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

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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 Faller

17 March 1986

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 [illegible]
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

ℓ	beam length (bay width)
Mp	moment capacity of section at formation of plastic hinge
Mpb	beam plastic moment capacity
Mpc	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
α	factor to take into account any deviation from full symmetry in structure or loading
γ	load parameter (load factor)
γ_c	critical load factor
γ_f	load factor at failure
γ_o	elastic buckling load factor
γ_p	rigid plastic collapse load factor

γ_q load factor at formation of hinge q

Δ lateral displacements

Δ_f first order lateral displacements

Δ_s second order lateral displacements

θ rotational displacements

λ slenderness ratio of member (L/r)

λ_ℓ 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"

ϕ 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 structure, 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 Δ_f would result. If the gravity loads were now applied to the structure in its deformed position these loads would interact with the displacements Δ_f to produce secondary moments, called the P-Delta moments. These moments in turn cause the structure to d... further into a new equilibrium position Δ_s. The first-order deflection has thus been magnified to Δ_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' concept does not always ensure frame stability. In the light of these [illegible] it would appear that the secondary effects should explicitly be taken 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 deflections 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 Δ_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 :

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

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 :

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

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 concrete 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 effects 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

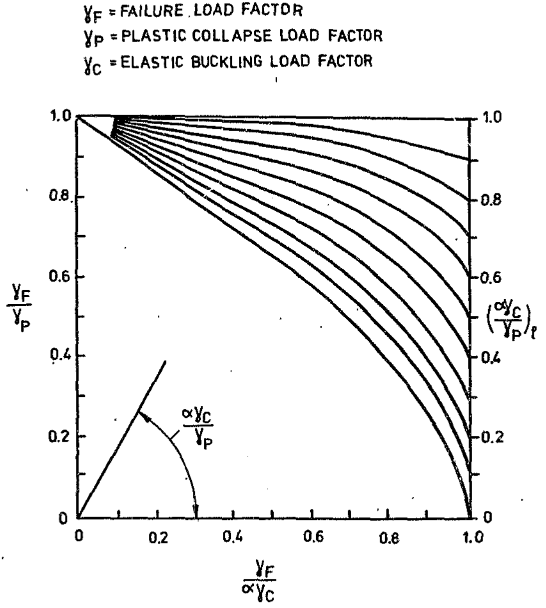

Figure 1. Basic interaction curves. Scholz (198[illegible])

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.

- thn 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

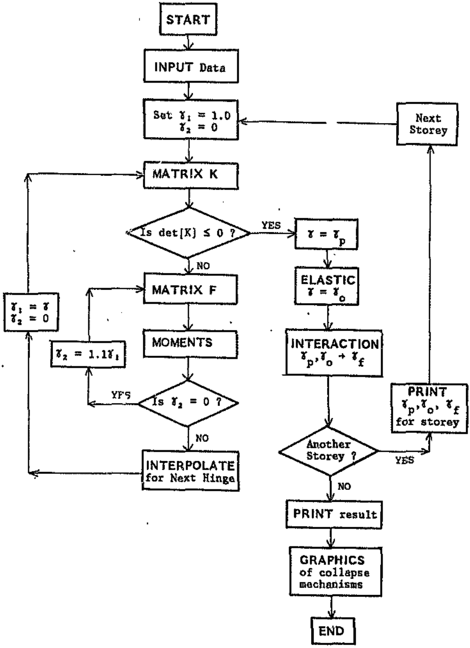

Figure 2. Flow Chart of Computer Program

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 γ and multiplying all loads by γ.

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, θ and the column chord slope Δ/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, ℓ_i = length of beam i and h = storey height.

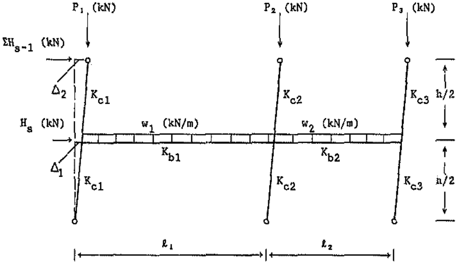

Figure 3. Typical internal storey model.

