribution." Engineering News Record. February 9, 1933, pp. 194-195.

4. Grinter, L.E., "Wind Stress Analysis Simplified," American Society of Civil Engineers Journal. Jan. 1933, pp. 3-27.

5. Houghton, E.L., N. B. Carruthers. Wind forces on buildings and structures: An Introduction. New York: John Wiley and Sons, 1976.

6. McCormac, Jack C. Structural Analysis. Scanton, Pa.: International Textbook Co., 1960.

7. McCormac, Jack C. Steel Building Design. 3rd. ed. New York: Harper Row, 1981.

8. Mac Donald, Angus J. Wind Loadings on Building's New York: John Wiley and Sons, 1975.

9. Parcel, John I., Robert B. B. Moorman. Analysis of Statically Indeterminate Structures. New York: John Wiley and Sons, 1955.

10. Schueller, Wolfgang. High-Rise Building Structures. New York: John Wiley and Sons, 1977.

11. Smolira, M. Analysis of tall buildings by the force-displacement method. New York: John Wiley and Sons, 1975.

12. Tall, Lambert, Ed. in Chief. Structural Steel Design. New York: Ronald Press Co., 1964.

13. West, Harry H. Analysis of Structures. New York: John Wiley and Sons, 1980.

14. Yamamoto, Y. J.F. Oden, R.W. Clough. eds. Advances in Computational Methods in Structural Mechanics and Design. Huntsville, Ala.: University of Alabama in Huntsville Press, 1972.

APPENDIX

101A 5310

GARRISON

******* JES3 JOB SUMMARY *******

171 CARDS READ

876 LINES SPOOLED

0 CARDS SPOOLED

***** WRITER CHARACTERISTICS *****

DEVICE= PRT2 TRAIN= PN

FORMS= 1100 FCB= 8

SYSOUT= A

TEXAS A&M UNIVERSITY
DATA PROCESSING CENTER

AMDAHL 470/V6/V8 MVS/JES3

12.42.35 PM 24 FEB 82.055

PORTAL

//GARRISON JOB (L025,101A,SC2,001,RK),'PORTAL'

INTEGRATED CIVIL ENGINEERING SYSTEM - IUG VERSION V2MO, JUNE 1976

- ICES -

FEB 21, 1992 TIME=12.41.49

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STRUDL 'CASE 1' 'PORTAL'

    ****************************************************
    *                                                  *
    *               ICES STRUDL-II                     *
    *        THE STRUCTURAL DESIGN LANGUAGE            *
    *                                                  *
    *        IUG VERSION V3M0,   JUNE 1976             *
    *           SIZE OF POOL   8K BYTES                *
    *           12:42:17        2/24/82                *
    *                                                  *
    ****************************************************

UNITS FEET KIPS
OUTPUT DECIMAL 2
TYPE PLANE FRAME
JOINT COOR
1 0 0 SUPPORT
2 20 0 SUPPORT
3 30 0 SUPPORT
4 50 0 SUPPORT
5 0 10
6 20 10
7 30 10
8 50 10
9 0 20
10 20 20
11 30 20
12 50 20
13 0 30
14 20 30
15 30 30
16 50 30
17 0 40
18 20 40
19 30 40
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20	50	40
21	0	50
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23	30	50
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25	0	60
26	20	60
27	30	60
28	50	60
29	0	70
30	20	70
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33	0	80
34	20	80
35	30	80
36	50	80
37	0	90
38	20	90
39	30	90
40	50	90
41	0	100
42	20	100
43	30	100
44	50	100
45	0	110
46	20	110
47	30	110
48	50	110
49	0	120

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50 20 120
51 30 120
52 50 120
MEMBER INC
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2 2 6
3 3 7
4 4 8
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6 6 7
7 7 8
8 5 9
9 6 10
10 7 11
11 8 12
12 9 10
13 10 11
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16 10 14
17 11 15
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20 14 15
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22 13 17
23 14 18
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26 17 18
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68 41 42
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71 41 45
72 42 46
73 43 47
74 44 48
75 45 46
76 46 47
77 47 48
78 45 49
79 46 50
80 47 51
81 48 52
82 49 50
83 50 51
84 51 52
UNITS INCHES
MEM 1 TO 84 PROP PRIS AX 10 AY 10 IX 20 IY 20 IZ 20
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CONSTANTS E 30000.0 ALL
UNITS FEET POUNDS
LOADING 1 'ALL REMAINING LOADS'
JOINT LOADS
5 FORCE X 1540
9 FORCE X 1540
13 FORCE X 1680
17 FORCE X 1920
21 FORCE X 2115
25 FORCE X 2310
29 FORCE X 2310
33 FORCE X 2310
37 FORCE X 2310
41 FORCE X 2695
45 FORCE X 3080
49 FORCE X 1540
LOADING LIST ALL
STIFFNESS ANALYSIS
LIST FORCES REACTIONS DISPLACEMENTS ALL
```

