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# Computer technique for the elastic-plastic analysis of multi-storey sway frames: a storey-by-storey approach

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# Computer technique for the Elastic-Plastic analysis of Multi-Storey Sway Frames - A storey-by-storey approach.

Johannesburg, 1985

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A project report submitted to the Faculty of Engineering, University of the Witwatersrand, Johannesburg, in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

## DECLARATION

I declare that this project report is my own, unaided work. It is being submitted for the Degree of Master of Science in Engineering in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.



G Fuller

17 March 1988

## ABSTRACT

This project report describes the development of a computer technique for the elastic-plastic analysis of multi-storey sway frames.

The program uses the Scholz interaction method applied to the structure on a storey-by-storey basis. The rigid plastic collapse load and elastic buckling load of each storey sub-structure are calculated and applied in the interaction method to obtain the storey failure load. The lowest storey failure load is then taken as the failure load of the structure.

The suitability of this technique for the analysis and design of multi-storey sway frames is investigated. This is done by designing a frame using the proposed method and then analysing it with rigorous methods to evaluate the design. The program is also used to analyse previously investigated frames and compare results with rigorous analysis results.

It has been concluded that the simplifications introduced by the proposed method make it extremely well suited to the design of multi-storey sway frames.

## DEDICATION

To the people of Langa, Uitenhage

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## LIST OF SYMBOLS

| | |
|---|---|
| A | section area |
| c | factor to proportion yield stress to give effective yield stress |
| $C_m$ | moment magnification factor |
| $\{D_f\}$ | displacement vector |
| E | Young's modulus |
| $\{F\}$ | load (force) vector |
| $f_m$ | magnification factor to take P-Delta effects into account |
| fyb | beam yield stress |
| fyc | column yield stress |
| H | horizontal (wind) point |
| $H_s$ | proportion of total horizontal load acting on storey s |
| h | storey height |
| $I_b$ | beam moment of inertia |
| $I_c$ | column moment of inertia |
| [K] | stiffness matrix |
| $K_b$ | beam member stiffness (EI/L) |
| $K_c$ | column member stiffness (EI/h) |
| L | member length |

| | |
|---|---|
| $\ell$ | beam length (bay width) |
| $M_p$ | moment capacity of section at formation of plastic hinge |
| $M_{pb}$ | beam plastic moment capacity |
| $M_{pc}$ | column plastic moment capacity |
| $P$ | column axial load |
| $P_f$ | failure load of structure |
| $P_o$ | elastic buckling load |
| $P_p$ | rigid plastic collapse load |
| $P_y$ | axial load of section at yield |
| $r$ | radius of gyration |
| $U$ | ratio of sum of axial loads on a column above and below floor level to the load on the column below floor level |
| $w$ | uniformly distributed beam load |
| y-max | distance from section centroid to extreme fibre |
| $Z_p$ | plastic section modulus |
| $\alpha$ | factor to take into account any deviation from full symmetry in structure or loading |
| $\gamma$ | load parameter (load factor) |
| $\gamma_c$ | critical load factor |
| $\gamma_f$ | load factor at failure |
| $\gamma_o$ | elastic buckling load factor |
| $\gamma_p$ | rigid plastic collapse load factor |

$\gamma_q$ load factor at formation of hinge q

$\Delta$ lateral displacements

$\Delta_f$ first order lateral displacements

$\Delta_s$ second order lateral displacements

$\theta$ rotational displacements

$\lambda$ slenderness ratio of member (L/r)

$\lambda_\ell$ slenderness ratio of member in the "limiting frame"

$\lambda_{0.4H}$ largest slenderness ratio associated with all conditions producing deviation from the special case of "symmetry buckling"

$\psi$ ratio of column stiffness to sum of adjoining beam stiffnesses

## 1.0 INTRODUCTION

It has long been realised in the design of multi-storey sway frames that economic advantages can be gained by taking full plasticity into account. This necessitates a second-order, elastic-plastic analysis of such frames which should take possible secondary effects into account. The most important secondary effect is the so-called P-Delta moment, i.e. the additional moments which arise when the axial forces on the columns interact with the storey sway.

As a result of the secondary effects failure of the structure through instability may occur before a plastic collapse mechanism can form. It is therefore imperative to take secondary effects into consideration in the analysis of multi-storey sway frames. The analysis of such frames can be approached in a number of different ways:

1. Elastic-plastic analysis of the entire structure.

2. Elastic-plastic analysis of single column sub-assemblages or storeys.

3. Interaction method using the Merchant-Rankine rule.

4. Interaction method by Scholz applied to the whole structure or on a storey-by-storey basis.

The analysis approaches mentioned above will be examined in more detail in the literature survey of this report.

## 1.1 RESEARCH OBJECTIVE

The objective of this report is to develop a computerised version of the Scholz Interaction method applied to steel sway frames on a storey-by-storey basis. Using this program, the Scholz method will be applied to a number of different frames. The results of these analyses will be compared with results obtained by other researchers on the same frames. The suitability of this method will also be investigated in regard to the design of large multi-storey steel sway frames.

The interaction method by Scholz (1983a) is a recent development which attempts to overcome the restrictions and shortcomings of the Merchant-Rankine method without adding to the complexity of the analysis. So far the method has only been applied to a few discrete frameworks on a storey-by-storey basis. The suitability of this method when applied to large multi-storey frameworks in general still needs to be evaluated.

Like the Merchant-Rankine method, the Scholz method makes use of the structure elastic buckling load and rigid-plastic collapse load to evaluate the failure load. The advantage of the Scholz method is that, if applied on a storey-by-storey basis, the computations are considerably simplified.

In the examples done to date, the elastic buckling load of the storey sub-structure has been calculated using the storey stiffness method of Cheong Siat-Moy (1976a). This method is simple to apply and has given good results. It would thus appear to be the most suitable way of calculating the elastic buckling load for the storey.

A possible disadvantage of the Scholz method and other interaction methods is the calculation of the rigid-plastic collapse load. On a large struc-

ture, even if only a storey sub-structure is being considered, many possible collapse mechanisms have to be investigated. If the Scholz method is to be computerised, a suitable rigid-plastic analysis will have to be developed which could be applied to the storey model. A method recently published by Liang and Yuan (1984) appears to offer the best solution to this step.

## 1.2 OUTLINE OF REPORT

1. A literature survey of the various existing methods of elastic-plastic analysis of multi-storey sway frames.

2. Development of the Computer Program: A method for calculating the rigid-plastic collapse load of the storey model is developed. Cheong Siat-Moy's storey stiffness method is then utilised to compute elastic buckling load for each storey. The program then uses both the rigid-plastic collapse load and the elastic buckling load to evaluate the storey failure load on the basis of the interaction technique by Scholz. The smallest storey failure load is taken as the elastic-plastic failure load of the structure.

3. Parametric comparisons: The Scholz method is first applied to a ten storey frame designed by Cheong Siat-Moy to investigate the suitability of the method for design. Comparisons are then made with analyses by other researchers and where discrepancies arise they are discussed.

4. Conclusions are presented on the application of the Scholz method in a computerised form on a storey-by-storey basis to large sway frames.

## 2.0 LITERATURE SURVEY

### 2.1 BACKGROUND

The accepted way of designing multi-storey sway frames subjected to gravity and wind loading is to modify the forces obtained by a first-order analysis. The first-order forces under-estimate the actual forces which would develop in the frame members. The difference in the first-order forces and the actual forces would be due to the so-called 'secondary effects'.

The most significant secondary effect arises when the gravity loads on the floors and columns interact with the lateral displacements caused by wind loads. If the frame were acted upon by the wind loads only, first-order displacements $\Delta_f$ would result. If the gravity loads were now applied to the structure in its deformed position these loads would interact with the displacements $\Delta_f$ to produce secondary moments, called the P-Delta moments. These moments in turn cause the structure to deflect further into a new equilibrium position $\Delta_s$. The first-order deflection has thus been magnified to $\Delta_s$, the new second-order deflection. It is obvious that these second-order moments and deflections should be taken into account in the design of multi-storey structures.

Several other sources which could increase the first-order deflections and thus moments have been identified (Cheong Siat-Moy, 1977). Semi-rigid connections between beams and columns would cause an increase in the lateral deformation of the structure. Panel-zone effects, column shortening and initial storey eccentricities would have a similar effect. The reduction in stiffness of members due to axial forces and residual

stresses in the members should also be taken into account. In large frames with columns under heavy axial loads these second-order effects can result in instability of frameworks at loads substantially less than those given by a first-order analysis. Thus consideration of the secondary effects is essential in determining the overall stability of the frame.

The general approach in building codes has been the introduction of the 'effective length' concept to take secondary effects into consideration. It is believed that the frame will posses adequate strength if it designed using the forces obtained from a first-order elastic analysis, provided that effective column lengths are used in the design. In this way the designer is spared all the complications of a second-order analysis.

However it has been demonstrated by Cheong Siat-Moy (1978) and Scholz (1982) that design using the 'effective length' ... does not always ensure frame stability. In the light of these ... it would appear that the secondary effects should explicitly be t... into account in the analysis and the design should be based on the resulting second-order forces.

It is normally accurate enough for engineering purposes to take only the P-Delta effects into account and ignore the others. There has been a great deal of research done in trying to develop suitable methods of taking second-order effects into account in the analysis and design of structures. A brief review of the work done on this topic is presented here. Three basic methods of approaching the problem can be identified:

1. Rigorous elastic-plastic analysis of the entire framework

2. Approximate second-order analysis applied to suitable sub-assemblages or to the structure as a whole.

3. Application of interaction formulae.

## 2.2 RIGOROUS ELASTIC-PLASTIC ANALYSIS OF THE ENTIRE FRAMEWORK

It was pointed out by Scholz (1981) that full-frame analyses, using the exact moment-curvature relationship of all the members, are rare and have only been performed on limited structures. Investigations on simple portal frames with pinned bases have been carried out by Chu and Pabacius (1964), Moses (1964), Yura and Galambos (1964) and Adams (1964). These analyses are very time consuming and place great demands on the computer. They are therefore not very useful to the design engineer.

It is more common to analyse the entire frame using a second-order elastic-plastic method of analysis. This is normally done by utilising the slope-deflection equations for each member and combining them to form the structure stiffness matrix. The load-displacement relationship is used to calculate the member forces and moments. By formulating the analysis on the deformed shape, both strength and stability effects are taken into account. This method assumes that when the fully plastic moment is reached at a certain point in a member (normally an end or midspan point), a plastic hinge forms at this discrete point while the rest of the member remains elastic. The loading is applied incrementally and a second order elastic analysis is performed at loads between the formation of consecutive hinges. With the formation of each additional hinge the stiffness of the structure is reduced and the stiffness matrix has to be adjusted accordingly. The final collapse load of the structure is attained when the determinant of the structure stiffness matrix changes sign or

becomes zero. A detailed survey on computer based methods for plastic analysis and design was published by Grierson (1964).

Jennings and Majid (1965) used the matrix method outlined above to develop a computer program which performs a second-order, elastic-plastic on general frames. This paper was expanded upon by Majid and Anderson (1968) where some of the difficulties of analysing large frameworks using this method are discussed. Comparison is also made in this paper between their program results and the results of other researchers.

The analysis method presented in these papers is based on monotonically increasing load paths and the formation of plastic hinges is predicted by iteration. Between the formation of plastic hinges the structure is treated in a piece-wise elastic manner. When a plastic hinge develops it is registered in the analysis by a new term in the structure stiffness matrix. Thus with the formation of each new hinge the size of the stiffness matrix is increased from an $(n)x(n)$ matrix to an $(n+1)x(n+1)$ matrix. Provision was not made in this method for the unloading of plastic hinges.

The matrix method program was further developed by Horne and Majid (1966) to incorporate both analysis and design features. Their program is capable of considering reductions in the moment capacities of the members due to axial loads. More secondary effects were taken into account by Parikh (1966) who incorporated the effect of both residual stresses and axial shortening of column members. Davies (1966) further extended this work by developing a program capable of analysis under variable loading patterns. His program also took into account the effect of strain hardening and hinge reversals on the second-order bending analysis.

Korn and Galambos (1968) utilised these developments in a matrix method to make a study of the effect of axial deformations on second-order

analysis results. They did not make provision for handling hinge reversals although their method did detect and report the occurrence of reversal of hinge angles. This report concluded that axial deformations have negligible effects on failure loads but that hinge reversals should be considered.

Vijakkhana, Nishino and Lee (1974) developed a method of second-order elastic-plastic analysis in which the stiffness matrix is so derived that it can be applied to any piece-wise elastic regime in the elastic-plastic deformation of a member. They make use of approximate stability functions to take the effect of axial forces into account and make comparisons with results obtained using the exact stability functions. The application of their analysis to the design of frames is also demonstrated in this paper.

Davison and Adams (1974) developed a second-order, elastic-plastic analysis program based on the matrix method in which provision for hinge reversals in the beams was made. In the columns the effect of axial shortening was taken into consideration but no provision was made for hinge reversals. Using their program they compared the effect of secondary P-Delta moments on braced and unbraced frames. They concluded that the P-Delta effects are as important for braced as for unbraced frames due to the fact that bracing is not infinitely stiff. The authors go on to suggest that current design practices, which imply that the P-Delta effects in braced frames are insignificant, should be discarded. A design procedure using a second-order analysis, with no distinction between braced and unbraced frames, should be introduced.

A further development to the matrix method was made by Liang and Yuan (1984) in their proposed method of second-order, elastic-plastic analysis. Instead of increasing the size of the stiffness matrix by adding a new column and row to accomodate an extra unknown rotation when a hinge forms, the stiffness matrix for the member with the hinge is modified.

The new member stiffness matrix is then inserted in the structure stiffness matrix. This modified structure stiffness matrix remains the same size as before, which, they claim, greatly reduces the computer time when analysing large structures.

When considering the suitability of these analysis methods for design application, one must bear in mind that all of them require the structure to be analysed as a whole. This would have important implications for the design of large structures as the data processing and program running time could prove unacceptable.

## 2.3 APPROXIMATE SECOND-ORDER METHODS

Sub-assemblage techniques have been developed in an effort to approximate second-order elastic-plastic analyses of structures without the use of computers. However, as noted by Scholz (1981) and Cheong Siat-Moy (1976a), these sub-assemblage methods seem to be limited for application only to single storey frames.

A series of lecture notes published by Lehigh University (1965) titled "Plastic Design of Multi-Story Frames", sets out an approximate second-order, elastic-plastic design method. They suggest that this method could be applied by hand but for large frameworks this would entail numerous repetitive calculations. In this method the members are initially sized using an approximate plastic analysis for the structure under factored gravity loads only. For the combined loading case a lateral deflection is assumed for each storey and a preliminary design is carried out. This is done by calculating the storey sway and then revising the members until the calculated lateral deflection and the assumed de-

flections approach each other to within tolerable limits. The adequacy of the member sizes thus selected must then be checked by calculating each storey's shear resistance. This is done by plotting the shear-displacement curve for each beam-column sub-assemblage in the storey and then superimposing these curves to obtain the storey shear-displacement curve. The selected member sizes are adequate if the total storey resistance is greater than the applied storey shear. Otherwise the members must be adjusted and the checking procedure repeated.

This method is very laborious and time consuming and not suitable for the design of large frameworks. Furthermore, results obtained by Davison and Adams (1974) and this report, indicate that this method is conservative. It appears that the procedure becomes more inaccurate the more columns there are in a storey. This could be attributed to the fact that each column in a storey is treated individually and then summed to obtain the total storey resistance. The more columns there are therefore, the greater the accumulation of errors.

As an alternative approximate method Cheong Siat-Moy (1976a,1976b) has developed a method to approximate a second-order elastic-plastic analysis for general frames under horizontal and vertical loading. This method relates the failure load of the frame to the storey-stiffness; instability failure being reached when the storey stiffness becomes zero or negative. The frame can thus be sub-divided into storeys and each storey can be analysed independently of the rest of the frame. This method thus has great potential for simplifying the analysis of large frameworks.

With frames subjected to pure gravity loads prediction of the critical load is greatly simplified by considering instability as a function of storey stiffness. By tracing the changes in storey stiffness as loading is increased, the load at the onset of instability is identified as the

load at which the story stiffness becomes zero. The buckling load of the structure is then the critical load of the weakest storey.

Inelastic sway buckling can be treated in a similar manner; allowance being made for plastic hinges. The drawback of this method is that five possible types of failure must be investigated for each column-beam sub-assemblage. This could prove to be a lengthy process when a large frame is being analysed.

The storey stiffness method can also be applied to the design of multi-storey frames. The load at which a hinge forms is determined by calculating the load at which a hinge would form in each beam of the storey model. The stiffness of the critical beam is then adjusted, which alters the storey stiffness, and the resulting moment changes are recorded. The process is then repeated to find the next hinge and a load vs. deteriorating storey stiffness curve is plotted. The beam members are sized so that the computed failure load factor is greater than the required design load factor.

Column sizes are then selected so that plastic hinges will form in them under the loading factored by the design load factor. The presence of these column hinges causes the storey stiffness to become zero. This method could be applied using hand computations but the plotting of the load vs. storey stiffness curve for each storey would require a great deal of work for a large framework.

In a later paper Cheong Siat-Moy (1977) shows how secondary effects other than P-Delta effects can be taken into account using his proposed method. It is shown that by magnifying the first-order deflections $\Delta_F$, the effects of semi-rigid connections, initial storey eccentricities, column axial shortening, panel-zone deformations and column yielding can be taken into account. He proposes too (1978) that all these second-order effects be

taken into account when designing multi-storey frames instead of camouflaging the secondary effects by the use of the 'effective length' concept in the design process.

Cheong Siat-Moy (1978) points out the inconsistencies of the 'effective length' approach and advises that in design overall frame stability should be considered separately from member stability. By applying the storey stiffness method the designer could calculate the approximate second-order forces and thus eliminate the 'effective length' concept.

Another approximate method presented by MacGregor and Hage (1977) is the Moment Magnifier solution for second-order effects. This method uses an approximation of the 'storey critical load' developed by Rosenbleuth (1967), Goldberg (1973) and Stevens (1967). The storey critical load is used to calculate a moment magnification factor which is applied to the first-order moments to take the P-Delta effects into account. This method is not very accurate and is used mainly for preliminary design.

The P-Delta Method of analysis is an approximate technique which will give an acceptable design estimate of the second-order forces and moments in an elastic structure. This analysis takes into account the 'sway forces' induced by the P-Delta moments. In this method the lateral and vertical loads are applied to the structure and the relative first-order lateral displacements are found for each storey using an elastic analysis on the whole structure. These displacements are used with the storey vertical loads to compute an additional sway force for each storey. The sway forces are then added to the original lateral loads. The total second-order forces and moments can now be computed using a first-order analysis. The final additional sway forces can either be calculated directly or by using an iterative procedure.

Wood, Beaulieu and Adams (1976) have shown that this method can be used for sway frames designed by the allowable stress technique suggested by the American (1978) and Canadian (1969) structural steel codes. In this method the second-order effects are usually compensated for in an empirical manner by the use of effective length and $C_m$ factors in the design of the members. If second-order moments are directly calculated the members can be designed on the basis of a sway prevented model, resulting in a more accurate analysis. The authors also point out in this paper that only the column moments will be adjusted when second-order effects are taken into account when using the effective length method. Using the results of a second-order elastic analysis will result in both the column and beam moments being adjusted for the P-Delta effects.

In their design optimisation program, Emkin and Litle (1970) use an iterative process, combined with the results of a second-order elastic analysis, to take P-Delta effects into account. In their method the structure is analysed storey-by-storey from the top storey down. The member sizes for each storey are obtained by iteration and adjusting member sizes until convergence of lateral deflections is obtained. When all the storeys have been treated in this manner the overall structure lateral deflections are calculated. If these deflections are within 5% of the deflections used to calculate each total storey shear the design is complete. If not, the calculated deflections become those of the P-Delta effect in the next cycle of the design. The iterative procedure then begins once again at the top storey and repeats until the convergence condition is met.

Thus, even though the design is carried out with a storey-by-storey procedure, the entire structure must be analysed to confirm the design of one storey. The computations involved would not be suitable for hand calculation. It is therefore not comparable with the storey-by-storey

approach advocated by Cheong Siat-Moy (1976b) with which one could isolate one storey and analyse it by hand.

The paper discussed earlier by MacGregor and Hage (1977) also makes mention of the Negative Member Bracing Method. Nixon, Beaulieu and Adams (1976) have shown that a direct solution of second-order effects can be obtained by using a standard first-order analysis incorporating fictitious diagonal bracing members of negative stiffness. These fictitious bracing members make the structure more flexible, the increased flexibility compensating for P-Delta effects.