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 (Δ/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 (Δ_1 and Δ_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.3 shows H_s applied at floor level and ΣH_{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, γ.

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 γ and then in a second cycle for the forces factored by 1.1γ. 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).

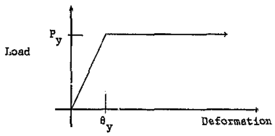

Figure 4. Idealised elastic-plastic response.

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 γ, 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 γ_1 and γ_2, we can predict the load factor at formation of a plastic hinge at this section by using the following relationship:

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

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 γ and accordingly the relationship to predict the load parameter at formation of a plastic hinge in a column member has to be adjusted to:

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

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 γ throughout the storey framework then gives the required γ_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 γ_1 equal to the γ 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 γ_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 γ_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 γ_o 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) :

$$\gamma_o \Sigma P = 12E/h^2 \sum_1^m [I_c/(1+U\psi)] \qquad \ldots[\text{ Eq.5 }]$$

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 ψ = 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 $\gamma_o \Sigma P$ gives the total load that will be applied to the storey when elastic buckling occurs. The elastic buckling load factor γ_o is then obtained by dividing $\gamma_o \Sigma P$ by the total load ΣP acting on that storey.

3.7 ELASTIC-PLASTIC FAILURE LOAD

Having calculated γ_p and γ_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 γ 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 $f_m = (1/(1 - 0.4Po/Po)) = 1.667$. 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 $\alpha = (0.4/(1-0.6 \times (700 - \lambda_{0.4H}/700)^2))$

- 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 λ_ℓ, the member slenderness ratio for the "limiting frame", at the critical sections. In the program λ_ℓ 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 λ/λ_ℓ

- Calculate $(\alpha\gamma_o/\gamma_p)$ and $(\alpha\gamma_o/\gamma_p)_\ell$. To calculate the value $(\alpha\gamma_o/\gamma_p)_\ell$ we can either multiply $(\alpha\gamma_o/\gamma_p)$ by λ/λ_ℓ, as derived by Scholz (1981), or we can use the value λ_ℓ to re-proportion the frame and repeat the calculations for γ_p and γ_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 $(\alpha\gamma_o/\gamma_p)_\ell$ and read off the value for γ_f. If the value of $(\alpha\gamma_o/\gamma_p) >$ 1.5 then the program uses the relationship $\gamma_f/\gamma_p = 1/(1+(i_o\gamma_p/\alpha\gamma_o))$ to calculate γ_f, where $i_o = 0.76[0.92 - (\alpha\gamma_o/\gamma_p)_\ell]$. 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 γ_f by interpolation.

3.8 PRINTOUT OF RESULTS

After calculating γ_f for a storey the program prints out the values of γ_p, γ_o and γ_f for that storey. If there are more storeys to analyse the program then repeats the procedure for the next storey. When γ_f for all storeys is known the program compares the failure load factors γ_f of all the storeys. The lowest γ_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.

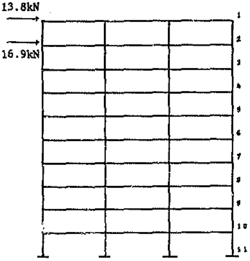
- Take each storey in turn and adjust a member or members until the storey load factor is within acceptable limits above the design load factor.

- 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. .. 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 el-- in the structure and thus it is impossible to identify exactly wh.. .embers 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

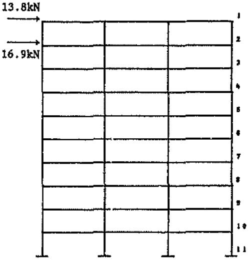
The Structure Collapse Load Factor = 1.28

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 γ_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 γ_q, namely the hinge pattern before and the hinge pattern after the hinge q has formed. A critical load parameter γ_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 γ_q and load factor γ_c before and after hinge formation, are printed out.