```
***************************
*RESULTS OF LATEST ANALYSIS*
***************************

JOB ID - CASE 1          JOB TITLE - PORTAL

ACTIVE UNITS -  LENGTH      FORCE       ANGLE       TEMPERATURE     TIME
                FEET        LB          RAD         DEGF            SEC

ACTIVE STRUCTURE TYPE - PLANE   FRAME

ACTIVE COORDINATES AXES  X Y

   ******************************************************************************
   *  LOADING - 1           ALL REMAINING LOADS                                  *
   ******************************************************************************

   MEMBER  FORCES
```

MEMBER	JOINT	FORCE AXIAL	FORCE SHEAR Y	FORCE SHEAR Z	MOMENT TORSIONAL	MOMENT BENDING Y	MOMENT BENDING Z
1	1	-22931.55	5078.16				37341.47
1	5	22931.55	-5078.16				13440.08
2	2	-45256.65	7599.60				45728.70
2	6	45256.65	-7599.60				30267.31
3	3	45256.82	7598.48				45722.85
3	7	-45256.82	-7598.48				30261.96
4	4	22931.38	5073.75				37316.18
4	8	-22931.38	-5073.75				13421.35
5	5	14.89	-2953.78				-31647.82
5	6	-14.89	2953.78				-27427.82
6	6	166.34	-9087.76				-45440.03
6	7	-166.34	9087.76				-45439.52
7	7	1519.09	-2953.25				-27424.03
7	8	-1519.09	2953.25				-31641.01
8	5	-19577.77	3553.04				18207.74
8	9	19577.77	-3553.04				17222.63
9	6	-30122.48	8351.06				42600.54
9	10	30122.48	-8351.06				40910.07
10	7	30122.12	8351.23				42601.59
10	11	-30122.12	-8351.23				40910.71
11	8	19578.12	3554.56				18219.66
11	12	-19578.12	-3554.56				17326.97
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12	13	-1322.56	3150.73				-29598.66
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13	11	-771.14	9968.29				-49841.37

14	11	212.48	-3150.80	-29599.09
14	12	-212.49	3150.80	-33416.96
15	9	-16827.04	3342.60	16093.35
15	13	16827.04	-3342.60	17332.67
16	10	-32304.92	7792.64	38530.05
16	14	32304.92	-7792.64	39396.29
17	11	32304.63	7792.57	38529.75
17	15	-32304.63	-7792.57	39395.99
18	12	16827.32	3342.18	16089.98
18	16	-16827.32	-3342.18	17331.83
19	13	1416.26	-2996.20	-31777.41
19	14	-1416.26	2996.20	-28146.62
20	14	839.71	-9343.46	-46717.35
20	15	-839.71	9343.46	-46717.26
21	15	263.49	-2996.08	-28145.77
21	16	-263.49	2996.08	-31775.84
22	13	-13830.83	3078.86	14444.73
22	17	13830.83	-3078.86	16343.86