With the exception of Cheong Siat-Moy's Storey Stiffness Method, all the approximate methods discussed above require analysis of the structure as a whole. This has a serious disadvantage especially in the preliminary design stage. Member sizes are often being changed at this stage and the effect of each change can only be ascertained by re-analysing the entire structure. If the Storey Stiffness Method is used a change in member size would necessitate the re-analysis of only one storey. However, the disadvantage of this method, as mentioned earlier, is that a number of different failure mechanisms have to be investigated for each storey. Furthermore, when using this method for plastic design the plotting of the load vs. storey stiffness curve makes it a cumbersome technique in the design of frames.

## 2.4 USE OF INTERACTION FORMULAE

The work of Rankine (1866) and Merchant (1954) has pioneered the concept of using interaction formulae in determining the stability of structural frameworks. In the Merchant-Rankine method, the failure load of the

structure $P_f$, is obtained by using the value of the plastic collapse load $P_p$ and the elastic buckling load $P_o$ in the following interaction rule :

$$P_f = P_p/(1 + P_p/P_o) \qquad \ldots[\text{ Eq.1 }]$$

Wood (1974) made modifications to the formula to take into account a minimum amount of strain hardening and restraining action due to composite behaviour. His revised rule takes the following format :

$$P_f = P_p/(0.9 + P_p/P_o) \qquad \ldots[\text{ Eq.2 }]$$

Scholz (1981) among others, has pointed out the inadequacy of this method when applied to slender and flexible frameworks. For this reason the use of this method has been limited to frames for which $P_p/P_o$ is less than 0.25. Frames not complying with this condition have to be analysed using alternative elastic-plastic methods.

Another second-order elastic-plastic design rule was developed by Lu (1965) from symmetrical portal frames with pinned bases subjected to symmetrical vertical loading. Scholz (1981) has pointed out a number of shortcomings of this method for the design of general frames. The main shortcomings are that the design rule is only applicable to vertical loading, it was developed for pinned base conditions and would have to be amended to take other base conditions into account and there is no allowance made for any deviation from full symmetry.

In an attempt to eliminate some of the shortcomings of the Merchant-Rankine method without adding to the complexity of the computation involved, Scholz (1981) has developed an improved version of the *Merchant-Rankine approximation* to evaluate the stability of sway frames. This method also incorporates the rigid-plastic collapse load $P_p$ and the elastic buckling load $P_o$. The application of this method to general frames

has no equivalent limitations imposed on it and gives a much better correlation with more rigorous methods than the Merchant-Rankine approach (which has been proposed by the European Convention for Structural Steelwork (ECCS) and is has been included in the new British Specifications for the Structural use of Steelwork in Buildings). Scholz's method can be applied to the framework as a whole or to suitable storey sub-assemblages.

A brief summary of the method described by Scholz (1983a) applied to a storey-wise analysis is outlined below:

- Sub-divide the frame into storeys assuming points of contraflexure to occur at mid-column height (except bottom storeys).

- Determine the rigid plastic collapse load of each storey sub-frame.

- Determine the elastic buckling load of each storey.

- Carry out an approximate second-order elastic analysis on each storey loaded with the appropriately factored elastic buckling load (factor = 1.0 for vertical loading only ; = 0.4 for combined case of vertical and horizontal loading).

- For each storey calculate the slenderness of the real frame and of the limiting frame and identify the lowest real frame/limiting frame slenderness ratio. The limiting frame is defined as a frame for which failure and first yield coincide, this criterion being used to determine its slenderness. It has been verified by Scholz (1981) that each real frame can be grouped into a specific family of frames represented by a curve in a frame interaction diagram as shown in Fig.1. A specific curve of Fig.1 is characterised by a frame not affected by P-Delta moments ( on left vertical axis of Fig.1 ) and the

so-called "limiting frame" ( intersection point of curve on right vertical axis ). The curved portion in between those boundaries is empirical and the shape has been confirmed by a large number of discrete experimental and rigorous analytical examples.

- Calculate the elastic buckling load/plastic collapse load ratio for the limiting frame.

- Read off the failure load from the relevant interaction graph, such as shown in Fig.1. The ratio of elastic buckling load to plastic collapse load together with the slenderness ratio for the "limiting frame" identifies the appropriate curve from the array of possible curves of Fig.1, whereas the same ratio for the real frame leads to an intersection point on the selected curve indicating the failure load.

So far this interaction technique has only been applied to a few discrete frame structures and a generalised computer version is outstanding. Scholz (1983b) has also shown how second-order effects other than P-Delta effects can be accounted for using his interaction method. In this paper he argues that the effects of non-uniform temperature, differential settlement, column axial shortening or initial storey eccentricities can be accounted for in the same way as the strength loss associated with applied horizontal loading. He also goes on to show how the affects of semi-rigid connections and residual stresses can be taken into account.

The interaction methods provide us with a powerful tool for taking second-order effects into account and ensuring stability in the design of structural frameworks. However, it is a time consuming task to calculate the rigid plastic collapse load of a large framework. The storey-by-storey approach suggested by Scholz (1981) thus has the potential for substantially reducing this task. If a more efficient method can

$\gamma_F$ = FAILURE LOAD FACTOR
$\gamma_P$ = PLASTIC COLLAPSE LOAD FACTOR
$\gamma_C$ = ELASTIC BUCKLING LOAD FACTOR

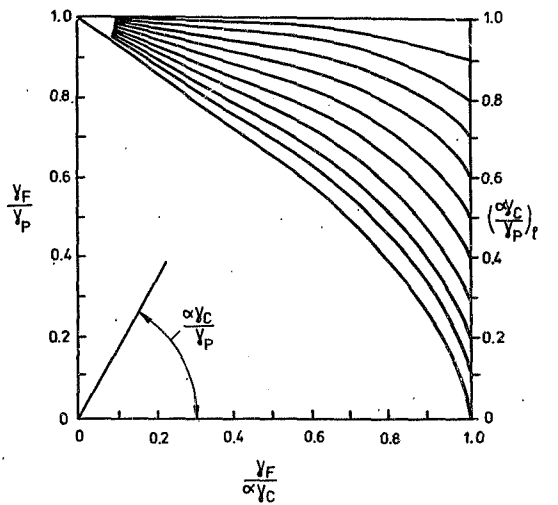



Figure 1. Basic interaction curves. Scholz (1981)

be found of calculating the rigid-plastic collapse load of the storey model we would have a very elegant technique at our disposal for the design of frames. This report develops a suitable rigid plastic collapse analysis which is applied to the story model and investigates the possibility of applying this analysis on a storey by storey basis.

## 3.0 COMPUTER PROGRAM

### 3.1 INTRODUCTION

The objective throughout the development of the computer program was to introduce as many simplifying assumptions as possible whilst maintaining sufficient accuracy for engineering purposes. The assumptions are made to economise on computer time required for the analysis.

The technique of analysing the structure on a storey-by-storey basis in itself helps a great deal in simplifying the analysis and saving on computer time. With only one storey being analysed at a time the number of unknowns involved at any stage of the computation is minimised. Furthermore, with a future view to developing the analysis technique into a design package, analysis of isolated stories has a major advantage. If the designer wants to adjust the size of one or two members he will not have to re-analyse the entire structure but only the affected storey or storeys. Using methods which analyse the entire structure as an entity, adjusting only one member would require re-analysis of the whole structure. This is a costly task for large structures and, furthermore, since the consequences of changing a member are not easy to foresee, may have to be repeated a number of times before achieving the desired effect.

Listed below are a few more commonly made simplifying assumptions which have been adopted in the presented computerised analysis technique:

- all frames are regular in geometry, planar and unbraced.
- there are no initial storey eccentricities.

- rigid member connections are assumed and bases are either fully rigid or pinned.

- members are compelled to bend in the plane of the frame about their major axes. No provision is made in the presented program for minor axis bending but could easily be incorporated if required.

- the possibility of lateral torsional and local member buckling is ruled out.

- shearing zone deformations and panel zone deformations are not considered.

- axial deformations and shortening due to bending have been ignored.

A flow chart outlining the program structure is given in Fig.2. The program has been written in BASIC for a HP9816 micro computer with a capacity of 512 kbytes. The maximum size structure that can be analysed by the program in its present format is twenty bays wide by fifty stories high. The base supports must either be all fixed or all pinned. Vertical loads must be made up of uniformly distributed loads on the beams and concentrated point loads at the beam-column intersection nodes. Horizontal loading consists of concentrated point loads applied at floor levels. Young's modulus,E is a constant for all members of the structure. Provision is made for the beam members and the column members to have different yield stresses but all beams must have the same yield stress and likewise all columns. The user is asked to specify the value for a factor,c by which all yield stresses will be factored to give the effective yield stresses i.e. the stress corresponding to the occurrence of first yield. For each member in the structure the following section properties are required : Moment of inertia I, Plastic modulus Zp, Area

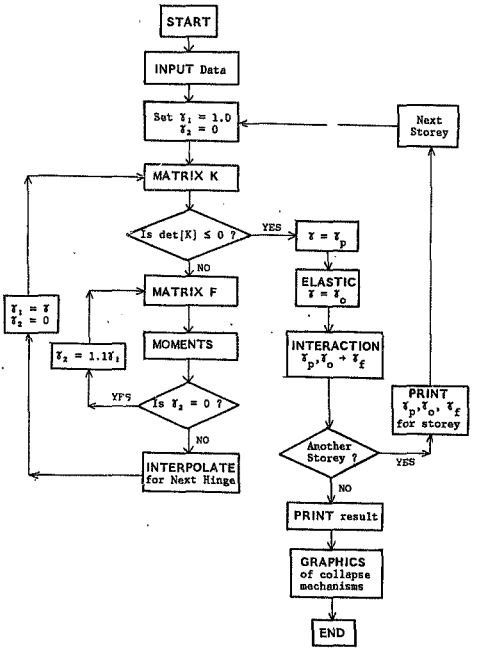



Figure 2. Flow Chart of Computer Program

A and Distance from the centroid to the extreme fibre, y-max, which, for symmetrical sections, equals half the section depth.

In the program the analysis begins by first analysing the top storey. Thereafter each remaining storey is isolated and analysed, starting at the top and working down to the bottom storey. The program is discussed step by step below, outlining more simplifying assumptions applicable to each step.

Sections 3.2 to 3.6 outline the calculation of the rigid plastic collapse load factor for a given storey. The flow chart shows that initially the entered loading (normally working load) is unfactored i.e. the load parameter $\gamma = 1.0$. The loading is incremented proportionally by increasing the load parameter $\gamma$ and multiplying all loads by $\gamma$.

## 3.2 THE STIFFNESS MATRIX

The stiffness matrix [K] is set up using the slope-deflection equations without stability functions. The stiffness matrix is simplified by assuming points of contraflexure to be at mid-height of the columns (except bottom storeys). This means that the column member stiffness equations can be expressed in terms of two unknowns, namely the rotation at the beam-column connection, $\theta$ and the column chord slope $\Delta/h$. Fig.3 shows a typical internal storey model, in which $K_{ci}$ = stiffness of column i, $K_{bi}$ = stiffness of beam i, $w_i$ = udl on beam i, $P_i$ = axial load on column i, $H_s$ = proportion of total wind load acting on storey s, $\ell_i$ = length of beam i and h = storey height.

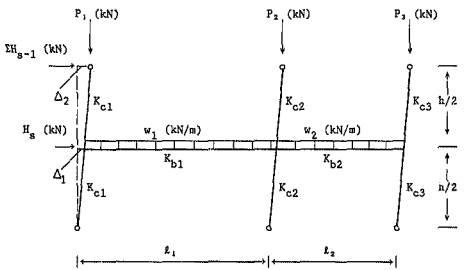



Figure 3. Typical internal storey model.

The slope of the beam members is assumed to remain constantly zero (i.e. no relative vertical displacements). This enables us to write the beam member equations in terms of two unknowns also, namely the end rotations at each end of the beam. No axial shortening of the beams is taken into account which implies that the lateral sway of all the columns at any given level is equal. Therefore the column chord slopes ($\Delta/h$) for all columns at a particular level are equal.

Thus for a storey model of n bays we would have n+1 columns and two lateral sway deformations ($\Delta_1$ and $\Delta_2$ in Fig.3). The number of unknowns to be solved therefore is $(n+1)+2 = n+3$, which is likewise the size of the structure stiffness matrix.

When the plastic hinges form in the beams or columns, the technique presented by Liang and Yuan (1984) for adjusting the member stiffness matrix is utilised and no extra unknowns are introduced into the computation. Assuming that the member with no hinges has a stiffness of 1.0 at each

end, then if a hinge forms at the one end the stiffness is reduced to 0.75 at the end remote from the hinge and 0 at the hinge end. If a second hinge forms in the member the stiffness will be 0 for both ends meaning that the member provides no resistance to lateral forces. If a third hinge forms in a member, a member mechanism has formed and the structure has reached its capacity load i.e. the rigid plastic collapse load.

A more common indication of when the rigid plastic collapse load has been reached is when an overall structure collapse mechanism forms. This condition is identified mathematically when the determinant of the structure stiffness matrix becomes zero. A combination of hinges forming a joint mechanism is similarly identified by the determinant of the stiffness matrix. If the situation arises where the yield load of a column is reached before formation of an overall structure mechanism or joint mechanism, the yielded column is identified and the load at which this occurs is taken as the rigid-plastic collapse load.

## 3.3 THE FORCE VECTOR

In the present program, vertical loading on the structure is restricted to uniformly distributed loads on the beams and concentrated point loads on the columns. Horizontal loading comprises of concentrated point loads applied at floor levels, representing the sum of the wind loading over the storey height.

Fig.2 shows $H_s$ applied at floor level and $\Sigma H_{s-1}$ applied at mid-column height. The mid-column load is the accumulative sum of all higher storey lateral loads and is automatically computed and included in the computation for each storey. The vertical uniformly distributed load of each beam

is proportioned equally to the two supporting columns and added to any applied point load on the columns. The accumulative axial column load above the storey being considered is computed and applied to the relevant columns at the level of the storey being analysed. The loading is static, proportional and monotonically increasing. This means that, prior to setting up the load vector {F}, all the applied loads are factored by the same load factor, $\gamma$.

The applied load vector is assembled by calculating the moments and forces at the ends of the members in a "fixed ended" structure due to the applied loads. The moments and forces required are those in directions corresponding to the unknown joint displacements. The process of setting up the force vector is well documented in structural analysis textbooks dealing with the stiffness method and will not be discussed further here.

When a plastic hinge forms in a member the force vector must be revised to take into account the "fixed moments" at the hinges. In the force vector the plastic moment capacity is taken as the moment acting at the the point where the hinge formed and the stiffness of the member is adjusted to incorporate a new hinge. This is also dealt with by Liang and Yuan (1984) in their technique of adjusting the stiffness and force vectors to take plastic hinges into account without introducing any new unknowns. In Appendix A a detailed worked example can be found of how the plastic collapse load is computed for a storey model. This will illustrate the technique more clearly.

## 3.4 MOMENTS

The unknown deformations $\{D_f\}$ are calculated from the stiffness matrix and force vector using the relationship $\{D_f\} = [K]^{-1}\{F\}$. These deformations are then used to calculate the moments using the appropriate slope-deflection equations. The moments are first calculated for the forces factored by load factor $\gamma$ and then in a second cycle for the forces factored by $1.1\gamma$. The number 1.1 is an arbitrary number which is used merely to locate a second point on the linear force-deformation curve.

## 3.5 INTERPOLATION FOR NEXT HINGE

The moments calculated in the two cycles are now used together with the section moment capacities (Mp) to calculate, by interpolation, the position at which the next plastic hinge will form. The effects of strain hardening and residual stresses are ignored which implies an elastic-fully plastic load-deformation response of the material (Fig.4).

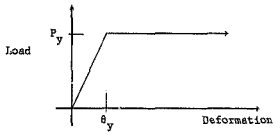



Figure 4. Idealised elastic-plastic response.

The concept of plastic hinges forming at discrete points is adopted implying that the spread of yielding in a member is not considered. Furthermore, no provision is made for the unloading of plastic hinges so that no hinge reversal occurs which could "unload" a plastic hinge to below the plastic moment capacity. This means that once a hinge has formed at a certain point in a member the moment at this point will be taken as the plastic moment capacity of the section for the remainder of the analysis.

The iteration procedure used to make a linear prediction of the load factor $\gamma$, at which the next plastic hinge forms is that outlined by Jennings and Majid (1965). Consider the bending moment at one end of a member to be $m_1$ for the first iteration and $m_2$ for the second. If the corresponding load factors are $\gamma_1$ and $\gamma_2$, we can predict the load factor at formation of a plastic hinge at this section by using the following relationship:

$$\gamma = \left|\frac{\gamma_2-\gamma_1}{m_2-m_1}\right| M_p + \frac{\gamma_1 m_2 - \gamma_2 m_1}{m_2-m_1} \qquad \ldots[\text{ Eq.3 }]$$

For beam members it is assumed that the maximum positive moments occur at midspan and the relationship above is used to calculate the load factor at which a hinge would form at three points in each beam, namely the left hand end, midspan and the right hand end. The midspan hinge was assumed to form at the beam centre to simplify the the calculation. It was felt that sufficient accuracy was being maintained with this approximation and thus unnecessary complexity was avoided. For column members the plastic moment capacity of the section is not a constant value : for steel sections it decreases with increasing axial load. The column plastic moment capacity for bending about the major axis is calculated from the approximate relationship Mpc = 1.18Mp(1-P/Py), where Mp = plastic moment capacity ignoring axial loads, P = axial load and Py = axial load of the section at yield. It can be seen that the column plastic moment capacity is related to the load parameter $\gamma$ and accordingly the relationship to predict the load parameter at formation of a plastic hinge in a column member has to be adjusted to:

$$\gamma = \frac{1.18Mp(\gamma_2-\gamma_1)Py + (\gamma_1 m_2 - \gamma_2 m_1)Py}{(m_2-m_1)Py + 1.18Mp(\gamma_2-\gamma_1)P} \qquad \ldots[\text{ Eq.4 }]$$

This relationship is used to predict the load parameter at which hinges form at the bottom ends of all upper columns and the top ends of all the lower columns in the storey model. The minimum value of $\gamma$ throughout the storey framework then gives the required $\gamma_p$ at which the next hinge will form.

The member in which this next hinge forms is identified and the member stiffness matrix adjusted to incorporate the new hinge. The structure stiffness matrix is then re-assembled using the new reduced member stiffness matrix and its determinant, det[K] is calculated. If det[K] remains greater than zero then the loop to calculate the next hinge is re-entered with the initial $\gamma_1$ equal to the $\gamma$ calculated at formation of the last plastic hinge. A stage will be reached where either there is a

member collapse mechanism or where det[K] becomes equal to zero, registering a structure collapse mechanism. The load parameter $\gamma_p$ at which this happens is the required rigid plastic collapse load factor.

The worked example in Appendix A illustrates more clearly the procedure used by the program to calculate $\gamma_p$. It must be noted however that in this example the column plastic moment capacities have not been adjusted for axial load. The procedure is nevertheless identical to that used by the program.

## 3.6 ELASTIC BUCKLING LOAD

The elastic buckling load parameter $\gamma_c$ is calculated using Cheong Siat-Moy's (1976b) storey stiffness method. The prediction of the critical load is made simple by considering buckling instability as a function of storey stiffness. The critical load of the storey is obtained from the relationship (for a storey with m columns) :

$$\gamma_c \Sigma P = 12E/h^2 \sum_{i}^{m} [I_c/(1+U\psi)] \qquad \ldots[\text{ Eq.5 }]$$

in which E = Young's modulus, h = storey height, $I_c$ = moment of inertia of the column, U = the ratio of the sum of the axial loads in the column above and below floor level to the load in the column below floor level, and $\psi$ = the ratio of the column stiffness to the sum of the stiffnesses of the adjoining beams. The term $\{I_c/(1+U\psi)\}$ is calculated using the beam and column member properties and lengths for each column-restraining beam sub-assemblage and then summed over the m columns in the storey. E is the Young's modulus and h the storey height. The left hand side of the equation $\gamma_c \Sigma P$ gives the total load that will be applied to the storey when

elastic buckling occurs. The elastic buckling load factor $\gamma_o$ is then obtained by dividing $\gamma_o \Sigma P$ by the total load $\Sigma P$ acting on that storey.

## 3.7 ELASTIC-PLASTIC FAILURE LOAD

Having calculated $\gamma_p$ and $\gamma_o$ for the storey the program now uses Scholz's interaction method to calculate the elastic-plastic load factor for the storey at failure. This method has been fully documented in the literature (Scholz : 1981,1983a) and only a brief outline of the steps followed will be presented here.

At the beginning of the analysis for each storey we have seen that the load factor $\gamma$ is taken as equal to 1.0, typically representing the working load level. Prior to calculating the moments with all loads factored by $\gamma = 1.0$, a loop in the program calculates the moments resulting from horizontal loads only factored by $\gamma = 1.0$ (vertical loads ignored). These moments are stored for use in the interaction method stage of the analysis. Similarly, the moments resulting from all loads, horizontal and vertical, factored by $\gamma = 1.0$, are stored in the initial stages of the analysis. Bearing this in mind the steps followed in applying the Scholz interaction method are outlined below:

- Apply horizontal loads corresponding to 0.4 times the elastic buckling load to the storey model. To take the elastic P-delta effects into account in this analysis factor the resulting elastic moments by a magnification factor $f_m = (1/(1 - 0.4Pc/Po)) = 1.667$. The program performs this step by retrieving the stored moments resulting from 1.0 x horizontal loads only and factoring them by $0.4 \times \gamma_o \times f_m$.