The values of γ_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 γ_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 γ_c. These members would then be adjusted more or less proportionately to the magnitude of the change in γ_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 γ_c for each of the 6 hinges it was found that for hinges no.1 and 4 the change in γ_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.

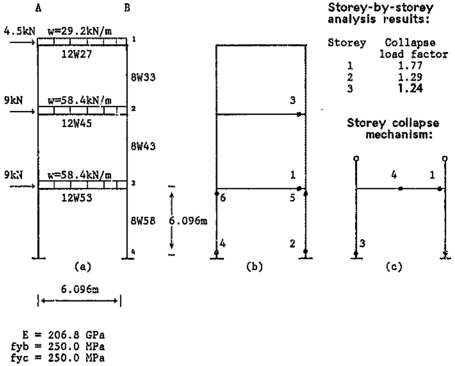


Figure 7. Comparison with frame design by Vijakkhana et al.(1974)

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.

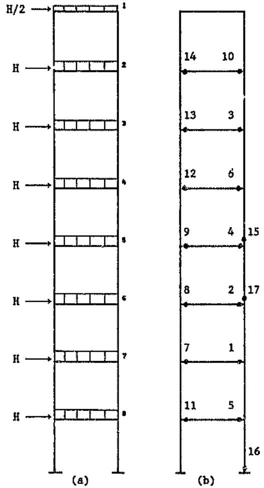
It is obvious too, from the collapse mechanism of this storey, that beam 3 or the lower columns should be adjusted to increase the storey failure load factor. Following Vijakkhana et al., the lower columns are increased to 10W54 sections and storey No.3 re-analysed. The failure load factor for storey 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 [illegible]ture has to be re-analysed once a member adjustment is made. [illegible] becomes more sever[illegible] the larger the frame being analysed. T[illegible] by-storey approach thus gives an accurate result wh[illegible] 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).

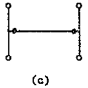
Using the storey-by-storey interaction method a failure load factor of 1.03 was obtained for the same structure. The storey failure load factors are given in Fig.8(c) together with a typical storey plastic collapse



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
fyb = 235.5 MPa
fyc = 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 beam 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:

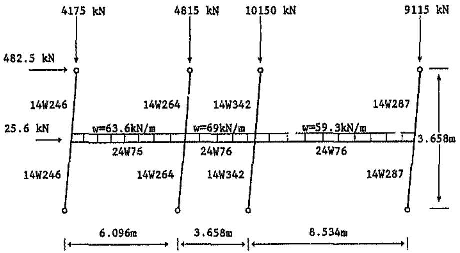
The first comparison with a storey sub-assemblage model is taken from a series of Lehigh University lecture notes (1965) titled "Plastic Design of Multi-Story Frames". The storey sub-assemblage under consideration is shown in Fig.9, which gives the dimensions, loading and member sizes of the arrangement. This sub-assemblage represents the twentieth storey of a twency-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 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 1.0 gravity load	1.35
Interaction method on frame revised by Emkin & Litle	—	1.0 wind load 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 rigorous 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 Notes (1965) 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.

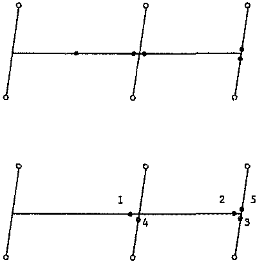
This example is discussed here because when analysed by the program presented in this paper a different result was obtained. Instead of a failure load of 1.30 for the structure the programmed version gave a storey failure load of 1.25. The same interaction method was used in both analyses and hence should 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-



(a) Sway mechanism used by Scholz in hand computation.