23	14	-25957.66	7216.29	35467.69
23	18	25957.66	-7216.29	36695.14
24	15	25957.25	7216.16	35467.04
24	19	-25957.25	-7216.16	36694.57
25	16	13831.24	3078.69	14444.00
25	20	-13831.24	-3078.69	16342.87
26	17	1647.17	-2764.48	-29292.22
26	18	-1647.17	2764.48	-25997.28
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30	22	20198.28	-6529.04	33368.87
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32	24	-11066.37	-2805.97	15111.62
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33	22	-1810.37	2495.83	-23489.93
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34	23	-1057.61	7592.00	-37959.92
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35	24	-394.83	2495.73	-26425.44
36	21	-8570.52	2501.40	11314.80
36	25	8570.52	-2501.40	13699.19
37	22	-15102.11	5776.28	28081.10
37	26	15102.11	-5776.28	29681.70
38	23	15101.49	5776.18	28080.61
38	27	-15101.49	-5776.18	29681.15
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41	27	-1154.74	6585.51	-32927.50
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42	28	-332.24	2198.58	-23258.15
43	25	-6371.89	2161.75	9559.56
43	29	6371.89	-2161.75	12057.93

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48	31	-1155.12	5541.80	-27709.01
49	31	335.89	-1885.94	-17789.72
49	32	-335.88	1885.94	-19929.14
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50	33	4485.95	-1826.09	10389.71
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51	34	7059.37	-4141.45	21562.78
52	31	7059.71	4141.45	19851.70
52	35	-7059.71	-4141.45	21562.75
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53	36	-4436.61	-1826.01	10389.32
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54	34	-1973.19	1569.58	-14827.99
55	34	1154.84	-4495.99	-22479.96
55	35	-1154.84	4495.99	-22479.97
56	35	336.53	-1569.59	-14828.07
56	36	-336.53	1569.59	-16563.77
57	33	-2917.37	1489.27	6173.91
57	37	2916.37	-1489.27	8718.77
58	34	-4132.95	3323.10	15745.17
58	38	4132.95	-3323.10	17485.85
59	35	4132.30	3323.14	15745.29
59	39	-4132.30	-3323.14	17486.13
60	36	2917.02	1489.48	6174.45
60	40	-2917.02	-1489.48	8720.36
61	37	1981.46	-1248.75	-13154.29
61	38	-1981.46	1248.75	-11820.75
62	38	1155.34	-3442.15	-17210.82
62	39	-1155.34	3442.15	-17210.68
63	39	328.98	-1248.67	-11820.14
63	40	-328.98	1248.67	-13153.32