- Use these moments to calculate $\lambda_{0.4H}$, defined by Scholz (1981) as the largest slenderness ratio associated with all conditions producing deviation from the special case of "symmetry buckling", at the critical sections. In the program $\lambda_{0.4H}$ is calculated at points of maximum moment for the columns and for beams at the left hand end, at midspan and at the right hand end.

- Identify the largest $\lambda_{0.4H}$ obtained for any one member in the storey and use this to calculate the factor $\alpha = (0.4/(1-0.6 \times (700 - \lambda_{0.4H}/700)^3))$

- Calculate moments resulting from all loads factored by $\alpha \times \gamma_o$, again taking P-delta effects on an elastic basis into account. This step is performed by taking the stored moments resulting from all loads factored by 1 0 and multiplying them by $\alpha \times \gamma_o$. The P-delta effects are taken into account by adjusting the moments resulting from horizontal forces using magnification factor $f_m = [1/(1-\alpha Po/Po)]$ i.e. $f_m = 1/(1-\alpha)$.

- Use the resulting moments to calculate $\lambda_g$, the member slenderness ratio for the "limiting frame", at the critical sections. In the program $\lambda_g$ is calculated at the same cross sections as $\lambda_{0.4H}$ described above. The actual slenderness ratio of the members, $\lambda = L/r$ is then calculated at the same cross sections, where L = the length of the member and r = the radius of gyration of the member.

- Determine the lowest ratio $\lambda/\lambda_g$

- Calculate $(\alpha\gamma_o/\gamma_p)$ and $(\alpha\gamma_o/\gamma_p)_g$. To calculate the value $(\alpha\gamma_o/\gamma_p)_g$ we can either multiply $(\alpha\gamma_o/\gamma_p)$ by $\lambda/\lambda_g$, as derived by Scholz (1981), or we can use the value $\lambda_g$ to re-proportion the frame and repeat the calculations for $\gamma_p$ and $\gamma_o$. The first option is an approximation and

is obviously much quicker. The version of the program presented in Appendix B however adopts the second, more accurate option. The difference in using either the first or second options never resulted in a difference in the failure load of more than 2.5% in any of the examples analysed in this project report.

- Enter the Scholz interaction curves with the values $(\alpha\gamma_o/\gamma_p)$ and $(\alpha\gamma_o/\gamma_p)_k$ and read off the value for $\gamma_f$. If the value of $(\alpha\gamma_o/\gamma_p) > 1.5$ then the program uses the relationship $\gamma_f/\gamma_p = 1/(1+(i_s\gamma_p/\alpha\gamma_o))$ to calculate $\gamma_f$, where $i_s = 0.76[0.92 - (\alpha\gamma_o/\gamma_p)_k]$. This is a straight line approximation to the interaction curves of Fig.1 and is also presented by Scholz (1981). Where $(\alpha\gamma_o/\gamma_p) < 1.5$ the program makes use of a table of data to obtain the value of $\gamma_f$ by interpolation.

## 3.8 PRINTOUT OF RESULTS

After calculating $\gamma_f$ for a storey the program prints out the values of $\gamma_p$, $\gamma_o$ and $\gamma_f$ for that storey. If there are more storeys to analyse the program then repeats the procedure for the next storey. When $\gamma_f$ for all storeys is known the program compares the failure load factors $\gamma_f$ of all the storeys. The lowest $\gamma_f$ is identified and printed out as the suggested value for the failure load factor of the entire structure.

## 3.9 GRAPHICS

Once the computations are complete the program offers a graphical representation of the storey rigid-plastic collapse mode. The program gives a graphical printout for the selected storey or storeys, plotting the hinges on the structure and numbering the hinges to give the sequence of hinge formation.

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# 4.0 INTERACTION METHOD EVALUATION

## 4.1 APPLICATION TO DESIGN

The work done in this research project has been aimed at developing an analysis technique which can be used to facilitate and optimise the design of multi-storey sway frames. As mentioned earlier, the design of a multi-storey frame would be made much simpler if it could be designed on a storey-by-storey basis. Each storey could then be isolated and designed independently of the rest of the structure. The design procedure envisaged would be as follows:

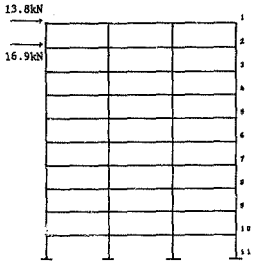
- Identify the design load factor required by the relevant design code.
- Use any simplified method to obtain trial member sizes for the whole structure. The most suitable approach would possibly be an elastic analysis and plastic design such as embodied in load and resistance factor or ultimate limit state design, ignoring effective length or alternatively, a rigid-plastic analysis and design.
- Analyse the structure on a storey-by-storey basis using the programmed interaction method and identify which storeys have a failure load factor lower than or excessively higher than the desired design load factor.
- Take each storey in turn and adjust a member or members until the storey load factor is within acceptable limits above the design load factor.

- The lowest storey failure load factor must be equal to or greater than the design load factor as the structure failure load factor is now taken as the lowest storey failure load factor.

This method thus provides us with an extremely simple solution to a complex problem. The whole structure can be optimised in a way which is virtually impossible using full frame analysis techniques. In a full frame analysis adjusting any one member in the structure might have repercussions anywhere else in the structure and thus it is impossible to identify exactly which members to modify in order to optimise the whole structure. Furthermore, each adjustment would require re-analysis of the entire structure to check what the consequences are for the rest of the structure. This is clearly impractical as a design approach.

Having optimised the structure by optimising each storey an important question arises: Does the fact that each storey can maintain the required design load factor imply that the structure as a whole is safely designed to this load factor? How representative is the behaviour of a structure analysed on a storey-by-storey basis of the behaviour of the framework as a whole?

To investigate these questions it was decided to analyse a trial frame on a storey-by-storey basis and then check the design using a rigorous second-order elastic-plastic method. The frame chosen was a three bay, ten storey high frame (Fig.5), which was previously designed by Cheong Siat-Moy (1976a) on a storey-by-storey basis. Cheong Siat-Moy designed the frame first for a load factor of 1.7 under gravity loads only and then adjusted the members to satisfy a load factor of 1.3 under combined gravity and wind loading. The member sizes arising out of his design for the loading indicated are shown in Fig.5. The results of the structure analysed by the storey-by-storey method are shown in the lower half of Fig.5




| Columns | | |
|---|---|---|
| Storeys | Int. | Ext. |
| 1-3 | 8W13 | 8W13 |
| 3-5 | 8W20 | 8W20 |
| 5-7 | 8W28 | 8W28 |
| 7-9 | 8W35 | 8W35 |
| 9-11 | 8W40 | 8W40 |

| Storeys | Beams |
|---|---|
| 1-2 | 12W22 |
| 3-4 | 14W22 |
| 5-8 | 14W26 |
| 9-10 | 16W26 |

Roof load = 20.43 kN/m
Floor load = 23.94 kN/m
Wind load = 5.84 kN/m height
Exterior column wall load = 42.25 kN/storey
Storey height = 2.896 m
Bay width = 6.096m

E = 200.8 GPa
fyb = 250.0 MPa
fyc = 345.0 MPa

Analysis using storey-by-storey interaction method:

Storey Collapse Load Factors :

| Storey | Plastic | Elastic | Elastic/Plastic |
|---|---|---|---|
| 1 | 2.17 | 32.12 | 2.11 |
| 2 | 1.95 | 13.27 | 1.81 |
| 3 | 2.03 | 12.93 | 1.86 |
| 4 | 1.83 | 9.36 | 1.63 |
| 5 | 1.79 | 9.69 | 1.58 |
| 6 | 1.72 | 7.96 | 1.49 |
| 7 | 1.64 | 7.60 | 1.41 |
| 8 | 1.56 | 6.59 | 1.32 |
| 9 | 1.50 | 6.95 | 1.28 |
| 10 | 1.44 | 11.11 | 1.28 |

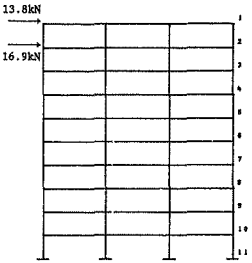
The Structure Collapse Load Factor = 1.28

Figure 5. Comparison with frame designed by Cheong Siat-Moy for load factor of 1.3 under combined loading.

From these results one can see that the uppermost storeys have a load factor against collapse greatly in excess of 1.3. This is no doubt as a result of the initial design under gravity loads only to a load factor of 1.7. In the uppermost storeys, where the second order effects are minimal, the gravity loads only design requirements would predominate. The two bottom storeys were found to be critical and failed at a load factor of 1.28. This means that our analysis produces a result slightly on the conservative side. It is clear from the storey failure loads shown in Fig.5 that the members in the uppermost storeys could be greatly optimised if we were to design only for the combined loading case with a load factor of 1.3.

As a test for the proposed design method, each storey was taken one at a time and by adjusting the beam member sizes optimised to as near as possible to a load factor of 1.3. For simplicity all the beams in any one storey were kept the same and the column members remained the same as those selected by Cheong Siat-Moy. The member sections used are American standard sections and because real sections were used it was not possible to obtain a failure load factor of exactly 1.3 for each storey. The member sizes of the optimised structure are shown in Fig.6 together with the storey failure load factors. Storey No.5 has the lowest failure load factor and therefore this value of 1.26 is taken as the structure failure load factor.

The optimised frame of Fig.6 was subsequently analysed on the basis of a rigorous elastic-plastic method using the program developed by Professor van Rensburg at the University of Pretoria. The full frame analysis from this program suggests a failure load factor of 1.27 for the entire frame. Hence the storey-by-storey approach using the interaction graphs is slightly on the conservative side with its failure load factor of 1.26.




All columns are identical to those shown in Fig.5

| Storey | Beams |
|---|---|
| 1 | 10B15 |
| 2 | 12B16.5 |
| 3 | 8W24 |
| 4 | 12B19 |
| 5 | 10M22.9 |
| 6 | 12W22 |
| 7 | 14W22 |
| 8 | 16W26 |
| 9 | 16W26 |
| 10 | 16W50 |

Roof load = 20.43 kN/m
Floor load = 23.94 kN/m
Wind load = 5.84 kN/m height
Exterior column wall load = 84.50 kN/storey
Storey height = 2.896 m
Bay width = 6.096 m

E = 200.8 GPa
fyb = 250.0 MPa
fyc = 345.0 MPa

**Analysis using storey-by-storey interaction method:**

Storey Collapse Load Factors :

| Storey | Plastic | Elastic | Elastic/Plastic |
|---|---|---|---|
| 1 | 1.33 | 21.98 | 1.29 |
| 2 | 1.39 | 10.42 | 1.29 |
| 3 | 1.45 | 7.26 | 1.30 |
| 4 | 1.47 | 6.92 | 1.29 |
| 5 | 1.48 | 5.81 | 1.26 |
| 6 | 1.53 | 2.70 | 1.28 |
| 7 | 1.51 | 6.16 | 1.29 |
| 8 | 1.53 | 6.77 | 1.30 |
| 9 | 1.50 | 6.38 | 1.27 |
| 10 | 1.39 | 11.68 | 1.28 |

**The Structure Collapse Load Factor = 1.26**

Figure 6. Frame optimised for a collapse load factor = 1.26

This example illustrates that it appears to be feasible to design a multi-storey framework on a storey-by-storey basis using the program outlined in this paper. In the following sections comparisons using the developed program have been made with the results of other analytical research and where discrepancies arise they are discussed. The points illustrated by these discussions should be taken into account in making an appraisal of the applicability of this method for general use.

## 4.2 SINGLE BAY FRAMES

### 4.2.1 EXAMPLE 1:

Vijakkhana, Nishino and Lee (1974) applied their second-order, elastic-plastic method to some single bay frames to demonstrate its applicability. Their example No.3 illustrates the application of their method to the design of an unbraced frame.

Their design process is similar to that proposed using the storey-by-storey interaction method. The members are initially sized on the basis of a simple rigid-plastic analysis. The entire frame is then analysed for the collapse load factor and the sequence of hinge formation is plotted. In the analysis procedure the load factor $\gamma_q$, at which hinge q forms, is identified prior to adjusting the structure stiffness for this hinge. It is noted that two possible hinge patterns exist under $\gamma_q$, namely the hinge pattern before and the hinge pattern after the hinge q has formed. A critical load parameter $\gamma_c$ is evaluated for each of these patterns giving the load parameters at which the structure would collapse

assuming elastic analysis with the given hinge pattern. At the formation of each hinge the load factor $\gamma_q$ and load factor $\gamma_c$ before and after hinge formation, are printed out.

The values of $\gamma_c$ are then used as an indication as to which members need to be adjusted to attain the required load factor against collapse. If at the formation of a hinge there is a significant reduction in the value of $\gamma_c$ it implies that the member in which this hinge forms contributes significantly to the overall stiffness of the frame.

The next step in the design procedure is thus to identify the members in which hinges cause a relatively large drop in the value of $\gamma_c$. These members would then be adjusted more or less proportionately to the magnitude of the change in $\gamma_c$ in an attempt to arrive at the required collapse load factor. Once this has been done the entire structure is re-analysed to ascertain what results the member adjustments have brought about.

Fig.7(a) shows the frame dimensions, members and loading as chosen by Vijakkhana et al. to demonstrate their design. The member sizes were selected on the basis of a preliminary design based on a simple rigid-plastic analysis. The frame is then analysed using the second-order, elastic-plastic method and found to have a failure load factor of 1.264. The sequence of hinge formation is shown in Fig.7(b). Examining the change in $\gamma_c$ for each of the 6 hinges it was found that for hinges no.1 and 4 the change in $\gamma_c$ is substantially greater than the corresponding changes at the formation of the other hinges. It is therefore deduced that either beam A3-B3 or column A3-A4 should be adjusted in order to bring up the failure load factor to the required 1.30. Column A3-A4 is increased to a 10W54 section and likewise column B3-B4, to preserve symmetry. Analysis of the revised structure shows it to have a failure load factor of 1.33.

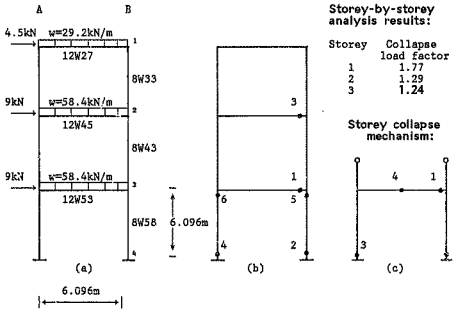



Figure 7. Comparison with frame design by Vijakkhana et al.(1974)

Using the presented program the frame shown in Fig.7(a) was re-analysed for comparison. The results obtained from this analysis are shown in Fig.7(c) together with the sequence of hinge formation for storey no.3. This storey, as we can see from Fig.7(c), is the critical storey, with a collapse load factor of 1.24. This value is 2% below that obtained by Vijakkhana et al.

It is obvious too, from the collapse mechanism of this storey, that beam 3 or the lower columns should be adjusted to increase the storey failure

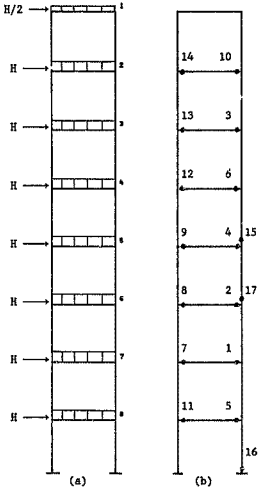
load factor. Following Vijakkhana et al., the lower columns are increased to 10W54 sections and storey No.3 re-analysed. The failure load factor for storey No.3 now becomes 1.30 which, assuming the failure load factor of 1.29 for storey No.2 is adequate, gives the structure the required load factor against failure. The result is again within 3% on the conservative side of the second-order, elastic-plastic analysis result.

This example illustrates the efficiency of the interaction method applied to the design situation. Being a single bay, three storey high frame, it does not highlight the drawback of the full frame analysis, namely that the whole structure has to be re-analysed once a member adjustment is made. [illegible] becomes more severe the larger the frame being analysed. The storey-by-storey approach thus gives an accurate result while at the same time cutting down on a great deal of analysis time.

### 4.2.2 EXAMPLE 2:

The second single bay frame on which a comparative analysis was done is an eight storey, one bay frame analysed by Korn and Galambos (1968). The frame dimensions, members and loading are shown in Fig.8(a) and is labelled Frame 8-1 by Korn and Galambos in the referenced paper. They refer to the arrangement of weak beams and strong columns in this frame as an extremely "weak beam design". Using their rigorous second-order, elastic-plastic analysis the frame was found to fail by instability at a load factor of 1.41. The sequence of hinge formation is shown in Fig.8(b).

Using the storey-by-storey interaction method a failure load factor of 1.03 was obtained for the same structure. The storey failure load factors are given in Fig.8(c) together with a typical storey plastic collapse




**Storey-by-storey analysis:**

| Storey | Collapse load factor |
|---|---|
| 1 | 3.25 |
| 2 | 1.80 |
| 3 | 1 34 |
| 4 | 1.41 |
| 5 | 1.33 |
| 6 | 1.23 |
| 7 | 1.03 |
| 8 | 2.18 |

**Typical storey collapse mechanism:**




| Storey | Columns | Beams |
|---|---|---|
| 1 | 6W15.5 | 8W17 |
| 2 | 8W31 | 8W20 |
| 3 | 8W31 | 10W21 |
| 4 | 8W35 | 12W27 |
| 5 | 10W49 | 14W30 |
| 6 | 12W79 | 14W34 |
| 7 | 14W87 | 14W34 |
| 8 | 14W95 | 14W38 |

Roof load = 19.6 kN/m
Floor load = 39.2 kN/m
Storey height = 3.048 m
Bay width = 3.048 m
Wind load: H = 21.92 kN

E = 200.8 GPa
$f_{yb}$ = 235.5 MPa
$f_{yc}$ = 235.5 MPa

Figure 8. Comparison with frame analysed by Korn and Galambos (1968)

mechanism. This result under-estimates the actual capacity by 20%. The reason for the great discrepancy in results can be found by examining the structure collapse mechanism of Fig.8(b) and the storey collapse mechanism of Fig.8(c). If a storey is being analysed in isolation with hinges at mid-storey height, then a hinge in the windward and leeward ends of the beam creates a storey collapse mechanism. From Fig.8(b) it can be seen that almost all the beams had windward and leeward hinges before a collapse mechanism formed. Using the storey-by-storey approach, collapse is prematurely registered when the first beam acquires its windward hinge.

This is a problem inherent in the storey-by-storey approach which is bound to arise when a frame proportioned on a "weak beam" or "weak column" basis is analysed. With a windward and leeward beam hinge forming in a storey with "weak beams", a check would have to be made with other storeys to see if one of them could combine with the storey under consideration to form a structure collapse mechanism. An under-estimation of the capacity would also result if hinges formed in two columns at the same level in a single bay frame. The columns at other levels and at the bases would have to be examined to see if they could combine and form the structure collapse mechanism. Having to do these checks would to a certain extent begin to nullify the greatest advantage of the storey-by-storey approach.

These problems however should not hinder us in using the storey-by-storey interaction method for the design of a structure unless we are explicitly aiming for an extremely "weak beam" or "weak column" design. If we were designing the structure just analysed to a load factor of say 1.41, then the storey collapse load factor of 1.03 in storey No.7 and its associated collapse mechanism would prompt us to increase the size of this beam. The storey would then be re-analysed and further adjustments made if necessary to the beams and column members to bring the storey collapse load factor as close to 1.41 as possible. In our example the columns could very likely be reduced in the process. The same procedure would then be repeated for

each storey of the frame. When the design adjustments have been completed, all the members in the structure would be close to failure at a load factor of 1.41 and no single storey would have a collapse load less than 1.41.

Thus in conclusion we can see that when used to compare analyses, the storey-by-storey method could give excessively conservative results. However if the storey-by-storey approach is used to design the structure, an analysis on the optimally designed frame would compare well with a second-order, elastic-plastic analysis. The initial conservative results are inconsequential when it is realised that the closer one gets to the optimum design, the more accurate the analysis results are. Frames 8-3, 8-4 and 15-1 analysed by Korn and Galambos (1968) were all sized using approximate design methods. The storey-by-storey method should thus give reasonable results for these frames when compared to the rigorous analysis results. This was borne out by the analysis of these three frames, where the capacities of the frames when analysed using the proposed method were all within 5% of the results obtained by Korn and Galambos.