(b) Failure mechanism identified by computer

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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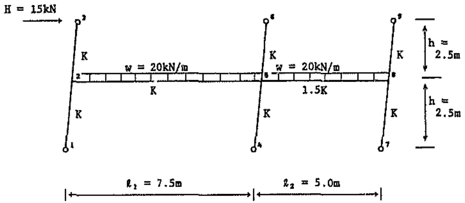
APPENDIX A

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):

$$\begin{matrix} F_2: \\ M_2: \\ F_3: \\ M_5: \\ M_8: \end{matrix} \overbrace{\begin{bmatrix} 2.88 & & -1.44 & & \\ & 10 & -1.20 & 2 & \\ -1.44 & -1.20 & 1.44 & -1.20 & -1.20 \\ & 2 & -1.20 & 16 & 3 \\ & & -1.20 & 3 & 12 \end{bmatrix}}^{\text{stiffness}} K \overbrace{\begin{Bmatrix} U_2 \\ \theta_2 \\ U_3 \\ \theta_5 \\ \theta_8 \end{Bmatrix}}^{\text{defl.}} = \overbrace{\begin{Bmatrix} 0 \\ 4.688w \\ H \\ -2.605w \\ 2.083w \end{Bmatrix}}^{\text{forces}}$$

det K = |K| = 2390

$\gamma = 1.0$	$\gamma = 1.1$
$KU_2 = 22.74$	$KU_2 = 25.01$
$KU_3 = 45.46$	$KU_3 = 50.00$
$K\theta_2 = 15.24$	$K\theta_2 = 16.76$
$K\theta_5 = -2.04$	$K\theta_5 = -2.24$
$K\theta_8 = 1.58$	$K\theta_8 = 1.74$

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

$$M_{52} = 2K\theta_2 + 4K\theta_5 + w\ell^2/12 = 116.08 \text{ kNm} \qquad M_{52} = 127.70 \text{ kNm}$$

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

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

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

Revised structure stiffness matrix (1 hinge) :

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

det |K| = 1288

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

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

$$\gamma_2 = \frac{2.96 - 2.69}{-95.0 + 89.8} * -100 + \frac{2.69 * -95.0 + 2.96 * 89.8}{-95.0 + 89.8} = 3.22$$

Revised member stiffness matrix for column member 5-4 :

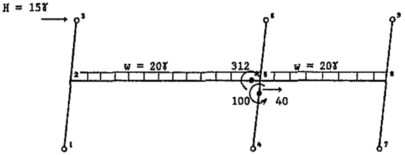
$$[K]_{54} = [0]$$

Revised structure stiffness matrix (2 hinges):

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

det |K| = 599.5

Diagrammatic representation :



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

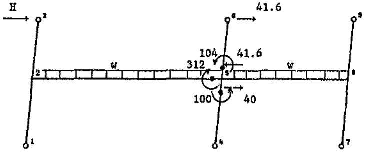
Load at formation of third hinge : $\gamma_3 = 3.36$

Structure stiffness matrix revised for 3rd hinge:

$$
\begin{matrix} F_2: \\ M_2: \\ F_3: \\ M_9: \\ M_8: \end{matrix}
\begin{bmatrix}
1.92 & & -0.96 & & \\
 & 9 & -1.20 & & \\
-0.96 & -1.20 & 0.96 & & -1.20 \\
 & & & 6 & 3 \\
 & & -1.20 & 3 & 12
\end{bmatrix}
K
\begin{Bmatrix} U_2 \\ \theta_2 \\ U_3 \\ \theta_9 \\ \theta_8 \end{Bmatrix}
=
\begin{Bmatrix} Mpu/h-40 \\ 7.031w-156 \\ H-Mpu/h \\ Mpu-212+2.083w \\ -2.083w \end{Bmatrix}
$$

det |K| = 199

Diagrammatic representation :