64	37	-1667.62	1160.73	4435.52
64	41	1667.62	-1160.73	7171.79
65	38	-1939.56	2496.98	11545.72
65	42	1939.56	-2496.98	13424.07
66	39	1938.83	2496.79	11544.69
66	43	-1938.83	-2496.79	13423.16
67	40	1669.35	1160.50	4432.96
67	44	-1669.35	-1160.50	7172.04
68	41	2298.59	-993.07	-9483.01
68	42	-2299.58	993.07	-8578.43
69	42	1348.25	-2313.34	-11566.85
69	43	-1348.25	2313.34	-11566.57
70	43	397.74	-902.77	-8576.29
70	44	-397.74	902.77	-9479.16
71	41	-764.55	764.32	2311.22
71	45	764.55	-764.32	5331.94
72	42	-529.29	1546.65	8721.21
72	46	529.29	-1546.65	8745.27
73	43	528.26	1546.27	6719.70
73	47	-528.26	-1546.27	8742.98

74	44	765.57	762.76	2307.12
74	48	-765.57	-762.76	5320.52
75	45	2568.63	-525.06	-5453.71
75	46	-2568.63	525.06	-5047.46
76	46	1537.28	-1102.42	-5511.86
76	47	-1537.29	1102.42	-5512.36
77	47	507.34	-525.37	-5049.75
77	48	-507.34	525.37	-5457.55
78	45	-232.49	252.95	121.77
78	49	232.49	-252.95	2407.72
79	46	48.08	515.30	1814.06
79	50	-48.09	-515.30	3338.91
80	47	-48.90	516.33	1819.12
80	51	48.10	-516.33	3344.17
81	48	240.21	255.43	137.03
81	52	-240.21	-255.43	2417.22
82	49	1287.05	-239.49	-2407.72
82	50	-1287.05	239.49	-2382.02
83	50	771.75	-191.41	-956.89
83	51	-771.75	191.41	-957.19
84	51	255.43	-240.21	-2386.97
84	52	-255.43	240.21	-2417.22

RESULTANT JOINT LOADS - SUPPORTS

JOINT		/------ FORCE ------ X FORCE	Y FORCE	Z FORCE	//------ MOMENT ------/ X MOMENT	Y MOMENT	Z MOMENT
1	GLOBAL	-5078.16	-22931.55				37341.47
2	GLOBAL	-7599.60	-45256.65				45728.70
3	GLOBAL	-7598.48	45256.82				45722.85
4	GLOBAL	-5071.75	22931.38				37316.18

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT		/------ DISPLACEMENT ------ X DISP.	Y DISP.	Z DISP.	//------ ROTATION ------/ X ROT.	Y ROT.	Z ROT.
1	GLOBAL	0.0	0.0				0.0
2	GLOBAL	0.0	0.0				0.0
3	GLOBAL	0.0	0.0				0.0
4	GLOBAL	0.0	0.0				0.0

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT		/------ DISPLACEMENT ------ X DISP.	Y DISP.	Z DISP.	//------ ROTATION ------/ X ROT.	Y ROT.	Z ROT.
5	GLOBAL	0.25	0.00				-0.03
6	GLOBAL	0.25	0.00				-0.02
7	GLOBAL	0.25	-0.00				-0.02
8	GLOBAL	0.25	-0.00				-0.03
9	GLOBAL	0.61	0.00				-0.03
10	GLOBAL	0.61	0.00				-0.02

PLOT PLANE

PLANE IDENTIFIED BY - PLANE Z EQUALS 0.0

IN PLANE JOINTS

JOINT	COORDINATES X	Y	Z
1	0.0	0.0	0.0