## 4.3 STOREY SUB-ASSEMBLAGES

### 4.3.1 EXAMPLE 1:

The first comparison with a storey sub-assemblage model is taken from a series of Lehigh University lecture notes (1965) titled "Plastic Design of Multi-Story Frames". The storey sub-assemblage under consideration is shown in Fig.9, which gives the dimensions, loading and member sizes of

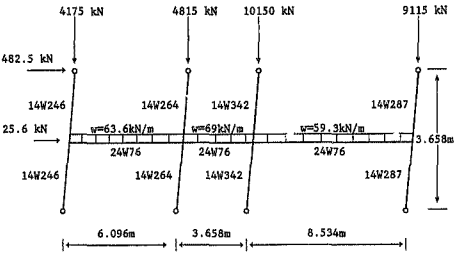
the arrangement. This sub-assemblage represents the twentieth storey of a twenty-four storey frame, labelled Frame C in the lecture notes. The beam and column elements for the entire Frame C have been sized using an approximate design method presented in the lecture notes.

In Lecture No.19 the storey sub-assemblage shown in Fig.9 is used to demonstrate an approximate analysis technique outlined in Lecture No.18. The members have been designed for a load factor of 1.70 under gravity loads only and 1.30 for combined gravity and wind loads. All gravity loads are now held constant at a load factor of 1.30 and the horizontal load is increased step-wise up till storey collapse. It would be expected that failure would occur at a load factor for horizontal loading of approximately 1.30.

Using their approximate analysis method the Lehigh authors found that the storey sub-assemblage failed at a load factor of 1.79. Thus at failure the applied loads were 1.30 times the gravity loading plus 1.79 times the wind load. This result suggests that the initial design erred on the conservative side.

A comparison between the Lehigh sub-assemblage analysis technique and the proposed storey-by-storey interaction method was then made. The computer program presented in this report increases the horizontal and vertical loads proportionally and this required some adjustment to be made to the loading to make a valid comparison. The gravity loads were applied unfactored i.e. working gravity loads were used. The horizontal load was applied with a factor of 1.79/1.3 = 1.38 times working load. This ensured that at a factor of 1.30 times the applied loads, the same loading would act on the sub-assemblage as those which caused failure in the Lehigh analysis.

**Storey sub-assemblage:**




Gravity loads unfactored ; Horizontal loads factored by 1.3 times service.

**Summary of analysis results:**

| Analysis method | Constant loads | Loads incremented by load factor | Failure load factor |
|---|---|---|---|
| Lehigh method | 1.3 gravity load | 1.0 wind load | 1.79 |
| Interaction method | — | 1.79/1.3 wind load<br>1.3 gravity load | 1.45 |
| Davison & Adams | 1.3 gravity load | 1.0 wind load | 2.02 |
| Interaction method | — | 2.02/1.3 wind load<br>1.0 gravity load | 1.35 |
| Interaction method on frame revised by Emkin & Litle | — | 1.0 wind load<br>1.0 gravity load | 1.34 |

Figure 9. Comparisons with Lehigh storey sub-assemblage analysis.

The result arising from the analysis of the storey sub-assemblage by the interaction method was failure at a load factor of 1.45. This is 12% over the expected result of 1.30. This implies that either the Lehigh analysis is conservative or that the proposed interaction method errs on the un-conservative side.

Davison and Adams (1974) analysed this same storey sub-assemblage using a second-order, elastic-plastic method. In their analysis the same loading was adopted as was used in the Lehigh analysis, namely constant vertical loads factored by 1.30 and horizontal loads incremented up to failure. This analysis confirmed that the Lehigh analysis is conservative. The storey sub-structure failed under a horizontal load of 2.02 times working load (compared to 1.79 in the Lehigh analysis).

The proposed interaction method was again employed to analyse the sub-structure, this time with 1.0 times the vertical loads and 2.02/1.3 = 1.55 times the horizontal working load. The result of this analysis was a failure load factor of 1.35 - only 4% above the expected value of 1.30. This slight over-estimation of the failure load could be attributed to the simplifications introduced into the computerised version of the proposed method. A further source of error could arise as a result of the different types of loading adopted in the different analysis methods. A more thorough study is required to determine the difference in failure loads when using proportional and non-proportional loading on an identical structure. From the results obtained in this study it would appear that the different applications of load could result in a small discrepancy.

It was mentioned earlier that the storey sub-assemblage under consideration was taken from a twenty-four storey frame, designed using an approximate method. Emkin and Litle (1970) re-designed this frame and changed the member sizes considerably. Their design was based on a rig-

orous second-order analysis with vertical and horizontal loads being applied simultaneously factored by the appropriate vertical and horizontal load factors of 1.3. The storey sub-assemblage was again analysed using the interaction method with the revised member sizes. The applied loading in this case was 1.0 times vertical and horizontal loads. The storey collapse load resulting from this analysis was 1.34 i.e. 3% over the design load factor of 1.30. The results of this analysis are presented in Appendix C as an illustration of the computer program output.

In this case the slight discrepancy could partly be attributed to the fact that real members were used in the design which would make it virtually impossible to choose members which would cause a failure load factor of exactly 1.30. This result thus affirms the accuracy of the proposed interaction method. A summary of the results of all the analyses done on the storey sub-assemblage is given in Fig.9 along with the structure dimensions, loading and section types.

### 4.3.2 EXAMPLE 2:

Scholz (1983a) used a two bay, storey sub-assemblage example from the same Lehigh Lecture Not [illegible]65) to demonstrate his interaction method applied to a sub-structure. The interaction method was applied by hand and the story failure load was found to be the same as that obtained by the Lehigh authors.

This example is discussed here because when analysed by the program presented in this paper a different result was obtained. Instead of a failure load of 1.30 for the structure the programmed version gave a storey

failure load of 1.25. The same interaction method was used in both analyses and hence should have yielded identical results.

On further investigation it was found that the collapse mechanism identified by the computer program was different to the hand calculated critical sway mechanism. The rigid-plastic collapse mechanism obtained by Scholz is shown in Fig.10(a) while Fig.10(b) shows the rigid-plastic mode of failure identified by the computer program. The mechanism in Fig.10(a) is an overall storey sway mechanism while Fig.10(b) shows a joint mechanism causing failure.

The joint mechanism causes failure only because hinge reversal is not taken into account by the computer program. With hinge No.2 in the leeward end of the beam (Fig.10(b)) and hinge No's.5 and 3 in the upper and lower columns respectively it becomes impossible for the sub-assemblage to carry any more load. The reason for this is that if the column axial load were to increase, the upper and lower columns' plastic moment capacities would be reduced and would no longer be capable of balancing the beam end moment, as required for joint equilibrium.

If the beam hinge were able to reverse direction at this stage the beam end moment would "unload" to below its plastic moment capacity, transferring some of the end moment into the span and the other end. The columns would then be able to resist a greater axial load while still balancing the beam end moment. The storey sub-structure could then be loaded further until a full sway mechanism forms.

Failure to take hinge reversals into account thus resulted in a slightly conservative result (4% on the conservative side). A similar result was obtained by Vijakkhana, Nishino and Lee (1974) when comparing the ultimate loads of a one bay, three storey frame analysed with and without consid-

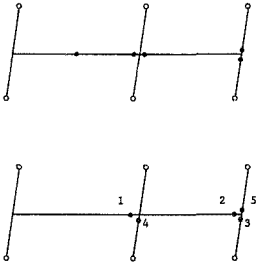



Figure 10. Different failure mechanisms for 2 bay, storey sub-assemblage.

eration of hinge reversals. The analysis done without taking hinge reversals into account produced a slightly lower ultimate load factor (1% less) than when hinge reversals were taken into account.

Although the discrepancy in ultimate loads is small, Vijakkhana et al. note that the deformed shape, the plastic hinge pattern and the mode of sway mechanism can be quite different if hinge reversals are neglected. The proposed program using the interaction method therefore could be improved by taking into consideration the effects of hinge reversals. However, the question of whether the added complexity would justify the improvement, is a subject which should be more fully investigated.

# 5.0 CONCLUSIONS

## 5.1 COMPUTER PROGRAM

Scholz's Interaction method can easily be computerised for any modern desk-top micro computer. The problem of finding an analysis which could calculate the rigid-plastic collapse load has been overcome. The matrix method used to make this calculation is greatly simplified by adopting the development proposed by Liang and Yuan (1984) and by applying it to the storey model with hinges at mid-column height.

Furthermore, a considerable saving in computer time is achieved over rigorous elastic-plastic methods due to the fact that no axial forces in the form of stability functions appear in the stiffness matrix. Because of this, rigorous elastic-plastic analysis requires approximately five to seven iterations within each load increment compared to a single matrix evaluation for each load level in the presented technique. Thus, assuming for instance, that ten load increments are investigated between the commencing load level and failure, forty to sixty matrix evaluations are avoided i.e. a saving of at least 80%. In addition, since the presented method is performed on one storey at a time, a much smaller matrix is involved compared with the complex matrix of the entire structure.

## 5.2 APPLICATION TO DESIGN

The design example done on the ten storey frame presented in Section 4.1 shows clearly that Scholz's method is extremely suited for the design of large multi-storey sway frames.

The Interaction method was applied here on a storey-by-storey basis. The failure load of each storey indicated in which storey members had to be adjusted to satisfy the required design load factor. In this example only the beam members were adjusted for simplicity. If optimisation of each individual member was required, a figure showing the plastic collapse mechanism and sequence of hinge formation could be displayed to assist in identifying the members which needed changing. The storey failure load could be adjusted in this way without affecting the rest of the structure. This provides us with a very efficient design method.

The simplifications introduced in the computerised version of the interaction method greatly reduce the complexity of the problem. The results of this report suggest that these simplifications in no case caused a discrepancy of greater than 5% in the predicted failure loads when compared with the results of other more complex analysis methods.

The program was applied in the design of only one full frame in this study. To substantiate these results, additional similar exercises should be performed on other frames. A good starting point on any future work in this direction would be to take the frame designed by Emkin and Litle (1970) and see if this program could be used to further optimise the design. A rigorous, second-order analysis would then have to be performed on the frame to assess the validity of the design.

## 5.3 ANALYSIS

It was shown in Section 4.2.2 that this method can not be applied unconditionally on a storey-by-storey basis for an accurate analysis. If the structure is so proportioned that it contains a number of "weak" beams or columns, applying the method on a storey-by-storey basis can give an underestimation of the structure capacity. However, wher the structure is optimally proportioned, the interaction method can accurately predict the failure load wh. applied storey-by-storey.

## 5.4 RECOMMENDED IMPROVEMENTS

**Hinge reversals:** By not providing for hinge reversals the program identified the incorrect rigid plastic failure mechanism for the storey sub-structure analysed in Section 4.3.2. This error resulted in only a small under-estimation of the failure load. Although the discrepancy was small and on the conservative side, an effort should be made to take hinge reversals into consideration. To avoid unnecessary complication, the method of Davison and Adams (1974) should be followed here, where provision is made for hinge reversals in beam members only. A more thorough study could be made into the effects of hinge reversals to see if the benefits gained from the provision for hinge reversals justify the additional complexity introduced into the computations.

**Other second-order effects:** The literature indicates that second-order effects other than the P-Delta effects need not be considered in the design of multi-storey frames. If evidence to the contrary is forwarded, the program could be easily modified to take these effects into account.

Scholz (1983b) has outlined how these second-order effects can be accommodated in the proposed interaction method.

**Non-proportional loading:** The program presented in this dissertation factors both horizontal and vertical loads proportionally by the same load factor. It would be a simple matter to modify the program to accommodate different vertical and horizontal load factors. The interaction method however has initially been derived using proportional loading, and its validity under non-proportional loading should be thoroughly examined. The result of this investigation could have important consequences for the suitability of the method for design applications which require non-proportional loading e.g. earthquake design.

**Application to 3-D frameworks:** The presented program could be extended to cover the analysis of three dimensional, asymmetric frameworks made up of frames on an intersecting rectangular grid and subjected to torsion. Such an analysis is very complex if full scale rigorous methods are employed. Scholz (1983b) has shown that in principle the interaction method can be applied to such problems.

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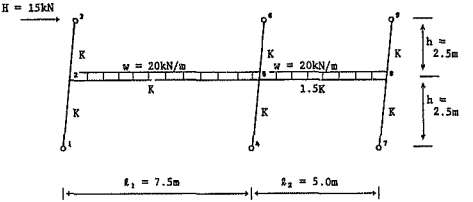
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# APPENDIX A

## APPENDIX A. WORKED EXAMPLE




K = member stiffness (EI/ℓ)
Mpb = plastic moment capacity of beams = 312kNm.
Mpu = plastic mom. capacity upper cols = 104kNm.
Mpl = plastic mom. capacity lower cols = 100kNm.

**Member stiffness matrices:**

Member 1-2 (sim. members 4-5, 7-8):

$$\{K\}\{Df\} = \begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & 3/h^2 & -3/h \\ 0 & 0 & -3/h & 3 \end{bmatrix} K \begin{Bmatrix} U_1 \\ \theta_1 \\ U_2 \\ \theta_2 \end{Bmatrix}$$

Member 2-3 (sim. members 5-6 & 8-9) :

$$\{K\}\{Df\} = \begin{bmatrix} 3/h^2 & 3/h & -3/h^2 & 0 \\ 3/h & 3 & -3/h & 0 \\ -3/h^2 & -3/h & 3/h^2 & 0 \\ 0 & 0 & 0 & 0 \end{bmatrix} K \begin{Bmatrix} U_2 \\ \theta_2 \\ U_3 \\ \theta_3 \end{Bmatrix}$$

Member 2-5 :

$$[K]\{Df\} = \begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & 4 & 0 & 2 \\ 0 & 0 & 0 & 0 \\ 0 & 2 & 0 & 4 \end{bmatrix} K \begin{Bmatrix} V_2 \\ \theta_2 \\ V_5 \\ \theta_5 \end{Bmatrix}$$

Member 5-8 :

$$[K]\{Df\} = \begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & 6 & 0 & 3 \\ 0 & 0 & 0 & 0 \\ 0 & 3 & 0 & 6 \end{bmatrix} K \begin{Bmatrix} V_5 \\ \theta_5 \\ V_8 \\ \theta_8 \end{Bmatrix}$$

**Structure stiffness matrix (No hinges):**

| | stiffness | | | | | | defl. | forces |
|---|---|---|---|---|---|---|---|---|
| $F_2$: | 2.88 | | -1.44 | | | | U | 0 |
| $M_2$: | | 10 | -1.20 | 2 | | | $\theta_2$ | 4.688w |
| $F_5$: | -1.44 | -1.20 | 1.44 | -1.20 | -1.20 | K | $U_5$ = | H |
| $M_5$: | | 2 | -1.20 | 16 | 3 | | $\theta_5$ | -2.605w |
| $M_8$: | | | -1.20 | 3 | 12 | | $\theta_8$ | 2.083w |

det K = |K| = 2190

| $\gamma$ = 1.0 | $\gamma$ = 1.1 |
|---|---|
| $KU_2$ = 22.74 | $KU_2$ = 25.01 |
| $KU_5$ = 45.46 | $KU_5$ = 50.00 |
| $K\theta_2$ = 15.24 | $K\theta_2$ = 16.76 |
| $K\theta_5$ = -2.04 | $K\theta_5$ = -2.24 |
| $K\theta_8$ = 1.58 | $K\theta_8$ = 1.74 |

The first plastic hinge was found to form in beam 2-5 at end 5. Only the computations for this beam are presented here:

$M_{s2} = 2K\theta_2 + 4K\theta_5 + w\ell^2/12$
$= 116.08$ kNm

$M_{s2} = 127.70$ kNm

$$\gamma_1 = \frac{1.1 - 1.0}{127.7 - 116.08} * 312 + \frac{1.0 * 127.7 - 1.1 * 116.08}{127.7 - 116.08} = 2.69$$

With a hinge at end 5 the member stiffness matrix for beam 2-5 becomes:

$$[K]\{D\} = \begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & 3 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \end{bmatrix} \times \begin{Bmatrix} V_2 \\ \theta_2 \\ V_5 \\ \theta_5 \end{Bmatrix}$$

**Revised structure stiffness matrix (1 hinge) :**

$$\begin{array}{l} F_2: \\ M_2: \\ F_3: \\ M_5: \\ M_8: \end{array} \begin{bmatrix} 2.88 & & -1.44 & & \\ & 9 & -1.20 & & \\ -1.44 & -1.20 & 1.44 & -1.20 & -1.20 \\ & & -1.20 & 12 & 3 \\ & & -1.20 & 3 & 12 \end{bmatrix} K \begin{Bmatrix} U_2 \\ \theta_2 \\ U_3 \\ \theta_5 \\ \theta_8 \end{Bmatrix} = \begin{Bmatrix} 0 \\ 7.031w\text{-}Mpb/2 \\ H \\ 2.083w\text{-}Mpb \\ -2.083w \end{Bmatrix}$$

det $|K| = 1288$

Hinge no.2 forms in column 5-4 at 5.

With $\bar{x} = 2.69$ : $M_{54} = -89.8$ kNm
$\bar{x} = 1.1*2.69$ : $M_{54} = -94.4$ kNm

$$x_2 = \frac{2.96 - 2.69}{-95.0 + 89.8} * -100 + \frac{2.69 * -95.0 + 2.96 * 89.8}{-95.0 + 89.8} = 3.22$$

Revised member stiffness matrix for column member 5-4 :

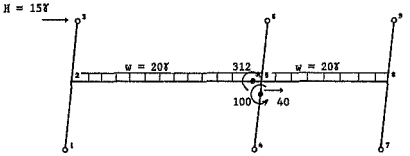
$$[K]_{54} = [0]$$

**Revised structure stiffness matrix (2 hinges):**

$$\begin{array}{l} F_2: \\ M_2: \\ F_3: \\ M_5: \\ M_8: \end{array} \begin{bmatrix} 2.40 & & -1.44 & 1.20 & \\ & 9 & -1.20 & & \\ -1.44 & -1.20 & 1.44 & -1.20 & -1.20 \\ 1.20 & & -1.20 & 9 & 3 \\ & & -1.20 & 3 & 12 \end{bmatrix} K \begin{Bmatrix} U_2 \\ \theta_2 \\ U_3 \\ \theta_5 \\ \theta_8 \end{Bmatrix} = \begin{Bmatrix} 0\text{-}Mp1/h \\ 7.031w\text{-}156 \\ H \\ Mp1\text{-}312+2.083w \\ -2.083w \end{Bmatrix}$$

det $|K| = 599.5$

**Diagrammatic representation :**




The third hinge forms in column member 5-6 at 5 :

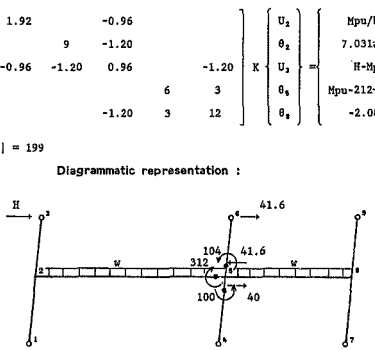
Load at formation of third hinge : $Y_3 = 3.36$

**Structure stiffness matrix revised for 3rd hinge:**

$$\begin{matrix} F_2: \\ M_2: \\ F_3: \\ M_5: \\ M_6: \end{matrix} \begin{bmatrix} 1.92 & & -0.96 & & & \\ & 9 & -1.20 & & & \\ -0.96 & -1.20 & 0.96 & & & -1.20 \\ & & & 6 & 3 & \\ & & -1.20 & 3 & 12 & \end{bmatrix} K \begin{Bmatrix} U_2 \\ \theta_2 \\ U_3 \\ \theta_5 \\ \theta_6 \end{Bmatrix} = \begin{Bmatrix} Mpu/h-40 \\ 7.031w-156 \\ H-Mpu/h \\ Mpu-212+2.083w \\ -2.083w \end{Bmatrix}$$

$\det |K| = 199$

**Diagrammatic representation :**




4th hinge forms in member 7-8 at 8 : $Y_4 = 3.48$

5th hinge forms in member 8-9 at 8 : $Y_5 = 3.56$

6th hinge forms in member 2-5 at midspan : $Y_6 = 3.77$

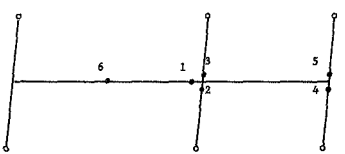
**Structure stiffness matrix revised for 6 plastic hinges:**

$$[K] = \begin{bmatrix} 0.96 & & -0.48 & & & \\ & 6 & -1.20 & & & \\ -0.48 & -1.20 & 0.48 & & & \\ & & & 6 & 3 & \\ & & & 3 & 6 & \end{bmatrix}$$

$\det |K| = 0 \Rightarrow$ structure collapse mechanism has formed and $\gamma = \gamma_p$

i.e. $\gamma_p = 3.77$

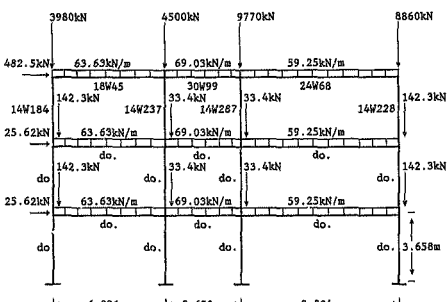
**Sequence of plastic hinge formation:**



# APPENDIX B

## APPENDIX B. COMPUTER PROGRAM PRINTOUT

This example is presented here to illustrate the capabilities of the computer program. The structure geometry, member sizes and loading are shown in the diagram below. The frame is analysed by the program and the printout is given on the following pages.