4th hinge forms in member 7-8 at 8 : $\gamma_4 = 3.48$

5th hinge forms in member 8-9 at 8 : $\gamma_5 = 3.56$

6th hinge forms in member 2-5 at midspan : $\gamma_6 = 3.77$

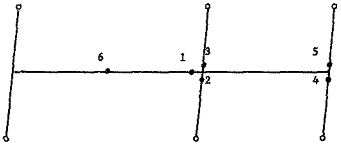
Structure stiffness matrix revised for 6 plastic hinges:

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

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

i.e. $\gamma_p = 3.77$

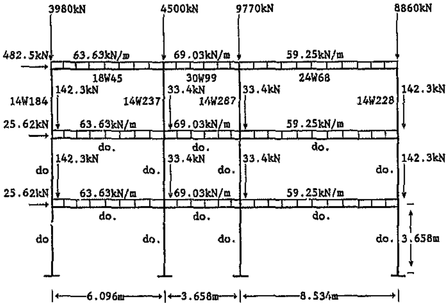
Sequence of plastic hinge formation:



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 & EMKIN (Program COLLAPSE2)

Young's Modulus =199800000 kN/m2
Material Yield Stresses: Beams=248200kN/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	.008541	.227
	2	3.658	.001660	.005113	.018779	.376
	3	8.534	.000755	.002876	.012902	.302
2	1	6.096	.000293	.001468	.008541	.227
	2	3.658	.001660	.005113	.018779	.376
	3	8.534	.000755	.002876	.012902	.302
3	1	6.096	.000293	.001468	.000541	.227
	2	3.658	.001660	.005113	.018779	.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	3980.00	.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

Storey	Col.	Axial load(kN)	I(m4)	zp(m3)	Area(m2)	y_max(m)
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

Storey	Col.	Axial load(kN)	I(m4)	zp(m3)	Area(m2)	y_max(m)
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

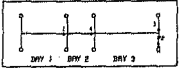
Storey	Hor.load(kN)	Beam	Udl(kN/m)
1	482.50	1	63.63
		2	69.03
		3	59.25
2	25.62	1	63.63
		2	69.03
		3	59.25
3	25.62	1	63.63
		2	69.03
		3	59.25