2	20.0000	0.0	0.0
3	30.0000	0.0	0.0
4	50.0000	0.0	0.0
5	0.0	10.0000	0.0
6	20.0000	10.0000	0.0
7	30.0000	10.0000	0.0
8	50.0000	10.0000	0.0
9	0.0	20.0000	0.0
10	20.0000	20.0000	0.0
11	30.0000	20.0000	0.0
12	50.0000	20.0000	0.0
13	0.0	30.0000	0.0
14	20.0000	30.0000	0.0
15	30.0000	30.0000	0.0
16	50.0000	30.0000	0.0
17	0.0	40.0000	0.0
18	20.0000	40.0000	0.0
19	30.0000	40.0000	0.0
20	50.0000	40.0000	0.0
21	0.0	50.0000	0.0
22	20.0000	50.0000	0.0
23	30.0000	50.0000	0.0
24	50.0000	50.0000	0.0
25	0.0	60.0000	0.0
26	20.0000	60.0000	0.0
27	30.0000	60.0000	0.0
28	50.0000	60.0000	0.0
29	0.0	70.0000	0.0
30	20.0000	70.0000	0.0
31	30.0000	70.0000	0.0
32	50.0000	70.0000	0.0
33	0.0	80.0000	0.0
34	20.0000	80.0000	0.0
35	30.0000	80.0000	0.0
36	50.0000	80.0000	0.0
37	0.0	90.0000	0.0
38	20.0000	90.0000	0.0
39	30.0000	90.0000	0.0
40	50.0000	90.0000	0.0
41	0.0	100.0000	0.0
42	20.0000	100.0000	0.0
43	30.0000	100.0000	0.0
44	50.0000	100.0000	0.0
45	0.0	110.0000	0.0

46	20.0000	110.0000	0.0
47	30.0000	110.0000	0.0
48	50.0000	110.0000	0.0
49	0.0	120.0000	0.0
50	20.0000	120.0000	0.0
51	30.0000	120.0000	0.0
52	50.0000	120.0000	0.0

IN PLANE MEMBERS
MEMBER INCIDENCES

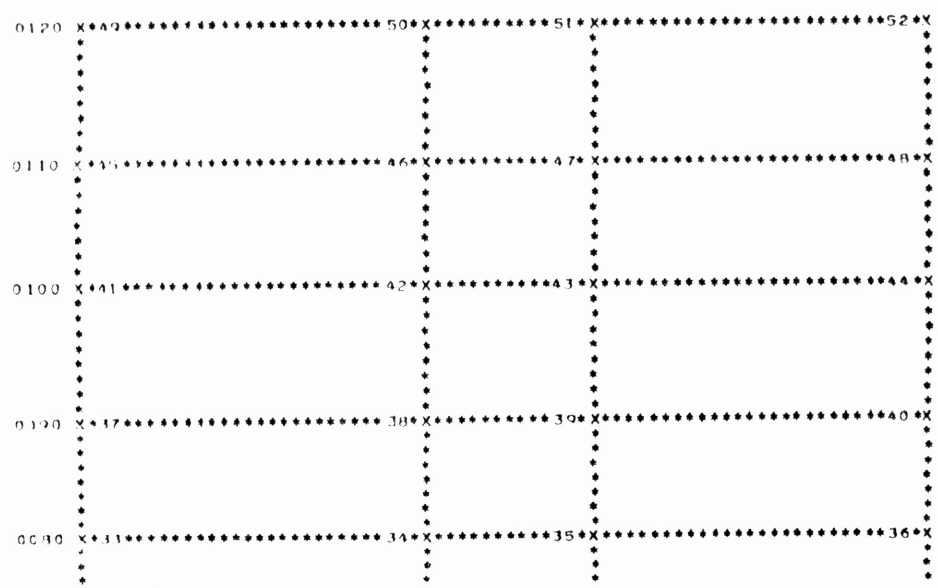
MEMBER	START	END
1	1	5
2	2	6
3	3	7
4	4	8
5	5	6
8	5	9
6	6	7
9	6	10
7	7	8
10	7	11
11	8	12
12	9	10
15	9	13
13	10	11
16	10	14
14	11	12
17	11	15
18	12	16
19	13	14
22	13	17
20	14	15
23	14	18
21	15	16
24	15	19
25	16	20
26	17	18
29	17	21
27	18	19
30	19	22
28	19	20
31	19	23
32	20	21
33	21	22
36	21	[illegible]
34	22	23
37	22	26
35	23	[illegible]
38	23	27
39	24	[illegible]
40	25	[illegible]
43	25	27
41	26	30
44	26	[illegible]
42	27	31
45	27	[illegible]
46	28	[illegible]

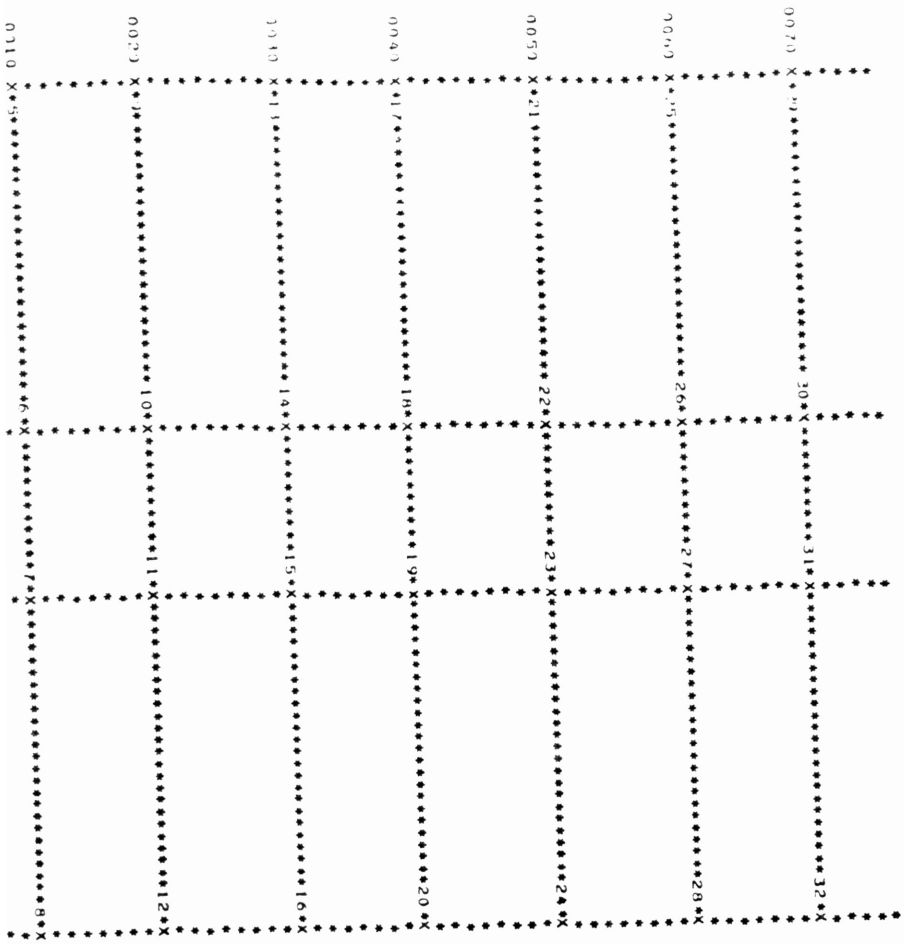
47	29	30
50	29	33
48	30	31
51	30	34
49	31	32
52	31	35
53	32	36
54	33	34
57	33	37
55	34	35
58	34	39
56	35	36
59	35	39
60	36	40
61	37	38
64	37	41
62	38	39
65	39	42
63	39	40
66	39	43
67	40	44
68	41	42
71	41	45
69	42	43
72	42	46
70	43	44
73	43	47
74	44	48
75	45	46
78	45	49
76	46	47
79	46	50
77	47	49
80	47	51
81	48	52
82	49	50
83	50	51
84	51	52

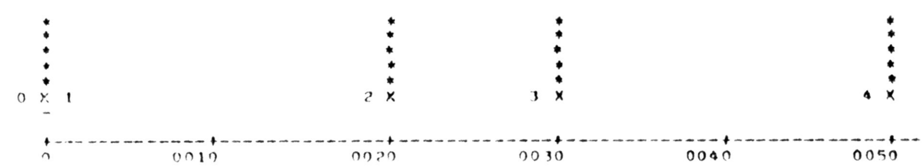
ORIENTATION Y X

HORIZONTAL SCALE 7.0000 UNITS PER INCH

VERTICAL SCALE 7.0000 UNITS PER INCH






0 X 1
2 X
3 X
4 X
0
0010
0020
0030
0040
0050

FINISH

GOOD-BYE