Young's Modulus: E = 199.8 GPa

Yield stress, beams: fyb = 248.2 MPa

Yield stress, columns: fyc = 344.5 MPa

RUN DESCRIPTION: FRAME C : LITLE & EHKIN (Program COLLAPSE2)

Young's Modulus =198000000 kN/m2
Material Yield Stresses: Beams=240200kN/m2 ; Columns=344500kN/m2

Beam Properties

| Storey | Beam | Length(m) | I(m4) | zp(m3) | Area(m2) | y_max(m) |
|---|---|---|---|---|---|---|
| 1 | 1 | 6.096 | .000293 | .001468 | .000541 | .227 |
| | 2 | 3.658 | .001860 | .005113 | .019779 | .376 |
| | 3 | 8.534 | .000755 | .002876 | .012902 | .302 |
| 2 | 1 | 6.096 | .000293 | .001468 | .000541 | .227 |
| | 2 | 3.658 | .001860 | .005113 | .019779 | .376 |
| | 3 | 8.534 | .000755 | .002876 | .012902 | .302 |
| 3 | 1 | 6.096 | .000293 | .001468 | .000541 | .227 |
| | 2 | 3.658 | .001860 | .005113 | .019779 | .376 |
| | 3 | 8.534 | .000755 | .002876 | .012902 | .302 |

Column Properties

| Storey | Col. | Axial load(kN) | I(m4) | zp(m3) | Area(m2) | y_max(m) |
|---|---|---|---|---|---|---|
| 1 | 1 | 3986.00 | .000947 | .005531 | .034881 | .195 |
| | 2 | 4500.00 | .001282 | .007299 | .044957 | .205 |
| | 3 | 9770.00 | .001628 | .009039 | .054432 | .213 |
| | 4 | 8860.00 | .001225 | .007000 | .043260 | .203 |
| Storey height = 3.658m | | | | | | |
| 2 | 1 | 142.30 | .000947 | .005531 | .034881 | .195 |
| | 2 | 33.40 | .001282 | .007299 | .044957 | .205 |
| | 3 | 33.40 | .001628 | .009039 | .054432 | .213 |
| | 4 | 142.30 | .001225 | .007000 | .043260 | .203 |
| Storey height = 3.658m | | | | | | |
| 3 | 1 | 142.30 | .000947 | .005531 | .034881 | .195 |
| | 2 | 33.40 | .001282 | .007299 | .044957 | .205 |
| | 3 | 33.40 | .001628 | .009039 | .054432 | .213 |
| | 4 | 142.30 | .001225 | .007000 | .043260 | .203 |
| Storey height = 3.658m | | | | | | |

Structure Loading

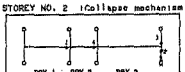
| Storey | Hor.load(kN) | Beam | Udl(kN/m) |
|---|---|---|---|
| 1 | 482.58 | 1 | 63.63 |
| | | 2 | 69.03 |
| | | 3 | 59.25 |
| 2 | 25.62 | 1 | 63.63 |
| | | 2 | 69.03 |
| | | 3 | 59.25 |
| 3 | 25.62 | 1 | 63.63 |
| | | 2 | 69.03 |
| | | 3 | 59.25 |

Elastic/Plastic Collapse Load Factors

| Storey No. | Plastic | Elastic | Elastic/Plastic |
|---|---|---|---|
| 1 | 1.64 | 13.02 | 1.47 |
| 2 | 1.57 | 8.42 | 1.34 |
| 3 | 1.50 | 17.95 | 1.39 |

STRUCTURE COLLAPSE LOAD FACTOR = 1.34

STOREY NO. 2 :Collapse mechanism



# APPENDIX C

## APPENDIX C. PROGRAM DESCRIPTION

### C.1 PROGRAM OUTLINE

| | | |
|---|---|---|
| Lines | 5 - 1000 | Declaration and description of variables. |
| | | Main body of program. |
| | | Editing, re-run and end options. |
| | 1005 - 2000 | Input of structure dimensions and loading. |
| | 2000 - 2275 | Stiffness Matrix [K] |
| | 3000 - 3620 | Force Vector (F) |
| | 4000 - 4580 | Calculation of beam and column moments. |
| | 5000 - 5880 | Identification of Next Hinge. |
| | 6000 - 6440 | Printout of results. |
| | 7000 - 7160 | Calculation of Elastic Buckling load. |
| | 8000 - 8665 | Application of Scholz Interaction method to calculate storey Failure load. |
| | 9000 - 10205 | Graphical printout of collapse mechanism. |

### C.2 VARIABLE DESCRIPTIONS

The variables used in the program listing are discussed here in more detail. Lines 40 - 205 declare all the real and integer variables and give a brief explanation of their function in the program. A few of these variables may require clarification :

| Line | Variable | Description |
|---|---|---|
| 75 | X | X = 0 for first iteration with $\gamma = 1.0\gamma_p$ |
| | | X = 1 for second iteration with $\gamma = 1.1\gamma_p$ |
| 80 | Cycle | Cycle = 1 for calculations on actual structure. |
| | | Cycle = 2 for calculations on "limiting frame". |
| 90 | Ks | Ks = EI/ℓ |
| 140 | FEM2hl | For beam initially with 1 hinge at RHS ; |
| | | FEM2hl = moment at LHS when midspan hinge forms. |

Lines 220 - 270 declare all the arrays which are used in the input stage and in the course of the computations. A brief description of the function of the array is given. The array is named, its maximum size is given in brackets, followed by the array description. The arrays are so dimensioned to accommodate a structure of maximum fifty storeys by twenty bays wide.

| | |
|---|---|
| Bays (50) | Keeps record of the number of bays in each storey |
| Hinges (50) | Record of the number of hinges in the collapse mechanism of each storey. |
| Bcondition (50,20) | Record of the number of hinges in each beam |
| | No hinges: Bcondition = 0 |
| | 1 hinge (LHS): = 10 |
| | 1 hinge (RHS): = 1 |
| | 2 hinges (LHS + midspan): = 20 |
| | 2 hinges (RHS + midspan): = 2 |
| | 2 hinges (LHS + RHS): = 11 |
| Gcondition (2,50,21) | Record of the number of hinges in each column |
| | No hinges: Gcondition = 0 |
| | 1 hinge (upper column): = 10 |
| | 1 hinge (lower column): = 1 |
| | 2 hinges: = 11 |
| Beamprop (5,50,20) | Records length, moment of inertia, plastic |

| | |
|---|---|
| | modulus, area and distance from centroid to extreme fibre for each beam section |
| Colprop (5,50,21) | Same as above for each column section |
| Load (50,21) | Records Udl for each beam and horizontal load for each storey |
| Thrust (50,21) | Records applied column axial loads |
| F (23) | The force vector {F} |
| Df (23) | Unknown deformations {Df} |
| K_inv_inv (23,23) | Arrays set up in the calculation to determine if det \|K\| = 0 |
| Differ (23,23) | |
| Compare (23,23) | |
| Compare1 (23,23) | |
| K (23,23) | The stiffness matrix [K] |
| Kt (23,23) | The transpose of the upper half of [K]. [K] is symmetric about the leading diagonal, therefore only the top half is assembled and then added to its transpose to get [K] |
| Kinv (23,23) | The inverse of [K] i.e. $[K]^{-1}$ |
| Beamom (3,51,21) | The calculated moments at the *LHS, midspan and* the RHS of each beam |
| Colmom (2,51,21) | The upper and lower column moments *for each storey* |
| Endmom (2,20) | The fixed end moments at the end of each beam for the loading and storey under *consideration* |
| Basemom (2,21) | Base moment of the bottom storey columns |
| Que$ [1] | Array to accept *answer* "Y" *or* "N" |
| Describe [80] | Array to record description of run |
| Axial (2,21) | Records *accumulative axial load on* each column for the storey under consideration |
| Hmombm (3,20) | Records the *moments calculated at* the LHS, midspan and the RHS of each beam in the storey *under consideration with service* horizontal loads only |

| | |
|---|---|
| Hmomcol (2,21) | Moments in upper and lower columns with service horizontal loads only |
| Hmombase (21) | Base loads with service horizontal loads only |
| Holdaxl (21) | Records axial loads on actual frame while loads are adjusted for the "limiting frame" calculations |
| Servmombm (3,20) | Records the moments calculated at the LHS, midspan and the RHS of each beam for the storey under consideration with service loads |
| Servmomcol (2,21) | Records column moments with service loads |
| Servmombase (21) | Records base moments with service loads |
| Curve (10,15) | Stores the data for Scholz Interaction curves from which the failure load is obtained by interpolation |
| Factor (50) | Records the failure load factor for each storey |
| Location (2,50,150) | Records the column or beam number and hinge location at the formation of each plastic hinge. This is used in the GRAPHICS sub-routine to plot the sequence of hinge formation in the plastic collapse mechanism |