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	FEM2h1	For beam initially with 1 hinge at RHS ;
		FEM2h1 = 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
Ccondition (2,50,21)	Record of the number of hinges in each column		
	No hinges:	Ccondition	= 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)	
Comparel (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:

```
5       !Program name is COLLAPSE
10      !------------------------
15      !
20      !
25      !This program combines the Plastic and Elastic Collapse load factors to ob
tain the Overall Collapse Load Factor for the structure.
30      !The load factors are calculated on a Storey-by-Storey basis and the criti
cal load factor is the lowest Storey Load Factor.
35      !
40      OPTION BASE 1
45      INTEGER S,N                    !No. of storeys,No. of bays.
50      INTEGER Storey,Beam,Col,No     !Counters in subr. INPUT
55      INTEGER Dim                    !Redim. of storage arrays in subr. INPUT
60      INTEGER Beamno,Colno           !Location of plastic hinges in subr. NEXT HINGE
65      INTEGER Loc,Hingeloc           !Location of plasic hinge in subr. NEXT HINGE
70      INTEGER Hinge                  !Hinge no. counter in subroutine NEXT HINGE
75      INTEGER X                      !Variable for moment storage in subr. MOMENTS
80      INTEGER Cycle                  !Denotes cycle 1 or 2
85      INTEGER Edit,Ed                !Indicates initial input or editing
90      REAL L,Ib,Ic,Mpb,Mpc,H,Ks      !Length,Mom.inertia,Plastic cap,Height in INPUT
95      REAL H1,H2                     !Half height lower cols., upper cols.
100     REAL A,B,C,D,U,G,M,J,P,Q,R     !Intermediate results in subr. MATRIX [K]
105     REAL E                         !Young's modulus
110     REAL Fy,Fp,Py,Fac            !Yield stress and squash load calcs.
115     REAL Total,Test                !Variables to check if DET K ill-conditioned
120     REAL Lambda,Lambdap            !Intermediate load factors in subr. NEXT HINGE
125     REAL Lambdai,Lambda2           !Load factor on 1st and 2nd iterations
130     REAL Udl,Fh                    !Vertical and horizonal loads in subr. INPUT
135     REAL Ml,Mr                     !FEM's left and right
140     REAL Fem2hl,Fem2hr             !FEM's left and right at form. of 2nd hinge
145     REAL Hor,Hor1,Hor2,Shor        !Horizontal forces for calcs in  MATRIX (F)
150     REAL M1,M2,M3,Mlo,Mup          !Beam and col. moments in subr. MOMENTS
155     REAL Mom1,Mom2,Mpl             !Moments 1st and 2nd iterations in NEXT HINGE
160     REAL Bmplastic,Colplastic      !Indicates hinge in beam or col. in NEXT HINGE
165     REAL Mbase,Base                !Column moments at base; Base=0 for fixed base
170     REAL Sum,Axtotal,Phi,Us,Subt    !Elastic collapse load in ELASTIC
175     REAL Pstorey,Lambdae,Mag        !Elastic collapse load in ELASTIC
180     REAL Lambda04,Bigmom,Alpha      !Identifying highest slenderness with 0.4H
185     REAL Slendratio,Ratio,Mtotal    !Identifying lowest slenderness ratio
190     REAL Alphai,Slendratiol         !Alpha and slend ratio for actual frame
195     REAL Io,Fail,Inter1,Inter2,Inter3     !Extracting values from curves
200     REAL Angle,Y_axis,X1,X2,Y1,Y2         !Extracting values from curves
205     REAL Axl                              !Axial load
210     !
215     !
220     INTEGER Bays(50),Hinges(50),Bcondition(50,20),Ccondition(2,50,21)
225     DIM Beamprop(5,50,20),Colprop(5,50,21)
230     DIM Load(50,21),Thrust(50,21),F(23),Df(23)
235     DIM K_inv_inv(23,23),Differ(23,23),Compare(23,23),Comparei(23,23)
240     DIM K(23,23),Kt(23,23),Kinv(23,23)
245     DIM Beamom(3,51,21),Colmom(2,51,21),Endmom(2,20),Basemum(2,21)
250     DIM Que$[1],Describe$[80]