## C.3 PROGRAM LISTING

The full listing of the program is given in the following pages:

```
!Program name is COLLAPSE
!-------------------------
!
!This program combines the Plastic and Elastic Collapse load factors to ob
tain the Overall Collapse Load Factor for the structure.
!The load factors are calculated on a Storey-by-Storey basis and the criti
cal load factor is the lowest Storey Load Factor.
!
OPTION BASE 1
INTEGER S,N                     !No. of storeys,No. of bays.
INTEGER Storey,Beam,Col,No      !Counters in subr. INPUT
INTEGER Dim                     !Redim. of storage arrays in subr. INPUT
INTEGER Beamno,Colno            !Location of plastic hinges in subr. NEXT HINGE
INTEGER Loc,Hingeloc            !Location of plastic hinge in subr. NEXT HINGE
INTEGER Hinge                   !Hinge no. counter in subroutine NEXT HINGE
INTEGER X                       !Variable for momentstorage in subr. MOMENTS
INTEGER Cycle                   !Denotes cycle 1 or 2
INTEGER Edit,Ed                 !Indicates initial input or editing
REAL L,Ib,Ic,Fpb,Mpc,H,Ks       !Length,Mom.Inertia,Plastic cap,Height in INPUT
REAL H1,H2                      !Half height lower cols., upper cols.
REAL A,B,C,D,U,G,M,J,P,D,R      !Intermediate results in subr. MATRIX CK)
REAL E                          !Young's modulus
REAL Fy,Fp,Py,Fac               !Yield stress and squash load calcs.
REAL Total,Test                 !Variables to check if DET K ))-conditioned
REAL Lambda,Lambdap             !Intermediate load factors in subr. NEXT HINGE
REAL Lambda1,Lambda2            !Load factor on 1st and 2nd iterations
REAL Vd1,Fh                     !Vertical and horizonal loads in subr. INPUT
REAL M1,Mr                      !FEM's a left and right
REAL Fem2h1,Fem2hr              !FEM's left and right at form. of 2nd hinge
REAL Hor,Hor1,Hor2,Shor         !Horizontal forces for calcs in  MATRIX (F)
REAL M1,M2,M3,M4,Mup            !Beam and Col. moments in subr. MOMENTS
REAL Mom1,Mom2,Mpl              !Moments 1st and 2nd iterations in NEXT HINGE
REAL Dmplastic,Colplastic       !Indicates hinge in beam or Col. in NEXT HINGE
REAL Mbase,Base                 !Column moments at base; Base=0 for fixed base
REAL Sum,Axbtal,Phi,Us,Subt     !Elastic collapse load in ELABTIC
REAL Peterey,Lambda,Meq         !Elastic collapse load in ELABTIC
REAL Lambda04,Bigmon,Alpha      !Identifying highest slenderness with 0.4H
REAL Slendratio,Ratio,Mtotal    !Identifying lowest slendernes ratio
REAL Alphal,Slendratio(         !Alpha and slend ratio for actual frame
REAL Io,Fail,Inter1,Inter2,Inter3   !Extracting values from curves
REAL Angle,Y_axis,X1,X2,Y1,Y2       !Extracting values from curves
REAL Axl                        !Axial load
!
!
INTEGER Bays(50),Hinges(50),Bcondition(50,20),Ccondition(2,50,21)
DIM Beamprop(5,50,20),Colprop(2,50,21)
DIM Load(50,21),Thrust(50,21),F(23),Gf(23)
DIM K_inv_inv(23,23),Differ(23,23),Comparei(23,23),Comparsi(23,23)
DIM K(23,23),Ki(23,23),Kinv(23,23)
DIM Beamm(2,51,21),Colmom(2,51,21),Endmom(2,20),Basemom(2,21)
DIM Due$[13,Describe$[80]
DIM Axial(2,21),Hmombm(3,20),Hmomcol(2,21),Hmombase(21),Holdaxl(21)
DIM Barvmombm(3,20),Barvmomcol(2,21),Barvmombase(21)
DIM Curve(10,15),Factor(50)
DIM Location(2,50,150)
!
!
DATA .09,.19,.27,.33,.40,.45,.49,.52,.54,.57,.61,.63,.65,.67,.68
DATA .10,.19,.28,.35,.41,.47,.52,.55,.59,.61,.64,.66,.69,.70,.71
DATA 0,.20,.29,.36,.43,.49,.54,.57,.61,.64,.66,.69,.70,.72,.73
DATA 0,0,.30,.38,.45,.51,.56,.60,.64,.67,.70,.72,.73,.75,.77
DATA 0,0,0,.40,.48,.55,.60,.64,.68,.71,.73,.75,.77,.79,.80
DATA 0,0,0,0,.50,.57,.63,.68,.72,.75,.77,.79,.81,.82,.83
DATA 0,0,0,0,0,.60,.67,.72,.76,.79,.82,.84,.85,.86,.87
DATA 0,0,0,0,0,0,0,.70,.76,.80,.83,.86,.88,.89,.90,.91
```

```
DATA 0,0,0,0,0,0,0,0,.86,.84,.87,.89,.91,.93,.94,.95
DATA 0,0,0,0,0,0,0,0,0,.90,.92,.94,.95,.96,.97,.98
READ Curve(*)
MAT Ccondition= (1)
!
!
GOSUB Input                    ! Directs flow to subroutine INPUT
REDIM Bays(6),Factor(6)
FOR B=1 TO Storey-1
  Cycle=1
  N=Bays(B)
  MAT Endmoment= (0)
  REDIM K_inv_inv(N+3,N+3),Diffmr(N+3,N+3),Compare(N+3,N+3),Compare1(N+3,N+3)
  REDIM K(N+3,N+3),K(N+3,N+3),Kinv(N+3,N+3)
  REDIM Endmom(2,N),F(N+3),D(N+3),Beammom(2,N+1)
  REDIM Hmombm(3,N),Hmomcol(2,N+1),Hmombase(N+1)
  REDIM Servmombm(3,N),Servmomcol(2,N+1),Servmombase(N+1)
  FOR Count=1 TO N+1
    IF Cycle=1 THEN
      Axial(2,Count)=Axial(1,Count)
    END IF
    IF Count=1 THEN
      Axial(1,Count)=Axial(2,Count)+Load(B,Count)*Beamprop(1,B,Count)/2+Thrust(B,Count)
    ELSE
      IF Count=N+1 THEN
        Axial(1,Count)=Axial(2,Count)+Load(B,Count-1)*Beamprop(1,B,Count-1)/2+Thrust(B,Count)
      ELSE
        Axial(1,Count)=Axial(2,Count)+Load(B,Count)*Beamprop(1,B,Count)/2+Load(B,Count-1)*Beamprop(1,B,Count-1)/2+Thrust(B,Count)
      END IF
    END IF
  NEXT Count
  Lambda=1.0
  Hinge=0
  Hinge1=0
  Total=0
  Counter=0                    !Counter=0 causes calc of moments with Fh only.
  IF B=Storey-1 THEN
    H1=Colprop(1,B,1)
  ELSE
    H1=Colprop(1,B,1,/2
  END IF
  IF B=1 THEN
    H2=.00001
  ELSE
    IF Cycle=1 THEN
      H2=Colprop(1,B-1,1)/2
    ELSE
      H2=(Colprop(1,B-1,1)/2)*Slendratio1
    END IF
  END IF
  GOSUB Matrix_K
  FOR X=BASE(K,1) TO SIZE(K,1)+BASE(K,1)-1
    FOR Y=BASE(K,2) TO SIZE(K,2)+BASE(K,2)-1
      Total=Total+K(X,Y)^2
    NEXT Y
  NEXT X
  Test=DET(K)/SQR(Total)
  IF Test<.001 THEN
    Lambda=Lambda
    IF Cycle=1 THEN Hinges(B)=Hinge
    GOTO 770
  END IF
```

```
635     MAT K_inv_inv= INV(Kinv)
640     MAT Differ= K_inv_inv-K
645     IF Cycle=1 THEN
650       MAT Compare= Differ)(.001)
655       MAT Compare1= Differ<(-.001)
660     ELSE
665       MAT Compare= Differ)(500000)
670       MAT Compare1= Differ<(-500000)
675     END IF
680     IF SUM(Compare)+SUM(Compare1)>0 THEN
685       Lambdap=Lambda
690       IF Cycle=1 THEN Hinges(8)=Hinge
695       GOTO 770
700     ELSE
705       Lambda1=Lambda
710       Lambda2=0
715     END IF
720     GOSUB Matrix_f
725     GOSUB Moments
730     IF Hinge1oc)5 THEN GOTO 685
735     IF Lambda2=0 THEN
740       Lambda2=Lambda1*1.1
745       GOTO 720
750     ELSE
755       GOSUB Next_hinge
760       GOTO 575
765     END IF
770     GOSUB Elastic
775     GOSUB Interaction
780     GOSUB Result
785   NEXT S
790   Fail=MIN(Factor(*))
795   PRINT
800   PRINT
805   PRINT USING Bi0;Fail
810   IMAGE "STRUCTURE COLLAPSE LOAD FACTOR =",DD.DD,/,36("-")
815   DISP USING 820
820   IMAGE #,"Do you want Hard Copy of input/results_____(Y/N)"
825   INPUT Que$
830   IF Que$="Y" THEN
835     PRINTER IS 701
840     Print=1
845     GOTO 945
850   END IF
855   IF Que$="N" THEN Print=0
860   DISP USING 865
865   IMAGE #,"Picture of collapse mechanism_____(Y/N)"
870   INPUT Que$
875   IF Que$="Y" THEN
880     DISP USING 885
885     IMAGE #,"Collapse mechanism for Storey No.___"
890     INPUT S
895     GOSUB Graphic
900     GOTO 860
905   END IF
910   DISP USING 915
915   IMAGE #,"Do you want a re-run_____(Y/N)"
920   INPUT Que$
925   IF Que$="Y" THEN
930     PRINTER IS 1
935     MAT Ocondition= (0)
936     MAT Ccondition= (1)
947     IF Case=0 THEN
948       FOR Count=1 TO N+1
949         Ocondition(1,Storey-1,Count)=0
950       NEXT Count
```

```
951     END IF
952     Hingeloc=0
953     MAT Axial= (0)
954     MAT Colmom= (0)
955     MAT Beammom= (0)
956     MAT Endmom= (0)
957     MAT Beamnom= (0)
958     IF Print=1 THEN GOTO 365
959     DISP USING 960
960     IMAGE #,"Edit Column fixity/Material strengths_____(Y/N)"
961     INPUT Que$
962     Edit=1
963     IF Que$="Y" THEN GOTO 1010
964     IF Que$="N" THEN GOTO 960
965   END IF
966   IF Que$="N" THEN GOTO 10215
967   GOTO 910
968   St=Storey
969   DISP USING 970
970   IMAGE #,"Edit Beam/Column properties or loads_____(Y/N)"
971   INPUT Que$
972   IF Que$="Y" THEN
973     Edit=1
974     DISP USING 975
975     IMAGE #,"Edit Storey No.___"
976     INPUT S
977     Storey=S
978     N=Bays(S)
979     DISP USING 980
980     IMAGE #,"EDIT: Beams(1), Columns(2), Loads(3), No-edit(4)"
981     INPUT Ed
982     SELECT Ed
983     CASE =1
984       GOTO 1340
985     CASE =2
986       GOTO 1955
987     CASE =3
988       GOTO 1615
989     CASE =4
990       GOTO 969
991     END SELECT
992   END IF
993   IF Que$="N" THEN
994     IF Edit=1 THEN Storey=St
995     Edit=0
996     GOTO 365
997   END IF
998   GOTO 969
999   !-------------------------------------------------------------------
1000  !Subroutine INPUT calls for structure properties and dimensions.
1005  !-------------------------------------------------------------------
1010  Input:  !
1015  DISP USING 1020
1020  IMAGE #,"RUN DESCRIPTION (max 80 chars)"
1025  INPUT Describe$
1030  INPUT "Number of storeys",S
1035  INPUT "Young's modulus (kN/m2)",E
1040  DISP USING 1045
1045  IMAGE #, "Material yield stress for beams,columns( Fyb, Fyc (kN/M2)"
1050  INPUT Fyb,Fyc
1055  INPUT "Factor c, where Fp = cFy",Fac
1060  Fpb=Fyb*Fac
1065  Fpc=Fyc*Fac
1070  DISP USING 1075
1075  IMAGE #,"Are beams fixed or pinned_____(F/P)"
1080  INPUT Que$
```

```
1080  IF Ques$="F" THEN
1090    Base=0
1095    GOTO 1130
1100  END IF
1105  IF Ques$="P" THEN
1110    Base=1
1115    GOTO 1135
1120  END IF
1125  GOTO 1070
1130  IF Edit=1 THEN GOTO 959
1135  Dim=0
1140  FOR Storey=1 TO 8
1145    PRINTER IS 1
1150    PRINT FNClear$
1155    PRINT USING 1160;Storey
1160    IMAGE 25X,"STOREY NO.",2D,/,25X,12("-")
1165    INPUT "Number of bays",N
1170    Bays(Storey)=N
1175    IF N>Dim THEN
1180      Dim=N
1185      REDIM Beamprop(5,8,Dim),Colprop(5,8,Dim+1)
1190      REDIM Load(8,Dim+1),Thrust(8,Dim+1),Bcondition(8,Dim),Ccondition(2,8,D
im+1)
1193      IF Base=0 THEN
1200        FOR Count=1 TO Dim+1
1205          Ccondition(1,8,Count)=0
1210        NEXT Count
1215      END IF
1220      REDIM Beamcm(3,8+1,Dim),Colmom(2,8+1,Dim+1)
1225    END IF
1230    PRINT USING 1235
1235    IMAGE "Beam No.",6X,"Length(m)",4X,"I(m4)",6X,"zp(m3)",4X,"Area(m2)",4X,
"y_max(m)"/,8("-"),6X,9("-"),4X,5("-"),6X,6("-"),4X,8("-"),4X,8("-")
1240    FOR Beam=1 TO N
1245      DISP USING 1250;Beam
1250      IMAGE #,"BEAM NO.",2D,":  Length(m), I(m4), zp(m3), Area(m2), y_max(m)
"
1255      INPUT L,Ib,Zpb,Ab,Yb
1260      Beamprop(1,Storey,Beam)=L
1265      Beamprop(2,Storey,Beam)=Ib
1270      Beamprop(3,Storey,Beam)=Zpb
1275      Beamprop(4,Storey,Beam)=Ab
1280      Beamprop(5,Storey,Beam)=Yb
1285      PRINT USING 1290;Beam,L,Ib,Zpb,Ab,Yb
1290      IMAGE 2X,2D,10X,2D.3D,5X,D.6D,3X,D.6D,3X,D.6D,4X,D.3D
1295    NEXT Beam
1300    DISP USING 1305
1305    IMAGE #,"Are beam dimensions and properties OK.....(Y/N)"
1310    INPUT Ques$
1315    IF Ques$="Y" THEN
1320      IF Edit=1 THEN GOTO 969
1325      GOTO 1480
1330    END IF
1335    IF Ques$="N" THEN
1340      IF Edit=1 THEN
1345        PRINT FNClear$
1350        PRINT USING 1355;Storey
1355        IMAGE 25X,"STOREY NO.",2D,/,25X,12("-")
1360        PRINT USING 1365
1365        IMAGE "Beam No.",6X,"Length(m)",4X,"I(m4)",6X,"zp(m3)",4X,"Area(m2)"
,4X,"y_max(m)",/,8("-"),6X,9("-"),4X,5("-"),6X,6("-"),4X,8("-"),4X,8("-")
1370        FOR Beam=1 TO Bays(8)
1375          PRINT USING 1380;Beam,Beamprop(1,Storey,Beam),Beamprop(2,Storey,Be
am),Beamprop(3,Storey,Beam),Beamprop(4,Storey,Beam),Beamprop(5,Storey,Beam)
1380          IMAGE 2X,2D,10X,2D.3D,5X,D.3D,3X,D.6D,3X,D.6D,4X,D.3D
1385        NEXT Beam
```

```
1390      END IF
1395      DISP USING 1400
1400      IMAGE #,"Change beam No.___"
1405      INPUT No
1410      DISP USING 1415;No
1415      IMAGE #,"BEAM NO.",2D,":  Length(m), I(m4), zp(kNm), Area(m2), y_max(m
          )"
1420      INPUT L,Ib,Zpb,Ab,Yb
1425      Beamprop(1,Storey,No)=L
1430      Beamprop(2,Storey,No)=Ib
1435      Beamprop(3,Storey,No)=Zpb
1440      Beamprop(4,Storey,No)=Ab
1445      Beamprop(5,Storey,No)=Yb
1450      PRINT USING 1455;No,L,Ib,Zpb,Ab,Yb
1455      IMAGE 4x,10X,2D.3D,3X,D.6D,3X,D.6D,3X,D.6D,4X,D.3D
1460      GOTO 1300
1465    ELSE
1470      GOTO 1300
1475    END IF
1480    PRINT FNClear$
1485    PRINT USING 1490;Storey
1490    IMAGE 25X,"STOREY NO.",2D,/,25X,12("-")
1495    PRINT USING 1500
1500    IMAGE 10X,"Beam No.",10X,"Udl(kN/m)",/,10X,8("-"),10X,8("-")
1505    FOR Beam=1 TO N
1510      DISP USING 1515;Beam
1515      IMAGE #,"BEAM NO.",2D,": Udl(kN/m)"
1520      INPUT Udl
1525      Load(Storey,Beam)=Udl
1530      PRINT USING 1535;Beam,Udl
1535        IMAGE 12X,2D,14X,3D.DD
1540    NEXT Beam
1545    DISP USING 1550
1550    IMAGE #,"Horizontal load (kN)"
1555    INPUT Fh
1560    PRINT USING 1565;Fh
1565    IMAGE /,"Horizontal load =",3D.DD,"kN"
1570    Load(Storey,N+1)=Fh
1575    DISP USING 1580
1580    IMAGE #,"Are Udl's, Horizontal loads OK_____(Y/N)"
1585    INPUT Ques
1590    IF Ques="Y" THEN
1595      IF Edit=1 THEN GOTO 969
1600      GOTO 1770
1605    END IF
1610    IF Ques="N" THEN
1615      IF Edit=1 THEN
1620        PRINT FNClear$
1625        PRINT USING 1630;Storey
1630        IMAGE 25X,"STOREY NO.",2D,/,25X,12("-")
1635        PRINT USING 1640
1640        IMAGE 10X,"Beam No.",10X,"Udl(kN/m)",/,10X,8("-"),10X,8("-")
1645        FOR Beam=1 TO Bays(S)
1650          PRINT USING 1655;Beam,Load(Storey,Beam)
1655            IMAGE 12X,2D,14X,3D.DD
1660        NEXT Beam
1665        PRINT USING 1670;Load(Storey,Bays(S)+1)
1670        IMAGE /,"Horizontal load =",3D.DD,"kN"
1675      END IF
1680      DISP USING 1685
1685      IMAGE #,"Change Udl_____(Y/N)"
1690      INPUT Ques
1695      IF Ques="Y" THEN
1700        DISP USING 1705
1705        IMAGE #,"Change Udl for beam no.___"
1710        INPUT No
```

```
1715          DISP USING 1720;No
1720          IMAGE @,"BEAM NO.",2D,"  ;Ud(kN/m)"
1725          INPUT Ud
1730          Load(Storey,No)=Ud
1735          PRINT USING 1740;No,Ud
1740          IMAGE 10X,4*,14X,30.3D
1745        END IF
1750        IF Ques$="N" THEN GOTO 1545
1755        GOTO 1578
1760      END IF
1765      GOTO 1575
1770      PRINT PAGE
1775      PRINT USING 1780;Storey
1780      IMAGE 25X,"STOREY NO.",2D,/,25X,12("-")
1785      DISP USING 1790
1790      IMAGE @,"Storey height (m) ="
1795      INPUT H
1800      PRINT USING 1805;H
1805      IMAGE "Storey height =",2D.3D,"m",/
1810      DISP USING 1815
1815      IMAGE @,"Is storey height OK_____(Y/N)"
1820      INPUT Ques
1825      IF Ques="Y" THEN GOTO 1840
1830      IF Ques="N" THEN GOTO 1790
1835      GOTO 1810
1840      PRINT USING 1845
1845      IMAGE "Column No.",5X,"Axial load(kN)",4X,"I(m4)",5X,"zp(m3)",4X,"Area(m
2)",4X,"y_max(m)",/,10("-"),5X,14("-"),4X,5("-"),5X,6("-"),4X,8("-"),4X,8("-")
1850      FOR Col=1 TO N+1
1855        DISP USING 1860;Col
1860        IMAGE @,"COLUMN NO.",2D,":  Axial load(kN) ;I(m4), zp(kNm), Area(m2),
y_max(m)"
1865        INPUT Th,Ic,Zpc,Ac,Yc
1870        Colprop(1,Storey,Col)=H
1875        Colprop(2,Storey,Col)=Ic
1880        Colprop(3,Storey,Col)=Zpc
1885        Colprop(4,Storey,Col)=Ac
1890        Colprop(5,Storey,Col)=Yc
1895        Thrust(Storey,Col)=Th
1900        PRINT USING 1905;Col,Th,Ic,Zpc,Ac,Yc
1905        IMAGE 3X,2D,13X,5D.2D,5X,D.6D,2X,D.6D,3X,D.6D,5X,D.3D
1910      NEXT Col
1915      DISP USING 1920
1920      IMAGE @,"Are column values OK_____(Y/N)"
1925      INPUT Ques
1930      IF Ques="Y" THEN
1935        IF Edit=1 THEN GOTO 969
1940        GOTO 1997
1945      END IF
1950      IF Ques="N" THEN
1955        IF Edit=1 THEN
1960          PRINT PAGE#
1970          PRINT USING 1971;Storey
1971          IMAGE 25X,"STOREY NO.",2D,/,25X,12("-")
1972          PRINT USING 1973
1973          IMAGE "Column No.",5X,"Axial load(kN)",4X,"I(m4)",5X,"zp(m3)",4X,"Ar
ea(m2)",4X,"y_max(m)"
1974          PRINT USING 1975
1975          IMAGE 10("-"),2X,14("-"),4X,5("-"),5X,6("-"),4X,8("-"),4X,8("-")
1976          FOR Col=1 TO Beys(B)+1
1977            PRINT USING 1978;Col,Thrust(Storey,Col),Colprop(2,Storey,Col),Colp
rop(3,Storey,Col),Colprop(4,Storey,Col),Colprop(5,Storey,Col)
1978            IMAGE 3X,2D,13X,5D.2D,5X,D.6D,2X,D.6D,3X,D.6D,5X,D.3D
1979          NEXT Col
1980        END IF
1981        DISP USING 1982
```

```
1982      IMAGE #,"Change column No.___"
1983      INPUT No
1984      DISP USING 1985;No
1985      IMAGE #,"COLUMN NO.",2D,":  I(m4), zp(m3), Area(m2), y_max(m)"
1986      INPUT Ic,Zpc,Ac,Yc
1987      Colprop(2,Storey,No)=Ic
1988      Colprop(3,Storey,No)=Zpc
1989      Colprop(4,Storey,No)=Ac
1990      Colprop(5,Storey,No)=Yc
1991      PRINT USING 1992;No,H,Ic,Zpc,Ac,Yc
1992      IMAGE X,A#,1X,2D.3D,3X,D.6D,2X,D.6D,3X,D.6D,3X,D.3D
1993      GOTO 1915
1994    ELSE
1995      GOTO 1915
1996    END IF
1997  NEXT Storey
1998  RETURN
1999  !-----------------------------------------------------------------
2000  !Subroutine MATRIX [K] calculates structure stiffness matrix.
2005  !-----------------------------------------------------------------
2010 Matrix_K:
2015  MAT K= (0)
2020  A=0
2025  B=0
2030  FOR Count=1 TO N-1
2035    A=A+((Condition(1,S,Count)=0)*12+(Condition(1,S,Count)=1)*3+(Condition(1,S,Count)=10)*3)*Colprop(2,S,Count)
2040    IF S=1 THEN
2045      B=0
2050    ELSE
2055      B=B+(Condition(2,S,Count)=1)*3*Colprop(2,S-1,Count)
2060    END IF
2065  NEXT Count
2070  K(1,1)=(A/H1^3+B/H2^3)*E*4.5
2075  K(1,3)=-(B*E/H2^2)
2080  K(3,3)=-.5*K(1,3)
2085  C=((Condition(1,S,1)=0)+4+(Condition(1,S,1)=1)*3)*Colprop(2,S,1)
2090  IF S=1 THEN
2095    D=0
2100  ELSE
2105    D=(Condition(2,S,1)=1)*3*Colprop(2,S-1,1)
2110  END IF
2115  K(1,2)=(D/H2^2-C/H1^2)*E
2120  K(2,3)=-(D*E/H2^2)
2125  U=((Bcondition(S,1)=0)*4+(Bcondition(S,1)=1)*3)*E*Beamprop(2,S,1)/Beamprop(1,S,1)
2130  K(2,2)=((((Condition(1,S,1)=0)*1/1.5+(Condition(1,S,1)=1)*1)*C/H1+D/H2)*E+U)*.5
2135  K(2,4)=((Bcondition(S,1)=0)*2)*E*Beamprop(2,S,1)/Beamprop(1,S,1)
2140  FOR Count=4 TO N+3
2145    B=((Condition(1,S,Count-2)=0)*4+(Condition(1,S,Count-2)=1)*3)*Colprop(2,S,Count-2)
2150    IF S=1 THEN
2155      M=0
2160    ELSE
2165      M=(Condition(2,S,Count-2)=1)*3*Colprop(2,S-1,Count-2)
2170    END IF
2175    K(1,Count)=-(B*E/H1^2)+(M*E/H2^2)
2180    K(3,Count)=-(M*E/H2^2)
2185    J=((Condition(1,S,Count-2)=0)*4+(Condition(1,S,Count-2)=1)*3)*Colprop(2,S,Count-2)*E/H1+M*E/H2
2190    P=((Bcondition(S,Count-3)=0)*4+(Bcondition(S,Count-3)=10)*3)*E*Beamprop(2,S,Count-3)/Beamprop(1,S,Count-3)
2195    IF Count=N+3 THEN
2200      K(Count,Count)=(J+P)*.5)
2205    ELSE
```

```
2210       W=((Bcondition(B,Count-2)=0)*4+(Bcondition(B,Count-2)=1)*3)*E*Beamprop
(2,B,Count-2)/Beamprop(1,B,Count-2)
2215       K(Count,Count)=((3*P+Q)*.5)
2220     END IF
2225   NEXT Count
2230   FOR Count=5 TO N-3
2235     R= (Bcondition(B,Count-3)=0)*2)*E*Beamprop(2,B,Count-3)
2240     K(Count-1,Count)=R/Beamprop(1,B,Count-3)
2245   NEXT Count
2250   MAT Ktr= TRN(K)
2255   MAT K= K+Ktr
2260   IF B=1 THEN K(3,3)=1
2265   MAT Kinv= INV(K)
2270   RETURN
2275   !-------------------------------------------------------
3000   !Subroutine MATRIX (F) calculates the force matrix.
3005   !-------------------------------------------------------
3010 Matrix_f:    !
3015   IF Lambda2=0 THEN
3020     Lambda=Lambda1
3025     X=0
3030   ELSE
3035     Lambda=Lambda2
3040     X=1
3045   END IF
3050   FOR Count=1 TO N-1
3055     FOR Loc=1 TO 2
3060       IF B=1 THEN
3065         Py=Colprop(4,1,Count)*Fpc
3070         Ax1=Axial(1,Count)
3075       ELSE
3080         Py=Colprop(4,(B+1)-Loc,Count)*Fpc
3085         Ax1=Axial(Loc,Count)
3090       END IF
3095       IF Ax1*Lambda/Py>=1 THEN
3100         Lambdap=Py/Ax1
3105         IF Lambdap<Lambda THEN
3110           Lambda=Lambdap
3115           Colno=Count
3120           Hingeloc=5+Loc
3125           Location(1,B,Hinge+1)=0
3130           Location(2,B,Hinge+1)=Colno+Hingeloc/10
3135           X=0
3140         END IF
3145       END IF
3150     NEXT Loc
3155   NEXT Count
3160   IF Hingeloc>5 THEN Hinge=Hinge+1
3165   IF Counter=0 THEN GOTO 3460
3170   FOR Count=1 TO N
3175     W1=Load(B,Count)
3180     SELECT Bcondition(B,Count)
3185     CASE =0
3190       M1=W1*Lambda*(Beamprop(1,B,Count)^2)/12
3195       Mr=M1
3200     CASE =10
3205       M1=-Beamom(1,B,Count)
3210       Mr=W1*Lambda*(Beamprop(1,B,Count)^2)/8-M1/2
3215     CASE =1
3220       Mr=Beamom(3,B,Count)
3225       M1=W1*Lambda*(Beamprop(1,B,Count)^2)/8-Mr/2
3230     CASE =20
3235       M1=-Beamom(1,B,Count)
3240       Mr=W1*(Lambda-Lambda1)*(Beamprop(1,B,Count)^2)/4+Fem2hr
3245     CASE =2
3250       M1=W1*(Lambda-Lambda1)*(Beamprop(1,B,Count)^2)/4-Fem2h1
```

```
3255        Mr=Basmom(3,S,Count)
3260      CASE =11
3265        M)=-Basmom(1,S,Count)
3270        Mr=Basmom(3,S,Count)
3275      END SELECT
3280      Endmom(1,Count)=M)
3285      Endmom(2,Count)=Mr
3290    NEXT Count
3295    FOR Count=1 TO N+1
3300      FOR Loc=1 TO 2
3305        IF S=1 THEN
3310          Py=Colprop(4,1,Count)*Fpc
3315          Mp1=Colprop(3,1,Count)*Fpc
3320        ELSE
3325          Py=Colprop(4,(S+1)-Loc,Count)*Fpc
3330          Mp1=Colprop(3,(S+1)-Loc,Count)*Fpc
3335        END IF
3340        SELECT Condition(Loc,S,Count)
3345        CASE =0
3350        CASE =1
3355          IF S=Storey-1 THEN
3360            IF Loc=1 THEN
3365              IF Base=0 THEN
3370                Basemom(1+X,Count)=SGN(Basemom(1+X,Count))*MIN(ABS(Mp1),ABS(1.
18*Mp1*(1-Axial(1,Count)*Lambda/Py)))
3375              END IF
3380            END IF
3385          END IF
3390        CASE =10
3395          Colmom(Loc,S+X,Count)=SGN(Colmom(Loc,S+X,Count))*MIN(ABS(Mp1),ABS(1.
18*Mp1*(1-Axial(Loc,Count)*Lambda/Py)))
3400        CASE =11
3405          Colmom(Loc,S+X,Count)=SGN(Colmom(Loc,S+X,Count))*MIN(ABS(Mp1),ABS(1.
18*Mp1*(1-Axial(Loc,Count)*Lambda/Py)))
3410          IF S=Storey-1 THEN
3415            IF Loc=1 THEN
3420              IF Base=0 THEN
3425                Basemom(1+X,Count)=SGN(Basemom(1+X,Count))*MIN(ABS(Mp1),ABS(1.
18*Mp1*(1-Axial(1,Count)*Lambda/Py)))
3430              END IF
3435            END IF
3440          END IF
3445        END SELECT
3450      NEXT Loc
3455    NEXT Count
3460    F(2)=Endmom(1,1)-((Condition(1,S,1/=10)*Colmom(1,S+X,1))-((Condition(1,S
,1)=11)*Colmom(1,S+X,1))-((Condition(1,S,1)=1)*Basemom(1+X,1)/2)
3465    F(2)=F(2)-((Condition(2,S,1)=11)*Colmom(2,S+X,1))
3470    FOR Count=4 TO N+3
3475      F1=-((Condition(1,S,Count-2)=10)*Colmom(1,S+X,Count-2))-((Condition(1,
S,Count-2)=11)*Colmom(1,S+X,Count-2))
3480      F1=F1-((Condition(1,S,Count-2)=1)*Basemom(1+X,Count-2)/2)-((Condition(
2,S,Count-2)=11)*Colmom(2,S+X,Count-2))
3485      IF Count=N+3 THEN
3490        F(Count)=-Endmom(2,Count-3)+F1
3495      ELSE
3500        F(Count)=Endmom(1,Count-2)-Endmom(2,Count-3)+F1
3505      END IF
3510    NEXT Count
3515    Hor1=0
3520    Hor2=0
3525    FOR Count=1 TO N+1
3530      Hor1=((Condition(2,S,Count)=11)*Colmom(2,S+X,Count))/H2
3535      Hor2=Hor2+Hor1
3540      Hor=(((Condition(1,S,Count)=10)*Colmom(1,S+X,Count))+((Condition(1,S,C
ount)=11)*Basemom(1+X,Count)))*3/(2*H1)-Hor
```

```
3345    Hor=Hor+((Condition(1,S,Count)=1)*(Colmom(1,S+X,Count)+Basemom(1+X,Cou
nt))/H1)
3350    Hor1=Hor1+Hor
3355  NEXT Count
3360  F(1)=Hor1+Load(S,N+1)*Lambda
3365  Shor=0
3370  IF S=1 THEN
3375    Shor=0
3380  ELSE
3385    FOR Count=1 TO S-1
3390      Shor=Shor+Load(Count,N+1)*Lambda
3395    NEXT Count
3400  END IF
3405  IF Cycle=2 THEN Shor=Shor*Blendratio1
3410  F(3)=Hor2+Shor
3415  RETURN
3420  !-------------------------------------------------------------------------
4300  !Subroutine MOMENTS calculates the bending moments at for each load factor
      !-------------------------------------------------------------------------
4010  Moments:
4015  MAT D=e Kinv*F
4020  FOR Count=1 TO N
4025    Ks=Beamprop(2,S,Count)*E/Beamprop(1,S,Count)
4030    SELECT Condition(S,Count)
4035    CASE =0
4040      IF Count=1 THEN
4045        M1=(4*Df(2)+2*Df(4))*Ks-Endmom(1,1)
4050        M3=(2*Df(2)+4*Df(4))*Ks+Endmom(2,1)
4055      ELSE
4060        M1=(4*Df(Count+2)+2*Df(Count+3))*Ks-Endmom(1,Count)
4065        M3=(2*Df(Count+2)+4*Df(Count+3))*Ks+Endmom(2,Count)
4070      END IF
4075      Beamm(1,S+X,Count)=M1
4080      Beamm(3,S+X,Count)=M3
4085      Beamm(2,S+X,Count)=-(-M1+M3)/2+(Load(S,Count)*Lambda/8)*(Beamprop(1,S
,Count)^2)
4090      IF Counter=0 THEN
4095        Hmomba(1,Count)=M1
4100        Hmomba(2,Count)=-(-M1+M3)/2
4105        Hmomba(3,Count)=M3
4110      END IF
4115      IF Lambda=1 THEN
4120        Servmomba(1,Count)=M1
4125        Servmomba(2,Count)=Beamm(2,S+X,Count)
4130        Servmomba(3,Count)=M3
4135      END IF
4140    CASE =10
4145      M3=(3*Df(Count+3))*Ks+Endmom(2,Count)
4150      Beamm(1,S+X,Count)=Beamm(1,S,Count)
4155      Beamm(3,S+X,Count)=M3
4160      Beamm(2,S+X,Count)=(Load(S,Count)*Lambda/8)*(Beamprop(1,S,Count)^2)-(
-Beamm(1,S,Count)+M3)/2
4165    CASE =20
4170      M3=Endmom(2,Count)
4175      Beamm(1,S+X,Count)=Beamm(1,S,Count)
4180      Beamm(3,S+X,Count)=M3
4185      Beamm(2,S+X,Count)=Beamm(2,S,Count)
4190    CASE =1
4195      IF Count=1 THEN
4200        M1=(3*Df(2))*Ks-Endmom(1,1)
4205      ELSE
4210        M1=(3*Df(Count+2))*Ks-Endmom(1,Count)
4215      END IF
4220      Beamm(1,S+X,Count)=M1
4225      Beamm(3,S+X,Count)=Beamm(3,S,Count)
4230      Beamm(2,S+X,Count)=(Load(S,Count)*Lambda/8)*(Beamprop(1,S,Count)^2)-(
```

```
-M1+Beamom(3,8,Count))/2
4235    CASE =2
4240      IF Count=1 THEN
4245        M1=-EndMom(1,1)
4250      ELSE
4255        M1=-Endmom(1,Count)
4260      END IF
4265      Beamom(1,8+X,Count)=M1
4270      Beamom(3,8+X,Count)=Beamom(3,8,Count)
4275      Beamom(2,8+X,Count)=Beamom(2,8,Count)
4280    CASE =11
4285      M2=(Load'8,Count)*Lambda'8)*(Beamprop(1,8,Count)^2)
4290      Beamom(1,8+X,Count)=Beamom(1,8,Count)
4295      Beamom(3,8+X,Count)=Beamom(3,8,Count)
4300      Beamom(2,8+X,Count)=M2-((Beamom(3,8,Count)-Beamom(1,8,Count))/2)
4305    END SELECT
4310  NEXT Count
4315  FOR Count=1 TO N-1
4320    Ks=Colprop(2,8,Count)*E/H1
4325    SELECT Condition(1,8,Count)
4330    CASE =0
4335      IF Count=1 THEN
4340        M1=(4*Df(2)-(4*Df(1)/H1))*Ks
4345        Mbase=(2*Df(2)-(4*Df(1)/H1))*Ks
4350      ELSE
4355        M1=(4*Df(Count+2)-(4*Df(1)/H1))*Ks
4360        Mbase=(2*Df(Count+2)-(4*Df(1)/H1))*Ks
4365      END IF
4370      Colmom(1,8+X,Count)=M1
4375      Basemom(1+X,Count)=Mbase
4380      IF Counter=0 THEN
4385        Nmomcol(1,Count)=M1
4390        Nmombase(Count)=Mbase
4395      END IF
4400      IF Lambda=1 THEN
4405        Servmomcol(1,Count)=M1
4410        Servmombase(Count)=Mbase
4415      END IF
4420    CASE =1
4425      IF Count=1 THEN
4430        M1=(3*Df(2)-(3*Df(1)/H1))*Ks+Basemom(1,Count)/2
4435      ELSE
4440        M1=(3*Df(Count+2)-(3*Df(1)/H1))*Ks+Basemom(1,Count)/2
4445      END IF
4450      Colmom(1,8+X,Count)=M1
4455      IF Counter=0 THEN Nmomcol(1,Count)=M1
4460      IF Lambda=1 THEN Servmomcol(1,Count)=M1
4465    CASE =10
4470      Mbase=(-3*Df(1)/H1)*Ks+Colmom(1,8,Count)/2
4475      Basemom(1+X,Count)=Mbase
4480    CASE =11
4485    END SELECT
4490    IF 8>1 THEN
4495      IF Condition(2,8,Count)=1 THEN
4500        IF Count=1 THEN
4505          Mup=(3*Df(2)-(3*(Df(3)-Df(1))/H2))*Colprop(2,8-1,1)*E/H2
4510        ELSE
4515          Mup=(3*Df(Count+2)-(3*(Df(3)-Df(1))/H2))*Colprop(2,8-1,Count)*E/H2
4520        END IF
4525        Colmom(2,8+X,Count)=Mup
4530        IF Counter=0 THEN Nmomcol(2,Count)=Mup
4535        IF Lambda=1 THEN Servmomcol(2,Count)=Mup
4540      END IF
4545    END IF
4550  NEXT Count
4555  IF Counter=0 THEN
```

```
4560    Counter=1
4565    GOTO 720
4570  END IF
4575  RETURN
4580  !-------------------------------------------------------------------------
5000  !Subroutine NEXT HINGE calculates load factor and location of hinge.
5005  !-------------------------------------------------------------------------
5010 Next_hinge:    !
5015  Lambda=1000
5020  Colno=0
5025  FOR Count=1 TO N
5030    FOR Loc=1 TO 3
5035      Mom1=Beamom(Loc,S,Count)
5040      Mom2=Beamom(Loc,S+1,Count)
5045      IF Mom2<>Mom1 THEN
5050        IF SGN(Mom2-Mom1)=SGN(Mom1) THEN
5055          Mp1=SGN(Mom1)*Beamprop(3,S,Count)*Fpb
5060        ELSE
5065          Mp1=-1*SGN(Mom1)*Beamprop(3,S,Count)*Fpb
5070        END IF
5075        Lambdap=((Lambda2-Lambda1)/(Mom2-Mom1))*Mp1+(Lambda1*Mom2-Lambda2*Mo
m1)/(Mom2-Mom1)
5080        IF Lambdap<Lambda THEN
5085          Lambda=Lambdap
5090          Beamno=Count
5095          Hingeloc=Loc
5100          Replastic=Mp1
5105          IF Cycle=1 THEN Location(1,S,Hinge+1)=Beamno*Hingeloc/10
5110        END IF
5115      END IF
5120    NEXT Loc
5125  NEXT Count
5130  FOR Count=1 TO N+1
5135    FOR Loc=1 TO 2
5140      IF S=1 THEN
5145        Py=Colprop(4,1,Count)*Fpc
5150        Ax1=Axisl(1,Count)
5155      ELSE
5160        Py=Colprop(4,(S+1)-Loc,Count)*Fpc
5165        Ax1=Axial(Loc,Count)
5170      END IF
5175      Mom1=Colmom(Loc,S,Count)
5180      Mom2=Colmom(Loc,S+1,Count)
5185      IF Condition(Loc,S,Count)<>11 THEN
5190        IF SGN(Mom2-Mom1)=SGN(Mom1) THEN
5200          IF S=1 THEN
5205            Mp1=SGN(Mom1)*Colprop(3,1,Count)*Fpc
5210          ELSE
5215            Mp1=SGN(Mom1)*Colprop(3,(S+1)-Loc,Count)*Fpc
5220          END IF
5225        ELSE
5230          IF S=1 THEN
5235            Mp1=-1*SGN(Mom1)*Colprop(3,1,Count)*Fpc
5240          ELSE
5245            Mp1=-1*SGN(Mom1)*Colprop(3,(S+1)-Loc,Count)*Fpc
5250          END IF
5255        END IF
5260        IF S=1 THEN
5265          IF Loc=2 THEN GOTO 5330
5270        END IF
5275        Lambdap=((1.1@*(Lambda2-Lambda1))*Mp1*Py)+(Lambda1*Mom2-Lambda2*Mom
1)*Py)/((Mom2-Mom1)*Py+1.1@*(Lambda2-Lambda1)*Mp1*Ax1al(Loc,Count))
5280          IF Lambdap<Lambda THEN
5285            Lambda=Lambdap
5290            Colno=Count
```

```
5295            Hingeloc=Loc
5300            Colplastic=BSN(Mpl)*MIN(ABS(Mpl),ABS(1.18*Mpi*(1-Axial(1,Count)*
Lambdap/Py)))
5305              IF Cycle=1 THEN
5310                Location(1,B,Hinge+1)=0
5315                Location(2,B,Hinge+1)=Coino+Hingeloc/10
5320              END IF
5325            END IF
5330          END IF
5335        END IF
5340        IF Axi1=Lambda/Py>=1 THEN
5345          Lambda=Py/Axi
5350          IF Lambda>=Lambda THEN
5355            Lambda=Lambdap
5360            Coino=Count
5365            Hingeloc=Loc
5370            IF Cycle=1 THEN
5375              Location(1,B,Hinge+1)=0
5380              Location(2,B,Hinge+1)=Coino+Hingeloc/10
5385            END IF
5390            GOTO 615
5395          END IF
5400        END IF
5405      NEXT Loc
5410      IF B=Storey-1 THEN
5415        IF Base=0 THEN
5420          Mom1=Basemom(1,Count)
5425          Mom2=Basemom(2,Count)
5430          IF Condition(1,B,Count)<>11 THEN
5435            IF Condition(1,B,Count)<>1 THEN
5440              IF BSN(Mom2-Mom1)=BSN(Mom1) THEN
5445                Mpl=BSN(Mom1)*Colprop(3,B,Count)*Fpc
5450              ELSE
5455                Mpl=-1*BSN(Mom1)*Colprop(3,B,Count)*Fpc
5460              END IF
5465              Lambdap=((1.18*(Lambda2-Lambda1)*Mpl*Py)+(Lambda1*Mom2-Lambda2*M
om1)*Py)/((Mom2-Mom1)*Py+1.18*(Lambda2-Lambda1)*Mpl*Axial(1,Count))
5470              IF Lambda>=Lambda THEN
5475                Lambda=Lambdap
5480                Coino=Count
5485                Hingeloc=2
5490                Colplastic=Mpl
5495                IF Cycle=1 THEN
5500                  Location(1,B,Hinge+1)=0
5505                  Location(2,B,Hinge+1)=Coino+Hingeloc/10
5510                END IF
5515              END IF
5520            END IF
5525          END IF
5530        END IF
5535      END IF
5540    NEXT Count
5545    IF Coino=0 THEN
5550      IF Beondition(b,Beamno)=2 THEN
5555        IF Cycle=1 THEN Hingpos(B)=Hinge+1
5560        GOTO 770
5565      END IF
5570      IF Beondition(B,Beamno)=20 THEN
5575        IF Cycle=1 THEN Hinges(B)=Hinge+1
5580        GOTO 770
5585      END IF
5590      IF Beondition(B,Beamno)=11 THEN
5595        IF Cycle=1 THEN Hinges(B)=Hinge+1
5600        GOTO 770
5605      END IF
5610      IF Beondition(B,Beamno)=0 THEN
```

```
5615        IF Hingeloc=1 THEN
5620          Bcondition(B,Beamno)=10
5625        ELSE
5630          Bcondition(B,Beamno)=1
5635        END IF
5640      ELSE
5645        IF Bcondition(B,Beamno)=10 THEN
5650          IF Hingeloc=2 THEN
5655            Bcondition(B,Beamno)=20
5660            Udl=Load(B,Beamno)
5665            Fee2hr=(Beamom(3,B+1,Beamno)*(Udl*Lambda-Udl*Lambda1)-Beamom(3,B,B
eamno)*(Udl*Lambda-Udl*Lambda2))/(Udl*Lambda2-Udl*Lambda1)
5670          ELSE
5675            Bcondition(B,Beamno)=11
5680          END IF
5685        ELSE
5690          IF Hingeloc=2 THEN
5695            Bcondition(B,Beamno)=2
5700            Udl=Load(B,Beamno)
5705            Fee2hl=(Beamom(1,B+1,Beamno)*(Udl*Lambda-Udl*Lambda1)-Beamom(1,B,B
eamno)*(Udl*Lambda-Udl*Lambda2))/(Udl*Lambda2-Udl*Lambda1)
5710          ELSE
5715            Bcondition(B,Beamno)=11
5720          END IF
5725        END IF
5730      END IF
5735      Beamom(Hingeloc,B,Beamno)=Emplastic
5740      Beamom(Hingeloc,B+1,Beamno)=Emplastic
5745    ELSE
5750      IF Hingeloc=1 THEN
5755        IF Lcondition(1,B,Colno)=0 THEN
5760          Lcondition(1,B,Colno)=10
5765        ELSE
5770          Lcondition(1,B,Colno)=11
5775        END IF
5780        Lolmom(1,B,Colno)=Colplastic
5785        Lolmom(1,B+1,Colno)=Colplastic
5790      END IF
5795      IF Hingeloc=2 THEN
5800        Lcondition(2,B,Colno)=11
5805        Lolmom(2,B,Colno)=Colplastic
5810        Lolmom(2,B+1,Colno)=Colplastic
5815      END IF
5820      IF Hingeloc=3 THEN
5825        IF Lcondition(1,B,Colno)=0 THEN
5830          Lcondition(1,B,Colno)=1
5835        ELSE
5840          Lcondition(1,B,Colno)=11
5845        END IF
5850        Basemom(1,Colno)=Colplastic
5855        Basemom(2,Colno)=Colplastic
5860      END IF
5865    END IF
5870    Hinge=Hinge+1
5875    RETURN
5880    !-------------------------------------------------------------------------
6000    !Subroutine RESULT prints collapse load factors.
6005    !-------------------------------------------------------------------------
6010  Results:  !
6015    PRINT
6020    IF B=1 THEN
6025      PRINT USING 6030;Describe$
6030      IMAGE "RUN DESCRIPTION: ",K,/,16("-")
6035      PRINT
6040      PRINT USING 6045;E
6045      IMAGE "Young's Modulus =",9D,1X,"kN/m2"
```

```
6050     PRINT USING 6055;Fpb,Fpc
6055     IMAGE "Material Yield Stresses: Beams=",6D,"kN/m2 ; Columns=",6D,"kN/m2"
6060     PRINT
6065     PRINT USING 6070
6070     IMAGE "Beam Properties",/,15("-")
6075     PRINT USING 6080