255     DIM Axial(2,21),Hmombm(3,20),Hmomcol(2,21),Hmombase(21),Holdaxl(21)
260     DIM Servmombm(3,20),Servmomcol(2,21),Servmombase(21)
265     DIM Curve(10,15),Factor(50)
270     DIM Location(2,50,150)
275     !
280     !
285     DATA .09,.19,.27,.33,.40,.45,.49,.52,.56,.59,.61,.63,.65,.67,.68
290     DATA .10,.19,.28,.35,.41,.47,.52,.55,.59,.61,.64,.66,.68,.70,.71
295     DATA 0,.20,.29,.36,.43,.49,.54,.57,.61,.64,.66,.69,.70,.72,.73
300     DATA 0,0,.30,.38,.45,.51,.56,.60,.64,.67,.70,.72,.73,.75,.77
305     DATA 0,0,0,.40,.48,.55,.60,.64,.68,.71,.73,.75,.77,.79,.80
310     DATA 0,0,0,0,.50,.57,.63,.68,.72,.75,.77,.79,.81,.82,.83
315     DATA 0,0,0,0,0,.60,.67,.72,.76,.79,.82,.84,.85,.86,.87
320     DATA 0,0,0,0,0,0,.70,.76,.80,.83,.86,.88,.89,.90,.91
```

```
325   DATA 0,0,0,0,0,0,0,.80,.84,.87,.89,.91,.93,.94,.95
330   DATA 0,0,0,0,0,0,0,0,.90,.92,.94,.95,.96,.97,.98
335   READ Curve(*)
340   MAT Ccondition= (1)
345   !
350   !
355   GOSUB Input                     ! Directs flow to subroutine INPUT
360   REDIM Bays(S),Factor(S)
365   FOR S=1 TO Storey-1
370     Cycle=1
375     N=Bays(S)
380     MAT Endmom= (0)
385     REDIM K_inv_inv(N+3,N+3),Differ(N+3,N+3),Compare(N+3,N+3),Compare1(N+3,N
+3)
390     REDIM K(N+3,N+3),Kt(N+3,N+3),Kinv(N+3,N+3)
395     REDIM Endmom(2,N),F(N+3),Dr(N+3),Basemom(2,N+1)
400     REDIM Hmombm(3,N),Hmomcol(2,N+1),Hmombase(N+1)
405     REDIM Servmombm(3,N),Servmomcol(2,N+1),Servmombase(N+1)
410     FOR Count=1 TO N+1
415       IF Cycle=1 THEN
420         Axial(2,Count)=Axial(1,Count)
425       END IF
430       IF Count=1 THEN
435         Axial(1,Count)=Axial(2,Count)+Load(S,Count)*Beamprop(1,S,Count)/2+Th
rust(S,Count)
440       ELSE
445         IF Count=N+1 THEN
450           Axial(1,Count)=Axial(2,Count)+Load(S,Count-1)*Beamprop(1,S,Count-1
)/2+Thrust(S,Count)
455         ELSE
460           Axial(1,Count)=Axial(2,Count)+Load(S,Count)*Beamprop(1,S,Count)/2+
Load(S,Count-1)*Beamprop(1,S,Count-1)/2+Thrust(S,Count)
465         END IF
470       END IF
475     NEXT Count
480     Lambda=1.0
485     Hinge=0
490     Hingeloc=0
495     Total=0
500     Counter=0             !Counter=0 causes calc of moments with Fh only.
505     IF S=Storey-1 THEN
510       H1=Colprop(1,S,1)
515     ELSE
520       H1=Colprop(1,S,1)/2
525     END IF
530     IF S=1 THEN
535       H2=.00001
540     ELSE
545       IF Cycle=1 THEN
550         H2=Colprop(1,S-1,1)/2
555       ELSE
560         H2=(Colprop(1,S-1,1)/2)*Slendratiol
565       END IF
570     END IF
575     GOSUB Matrix_k
580     FOR X=BASE(K,1) TO SIZE(K,1)+BASE(K,1)-1
585       FOR Y=BASE(K,2) TO SIZE(K,2)+BASE(K,2)-1
590         Total=Total+K(X,Y)^2
595       NEXT Y
600     NEXT X
605     Test=DET(K)/SQR(Total)
610     IF Test<.001 THEN
615       Lambdap=Lambda
620       IF Cycle=1 THEN Hinges(S)=Hinge
625       GOTO 770
630     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(S)=Hinge
695        GOTO 770
700      ELSE
705        Lambda1=Lambda
710        Lambda2=0
715      END IF
720      GOSUB Matrix_f
725      GOSUB Moments
730      IF Hingeloc>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 810;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
945      MAT Dcondition= (0)
946      MAT Ccondition= (1)
947      IF Base=0 THEN
948        FOR Count=1 TO N+1
949          Ccondition(1,Storey-1,Count)=0
950        NEXT Count
```

```
951       END IF
952       Hingeloc=0
953       MAT Axial= (0)
954       MAT Colmom= (0)
955       MAT Beamom= (0)
956       MAT Endmom= (0)
957       MAT Basemom= (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 968
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 bases fixed or pinned____________________(F/P)"
1080    INPUT Que$
```

```