6080     IMAGE "Storey",4X,"Beam",4X,"Length(m)",4X,"I(m4)",6X,"zp(m3)",4X,"Area(
         m2)",4X,"y_max(m)"
6085     PRINT USING 6090
6090     IMAGE 6("-"),4X,4("-"),4X,9("-"),4X,5("-"),6X,6("-"),4X,8("-"),4X,8("-")
6095     FOR S=1 TO Storey-1
6100       FOR No=1 TO Bays(S)
6105         IF No=1 THEN
6110           PRINT USING 6115;S,No,Beamprop(1,S,No),Beamprop(2,S,No),Beamprop(3
         ,S,No),Beamprop(4,S,No),Beamprop(5,S,No)
6115           IMAGE 2X,2D,7X,2D,6X,2D.3D,4X,D.6D,3X,D.6D,3X,D.6D,6X,D.3D
6120         ELSE
6125           PRINT USING 6130;No,Beamprop(1,S,No),Beamprop(2,S,No),Beamprop(3,S
         ,No),Beamprop(4,S,No),Beamprop(5,S,No)
6130           IMAGE 11X,2D,6X,2D.3D,4X,D.6D,3X,D.6D,3X,D.6D,6X,D.3D
6135         END IF
6140       NEXT No
6145       PRINT
6150     NEXT S
6155     PRINT
6160     PRINT USING 6165
6165     IMAGE "Column Properties",/,17("-")
6170     PRINT USING 6175
6175     IMAGE "Storey",4X,"Col.",4X,"Axial load(kN)",4X,"I(m4)",6X,"zp(m3)",4X,"
         Area(m2)",4X,"y_max(m)"
6180     PRINT USING 6185
6185     IMAGE 6("-"),4X,4("-"),4X,14("-"),4X,8("-"),6X,6("-"),4X,8("-"),4X,8("-"
         )
6190     FOR S=1 TO Storey-1
6195       FOR No=1 TO Bays(S)+1
6200         IF No=1 THEN
6205           PRINT USING 6210;S,No,Thrust(S,No),Colprop(2,S,No),Colprop(3,S,No)
         ,Colprop(4,S,No),Colprop(5,S,No)
6210           IMAGE 2X,2D,7X,2D,7X,5D.2D,6X,D.6D,3X,D.6D,3X,D.6D,6X,D.3D
6215         ELSE
6220           PRINT USING 6225;No,Thrust(S,No),Colprop(2,S,No),Colprop(3,S,No),C
         olprop(4,S,No),Colprop(5,S,No)
6225           IMAGE 11X,2D,7X,5D.2D,6X,D.6D,3X,D.6D,3X,D.6D,6X,D.3D
6230         END IF
6235       NEXT No
6240       PRINT
6245       PRINT USING 6250;Colprop(1,S,1)
6250       IMAGE "Storey Height =",2D.3D,"m"
6255       PRINT
6260     NEXT S
6265     PRINT
6270     PRINT USING 6275
6275     IMAGE "Structure Loading",/,17("-")
6280     PRINT USING 6285
6285     IMAGE "Storey",5X,"Hor.load(kN)",10X,"Beam",5X,"Udl(kN/m)",/,6("-"),5X,1
         2("-"),10X,4("-"),5X,9("-")
6290     FOR S=1 TO Storey-1
6295       FOR No=1 TO Bays(S)
6300         IF No=1 THEN
6305           PRINT USING 6310;S,Load(S,Bays(S)+1),No,Load(S,No)
6310           IMAGE 2X,2D,10X,3D.DD,14X,2D,8X,3D.DD
6315         ELSE
6320           PRINT USING 6325;No,Load(S,No)
6325           IMAGE 34X,2D,8X,3D.DD
6330         END IF
6335       NEXT No
```

```
6340      PRINT
6345    NEXT B
6350    PRINT
6355    PRINT USING 6360
6360    IMAGE 10X,50("-")
6365    PRINT
6370    PRINT
6375    PRINT
6380    PRINT USING 6385;Describe$
6385    IMAGE "RUN DESCRIPTION: ",K,/,15("-")
6390    PRINT
6395    PRINT USING 6400
6400    IMAGE "Elastic/Plastic Collapse Load Factors",/,37("-")
6405    PRINT USING 6410
6410    IMAGE "Storey No.",15X,"Plastic",6X,"Elastic",6X,"Elastic/Plastic",/,10(
"-"),15X,7("-"),6X,7("-"),8X,15("-")
6415    N=1
6420  END IF
6425  PRINT USING 643 ;S,Lambdap1,Lambdae1,Fail
6430  IMAGE 4X,DD.2DX,DD.DD,10X,3D.DD,12X,DD.DD
6435  RETURN
6440  !-------------------------------------------------------------------
7000  !Subroutine ELASTIC calculates elastic and combined failure loads.
7005  !-------------------------------------------------------------------
7010 Elastic:       !
7015  Sum=0
7020  Axtotal=0
7025  FOR Count=1 TO N+1
7030    IF Count=1 THEN
7035      Phi=(Colprop(2,S,1)/(2*H1))*(1/(Beamprop(2,S,1)/Beamprop(1,S,1)))
7040    ELSE
7045      IF Count=N+1 THEN
7050        Phi=(Colprop(2,S,Count)/(2*H1))*(1/(Beamprop(2,S,Count-1)/Beamprop(1
,S,Count-1)))
7055      ELSE
7060        Phi=(Colprop(2,S,Count)/(2*H1))*(1/(Beamprop(2,S,Count-1)/Beamprop(1
,S,Count-1)+Beamprop(2,S,Count)/Beamprop(1,S,Count)))
7065      END IF
7070    END IF
7075    Ua=(Axial(2,Count)+Axial(1,Count))/Axial(1,Count)
7080    Axtotal=Axtotal+Axial(1,Count)
7085    IF S=Storey-1 THEN
7090      IF Base=1 THEN Subt=Colprop(2,S,Count)/(4+1.5*Ua*Phi)
7095      IF Base=0 THEN Subt=Colprop(2,S,Count)*(3+Phi)/(3+4*Phi)
7100    ELSE
7105      Subt=Colprop(2,S,Count)/(1+Ua*Phi)
7110    END IF
7115    Sum=Sum+Subt
7120  NEXT Count
7125  IF S=Storey-1 THEN
7130    Pstorey=12*E*Sum/(H1^2)
7135  ELSE
7140    Pstorey=12*E*Sum/((2*H1)^2)
7145  END IF
7150  Lambda=Pstorey/Axtotal
7155  RETURN
7160  !-------------------------------------------------------------------
8000  !Subroutine INTERACTION calculates the Elastic/Plastic failure load.
8005  !-------------------------------------------------------------------
8010 Interaction:   !
8015  Slendratio=100
8020  Mag=1/(1-((.4*Pstorey)/Pstorey))
8025  MAT Mombase= Mombase(Lambda#,4*Mag)
8030  MAT Momcol= Momcol*(Lambda#,4*Mag)
8035  MAT Mombase= Mombase*(Lambda#,4*Mag)
8040  Lambda#=0
```

```
8045  FOR Count=1 TO N
8050    Bigmom=MAX(ABS(Hmombm(1,Count)),ABS(Hmombm(2,Count)),ABS(Hmombm(3,Count)
))
8055    Lambda=(Beamprop(1,8,Count)/Fpb)*(BIGmom/Beamprop(2,8,Count))*(Beamprop(
8,8,Count)/BGR(Beamprop(2,8,Count)/Beamprop(4,8,Count)))
8060    IF Lambda>Lambda04 THEN Lambda04=Lambda
8065  NEXT Count
8070  FOR Count=1 TO N+1
8075    Subt=GGR(1+4*Fpr*Axial(1,Count)*.4*Lambdao*Colprop(2,8,Count)/(Hmomcol(1
,Count)^2))*.5+.5
8080    Lambda=(ABS(Hmomcol(1,Count))/Colprop(2,8,Count))*(Colprop(1,8,Count)/Fp
c)*Subt
8085    IF Lambda>Lambda04 THEN Lambda04=Lambda
8090    IF 8=Storey-1 THEN
8095      IF Base=0 THEN
8100        Subt=GGR(1+4*Fpc*Axial(1,Count)*.4*Lambdao*Colprop(2,8,Count)/(Hmomb
ase(Count)^2))*.5+.5
8105        Lambda=(ABS(Hmombase(Count))/Colprop(2,8,Count))*(Colprop(1,8,Count)
/Fpc)*Subt
8110      'F Lambda>Lambda04 THEN Lambda04=Lambda
8115      END IF
8120    END IF
8125  NEXT Count
8130  Alpha=.6/(1-.6*(((700-Lambda04)/700)^3))
8135  FOR Count=1 TO N
8140    FOR Loc=1 TO 3
8145      Mtotal=ABS(Alpha*Lambdao*Servmombm(Loc,Count)+(Hmombm(Loc,Count)/(.4*M
eg))*Alpha*((1/(1-Alpha))-1))
8150      Ratio=1/((Beamprop(5,8,Count)/Fpb)*(Mtotal/Beamprop(2,8,Count)))
8155      IF Ratio<Blendratio THEN Blendratio=Ratio
8160    NEXT Loc
8165  NEXT Count
8170  FOR Count=1 TO N+1
8175    FOR Loc=1 TO 2
8180      IF 8=1 THEN
8185        Mtotal=ABS(Alpha*Lambdao*Servmomcol(1,Count)+(Hmomcol(1,Count)/(.4*M
eg))*Alpha*((1/(1-Alpha))-1))
8190      ELSE
8195        Mtotal=ABS(Alpha*Lambdao*Servmomcol(Loc,Count)+(Hmomcol(Loc,Count)/(
.4*Meg))*Alpha*((1/(1-Alpha))-1))
8200      END IF
8205      Subt=GGR(1+4*Fpc*Lambdao*Axial(1,Count)*Colprop(2,8,Count)/((Colprop(5,
8,Count)/BGR(Colprop(2,8,Count)/Colprop(4,8,Count)))*Mtotal)^2)
8210      Ratio=1/((Colprop(5,8,Count)/Fpc)*(Mtotal/Colprop(2,8,Count))*Subt)
8215      IF Ratio<Blendratio THEN Blendratio=Ratio
8220    NEXT Loc
8225    IF 8=Storey-1 THEN
8230      IF Base=0 THEN
8235        Mtotal=WUB(Alpha*Lambdao*Servmombase(Count)+(Hmombase(Count)/(.4*Meg
))*Alpha*((1/(1-Alpha))-1))
8240        Subt=GGR(1+4*Fpc*Lambdao*Axial(1,Count)*Colprop(2,8,Count)/((Colprop(
5,8,Count)/BGR(Colprop(2,8,Count)/Colprop(4,8,Count)))*Mtotal)^2)
8245        Ratio=1/((Colprop(5,8,Count)/Fpc)*(Mtotal/Colprop(2,8,Count))*Subt)
8250        IF Ratio<Blendratio THEN Blendratio=Ratio
8255      END IF
8260    END IF
8265  NEXT Count
8270  IF Cycle=1 THEN
8275    Alpha1=Alpha
8280    Lambda1=Lambdao
8285    Lambdap1=Lambdap
8290    Angle=Alpha1*Lambdao/Lambdap
8295  END IF
8300  IF Cycle=1 THEN
8305    Blendratio1=Blendratio
8310    Y.Axis=Angle*Blendratio1
```

```
8315  ELSE
8320    Y_axis=(Alpha1*Lambdao/Lambdap)
8325  END IF
8330  IF Angle>1.5 THEN
8335    Io=.764*(.92-Y_axis)
8340    Fail=(1/(Io/Angle+1))*Lambdap1
8345  ELSE
8350    X1=INT(Angle*10)
8355    X2=X1+1
8360    Y1=INT(Y_axis*10)+1
8365    Y2=Y1+1
8370    Inter1=(((10*Angle)-X1)/(X2-X1))*(Curve(Y1,X2)-Curve(Y1,X1))+Curve(Y1,X1
        )
8375    Inter2=(((10*Angle)-X1)/(X2-X1))*(Curve(Y2,X2)-Curve(Y2,X1))+Curve(Y2,X1
        )
8380    Inter3=((10*Y_axis)-Y1)/(Y2-Y1))*(Inter2-Inter1)+Inter1
8385    Fail=Inter3*Lambdap1
8390  END IF
8395  IF Cycle=1 THEN
8400    FOR Count=1 TO N
8405      Beamprop(1,8,Count)=Beamprop(1,8,Count)/Blendratio1
8410      Colprop(1,8,Count)=Colprop(1,8,Count)/Blendratio1
8415    NEXT Count
8420    FOR Count=1 TO N+1
8425      IF Count=N+1 THEN
8430        Load(9,Count)=Load(8,Count)*Blendratio1
8435      ELSE
8440        Load(8,Count)=Load(8,Count)*(Blendratio1^2)
8445      END IF
8450      Thrust(8,Count)=Thrust(8,Count)*Blendratio1
8455      Axial(2,Count)=Axial(2,Count)*Blendratio1
8460      Holdax(Count)=Axial(1,Count)
8465    NEXT Count
8470    Cycle=2
8475    FOR Count=1 TO N
8480      Becondition(8,Count)=0
8485    NEXT Count
8490    FOR Count=1 TO N+1
8495      FOR Loc=1 TO 2
8500        IF Beforev=1 THEN
8505          IF base=0 THEN
8510            Ccondition(1,8,Count)=0
8515            Ccondition(2,8,Count)=1
8520          ELSE
8525            Ccondition(Loc,8,Count)=1
8530          END IF
8535        ELSE
8540          Condition(Loc,8,Count)=1
8545        END IF
8550      NEXT Loc
8555    NEXT Count
8560    MAT Colmom= (0)
8565    MAT Beamom= (0)
8570    GOTO 375
8575  ELSE
8580    FOR Count=1 TO N
8585      Beamprop(1,8,Count)=Beamprop(1,8,Count)*Blendratio1
8590      Colprop(1,8,Count)=Colprop(1,8,Count)*Blendratio1
8595    NEXT Count
8600    FOR Count=1 TO N+1
8605      IF Count=N+1 THEN
8610        Load(8,Count)=Load(8,Count)/Blendratio1
8615      ELSE
8620        Load(8,Count)=Load(8,Count)/(Blendratio1^2)
8625      END IF
8630      Thrust(8,Count)=Thrust(8,Count)/Blendratio1
```

```
8635        Axial(2,Count)=Axial(2,Count)/Blandratio
8640        Axial(1,Count)=HoldAxl(Count)
8645      NEXT Count
8650    END IF
8655    Factor(S)=Fail
8660    RETURN
8665    !-------------------------------------------------------------------------
9000    !Subroutine GRAPHIC draws structure showing collapse mechanism.
9005    !-------------------------------------------------------------------------
9010  Graphics:     !
9015    GINIT
9020    PLOTTER IS 3,"INTERNAL"
9025    GRAPHICS ON
9030    GCLEAR
9035    PRINT FNClear$
9040    Xsum=0
9045    FOR No=1 TO Bays(S)
9050      Xs=Beamprop(1,S,No)
9055      Xsum=Xsum+Xs
9060    NEXT No
9065    IF Bays(S)<=2 THEN Scale=((Bays(S)/2.5)*125)/Xsum
9070    IF Bays(S)>2 THEN
9075      IF Bays(S)<=5 THEN Scale=((Bays(S)/5)*125)/Xsum
9080      IF Bays(S)>5 THEN
9085        IF Bays(S)<=8 THEN
9090          Scale=((Bays(S)/8)*125)/Xsum
9095        ELSE
9100          Scale=((Bays(S)/10)*125)/Xsum
9105        END IF
9110      END IF
9115    END IF
9120    IF Bays(S)<=10 THEN
9125      Xstart=(130-Scale*Xsum)/2
9130      Ystart=50
9135      IF S>1 THEN
9140        MOVE 65,Ystart+(Scale*Colprop(1,S-1,1)/2)+11
9145      ELSE
9150        MOVE 65,Ystart+16
9155      END IF
9160      LORG 4
9165      LABEL "STOREY NO.";S;" :Collapse mechanism"
9170      MOVE Xstart,Ystart
9175      DRAW Xstart+(Scale*Xsum),Ystart
9180      IF S=Storey-1 THEN
9185        IF S>1 THEN
9190          VIEWPORT Xstart-10,Xstart+(Scale*Xsum)+10,Ystart-(Scale*Colprop(1,S,
1))-10,Ystart+(Scale*Colprop(1,S-1,1)/2)+10
9195        ELSE
9200          VIEWPORT Xstart-10,Xstart+(Scale*Xsum)+10,Ystart-(Scale*Colprop(1,S,
1))-10,Ystart+15
9205        END IF
9210      ELSE
9215        IF S>1 THEN
9220          VIEWPORT Xstart-10,Xstart+(Scale*Xsum)+10,Ystart-(Scale*Colprop(1,S,
1)/2)-10,Ystart+(Scale*Colprop(1,S-1,1)/2)+10
9225        ELSE
9230          VIEWPORT Xstart-10,Xstart+(Scale*Xsum)+10,Ystart-(Scale*Colprop(1,S,
1)/2)-10,Ystart+15
9235        END IF
9240      END IF
9245      FRAME
9250    ELSE
9255      Xstart=5
9260      Ystart=75
9265      MOVE 65,46
9270      LORG 6
```

```
9275   LABEL "SECTION NO.";S;":Collapse mechanism"
9280   MOVE Xstart,Ystart
9285   DRAW Xstart+128,Ystart
9290   LINE TYPE 4
9295   DRAW Xstart+128,Ystart
9300   MOVE Xstart-5,Ystart-50
9305   DRAW Xstart,Ystart-50
9310   LINE TYPE 1
9315   DRAW (Scale*Xmum)-120,Ystart-50
9320   !VIEWPORT 0,132,0,100
9325   FRAME
9330 END IF
9335 IF B>1 THEN
9340   MOVE Xstart+128+(Scale*Colprop(1,B-1,1)/2)
9345   LORG 4
9350   LABEL "o"
9355   IMOVE 0,6
9360 ELSE
9365   MOVE Xstart,Ystart
9370 END IF
9375 IF B=Storey-1 THEN
9380   LINE TYPE 10
9385   DRAW Xstart,Ystart-(Scale*Colprop(1,B,1))
9390   LINE TYPE 1
9395 ELSE
9400   DRAW Xstart,Ystart-(Scale*Colprop(1,B,1))/2)
9405   IMOVE 0,2
9410   LORG 4
9415   LABEL "o"
9420 END IF
9425 X1=0
9430 FOR Beam=1 TO Bays(B)
9435   CSIZE 5
9440   Xi=(Scale*Beamprop(i J,Beam))+X1
9445   Xincr=Xstart+X1
9450   IF Beam<=10 THEN
9455     IF B>1 THEN
9460       MOVE Xincr,Ystart+(Scale*Colprop(1,B-1,1)/2)
9465       LORG 4
9470       LABEL "o"
9475       IMOVE 0,6
9480     ELSE
9485       MOVE Xincr,Ystart
9490     END IF
9495     IF B=Storey-1 THEN
9500       LINE TYPE 10
9505       DRAW Xincr,Ystart-(Scale*Colprop(1,B,1))
9510       LINE TYPE 1
9515       MOVE Xincr-(Scale*Beamprop(1,B,Beam)/2),Ystart-(Scale*Colprop(1,B,1
)/4
9520     ELSE
9525       DRAW Xincr,Ystart-(Scale*Colprop(1,B,1)/2)
9530       IMOVE 0,2
9535       LORG 4
9540       LABEL "o"
9545       MOVE Xincr-(Scale*Beamprop(1,B,Beam)/2),Ystart-(Scale*Colprop(1,B,1
/2)-4
9550     END IF
9555     IF Bays(B)<=8 THEN
9560       CSIZE 4
9565     ELSE
9570       CSIZE 3
9575     END IF
9580     LABEL "WAY";Beam
9585   ELSE
9590     IF Beam=11 THEN
```

```
9595        IF S>1 THEN
9600          MOVE 5,(Ystart-50)+(Scale*Colprop(1,S-1,1)/2)
9605          LORG 4
9610          LABEL "n"
9615          IMOVE 0,6
9620        ELSE
9625          MOVE 5,Ystart-50
9630        END IF
9635        IF S<Storey-1 THEN
9640          LINE TYPE 10
9645          DRAW 5,(Ystart-50)-(Scale*Colprop(1,S,1))
9650          LINE TYPE 1
9655        ELSE
9660          DRAW 5,(Ystart-50)-(Scale*Colprop(1,S,1))/2)
9665          IMOVE 0,2
9670          LORG 6
9675          LABEL "n"
9680        END IF
9685      END IF
9690      IF S>1 THEN
9695        MOVE Xincr-120,(Ystart-50)+(Scale*Colprop(1,S-1,1))/2)
9700        LORG 4
9705        LABEL "n"
9710        IMOVE 0,6
9715      ELSE
9720        MOVE Xincr-120,(Ystart-50)
9725      END IF
9730      IF S<Storey-1 THEN
9735        LINE TYPE 10
9740        DRAW Xincr-120,(Ystart-50)-(Scale*Colprop(1,S,1))
9745        LINE TYPE 1
9750        MOVE (Xincr-120)-(Scale*Beamprop(1,S,Beam)/2),(Ystart-50)-(Scale*Col
prop(1,S,1))-4
9755      ELSE
9760        DRAW Xincr-120,(Ystart-50)-(Scale*Colprop(1,S,1))/2)
9765        IMOVE 0,2
9770        LORG 6
9775        LABEL "n"
9780        MOVE (Xincr-120)-(Scale*Beamprop(1,S,Beam)/2),(Ystart-50)-(Scale*Col
prop(1,S,1)/2)-4
9785      END IF
9790      IF Bays(B)=B THEN
9795        CSIZE 4
9800      ELSE
9805        CSIZE 5
9810      END IF
9815      LABEL "BAY";Beam
9820    END IF
9825  NEXT Beam
9830  FOR No=1 TO Ninces(S)
9835    Xincr=Xstart
9840    IF Location(1,S,No)<>0 THEN
9845      Number=Location(1,S,No)
9850    ELSE
9855      Number=Location(2,S,No)
9860    END IF
9865    IF INT(Number)>1 THEN
9870      FOR Beam=1 TO INT(Number)-1
9875        Xi=Beamprop(1,S,Beam)*Scale
9880        Xincr=Xi+Xincr
9885      NEXT Beam
9890    END IF
9895    IF INT(Number)<=10 THEN
9900      IF Bays(B)<=10 THEN
9905        MOVE Xincr,50
9910      ELSE
```

```
9915          MOVE Xincr,75
9920        END IF
9925      ELSE
9930        MOVE Xincr-120,25
9935      END IF
9940      IF Location(i,S,No)<>0 THEN
9945        IF INT(FRACT(Number)*10.5)=1 THEN IMOVE 2,0
9950        IF INT(FRACT(Number)*10.5)=2 THEN IMOVE (Scale*Bsmprop(i,S,INT(Number
))/2),0
9955        IF INT(FRACT(Number)*10.5)=3 THEN IMOVE (Scale*Bsmprop(i,S,INT(Number
)))-2,0
9960      ELSE
9965        IF INT(FRACT(Number)*10.5)=1 THEN IMOVE 0,-2
9970        IF INT(FRACT(Number)*10.5)=2 THEN IMOVE 0,2
9975        IF INT(FRACT(Number)*10.5)=3 THEN IMOVE 0,2-(Scale*Colprop(1,S,INT(Num
ber)))
9980        IF INT(FRACT(Number)*10.5)=6 THEN IMOVE 0,2-(Scale*Colprop(1,S,INT(Num
ber))/2)
9985        IF INT(FRACT(Number)*10.5)=7 THEN IMOVE 0,(Scale*Colprop(1,S,INT(Numbe
r))/2)+2
9990      END IF
9995      LORG 5
10000     CSIZE 2,1
10005     IF INT(FRACT(Number)*10.5)>5 THEN
10010       LABEL "X"
10015       LABEL "X"
10020       LABEL "X"
10025     ELSE
10030       LABEL "O"
10035       IMOVE 0.2
10040       CSIZE 1.1
10045       LABEL "O"
10050       IMOVE 0.2
10055       CSIZE 3,.5
10060       IF Location(i,S,No)<>0 THEN
10065         LORG 4
10070       ELSE
10075         IF INT(FRACT(Number)*10.5)=1 THEN
10080           LORG 3
10085         ELSE
10090           IF INT(FRACT(Number)*10.5)=2 THEN
10095             LORG 7
10100           ELSE
10105             LORG 8
10110           END IF
10115         END IF
10120       END IF
10125       LABEL No
10130     END IF
10135   NEXT No
10140   DISP USING 10145
10145   IMAGE @,"Do you want a Hard Copy,_____(Y/N)"
10150   INPUT Que$
10155   IF Que$="Y" THEN
10160     IF Bavs(6)<=8 THEN
10165       DUMP GRAPHICS #701
10170     END IF
10175     IF Bavs(6)>8 THEN
10180       DUMP DEVICE IS 701,EXPANDED
10185       DUMP GRAPHICS
10190     END IF
10195   END IF
10200   GCLEAR
10205   RETURN
10210   !-------------------------------------------------------------------------
10215 END
11000 DEF FNC1$a#
11005   OUTPUT 2;K$
11010   RETURN Dummy$
11015 FNEND
```

## Imbed Trace

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