1085  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 S
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,S,Dim),Colprop(5,S,Dim+1)
1190      REDIM Load(S,Dim+1),Thrust(S,Dim+1),Scondition(S,Dim),Ccondition(2,S,D
im+1)
1195      IF Base=0 THEN
1200        FOR Count=1 TO Dim+1
1205          Ccondition(1,S,Count)=0
1210        NEXT Count
1215      END IF
1220      REDIM Beamom(3,S+1,Dim),Colmom(2,S+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,6X,D.3D
1295    NEXT Beam
1300    DISP USING 1305
1305    IMAGE #,"Are beam dimensions and properties OK.......(Y/N)"
1310    INPUT Que$
1315    IF Que$="Y" THEN
1320      IF Edit=1 THEN GOTO 969
1325      GOTO 1480
1330    END IF
1335    IF Que$="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("-"),4X,6("-"),4X,8("-"),4X,8("-")
1370        FOR Beam=1 TO Bays(S)
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.6D,3X,D.6D,3X,D.6D,6X,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,5X,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)"
158"      INPUT Que$
155-      IF Que$="Y" THEN
1595        IF Edit=1 THEN GOTO 969
1600        GOTO 1770
1605      END IF
1610      IF Que$="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 Que$
1695        IF Que$="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," :Udl (kN/m)"
1725          INPUT Udl
1730          Load(Storey,No)=Udl
1735          PRINT USING 1740;No,Udl
1740          IMAGE 10X,4*,14X,3D.DD
1745        END IF
1750        IF Que$="N" THEN GOTO 1545
1755        GOTO 1575
1760      END IF
1765      GOTO 1575
1770      PRINT FNClear$
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 Que$
1825      IF Que$="Y" THEN GOTO 1840
1830      IF Que$="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 Que$
1930      IF Que$="Y" THEN
1935        IF Edit=1 THEN GOTO 969
1940        GOTO 1997
1945      END IF
1950      IF Que$="N" THEN
1955        IF Edit=1 THEN
1960          PRINT FNClear$
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("-"),5X,14("-",,4X,5("-"),5X,6("-"),4X,8("-"),4X,8("-")
1976          FOR Col=1 TO Bays(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,4*,13X,2D.3D,5X,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+((Ccondition(1,S,Count)=0)*12+(Ccondition(1,S,Count)=1)*3+(Ccondition(1,S,Count)=10)*3)*Colprop(2,S,Count)
2040     IF S=1 THEN
2045       B=0
2050     ELSE
2055       B=B+(Ccondition(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*.5
2075   K(1,3)=-(B*E/H2^3)
2080   K(3,3)=-.5*K(1,3)
2085   C=((Ccondition(1,S,1)=0)*6+(Ccondition(1,S,1)=1)*3)*Colprop(2,S,1)
2090   IF S=1 THEN
2095     D=0
2100   ELSE
2105     D=(Ccondition(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)=((((Ccondition(1,S,1)=0)*1/1.5+(Ccondition(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     G=((Ccondition(1,S,Count-2)=0)*6+(Ccondition(1,S,Count-2)=1)*3)*Colprop(2,S,Count-2)
2150     IF S=1 THEN
2155       M=0
2160     ELSE
2165       M=(Ccondition(2,S,Count-2)=1)*3*Colprop(2,S-1,Count-2)
2170     END IF
2175     K(1,Count)=-(G*E/H1^2)+(M*E/H2^2)
2180     K(3,Count)=-(M*E/H2^2)
2185     J=((Ccondition(1,S,Count-2)=0)*4+(Ccondition(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       Q=((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)=((J+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 Kt= TRN(K)
2255   MAT K= K+Kt
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         Axl=Axial(1,Count)
3075       ELSE
3080         Py=Colprop(4,(B+1)-Loc,Count)*Fpc
3085         Axl=Axial(Loc,Count)
3090       END IF
3095       IF Axl*Lambda/Py>=1 THEN
3100         Lambdap=Py/Axl
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     Udl=Load(B,Count)
3180     SELECT Bcondition(B,Count)
3185     CASE =0
3190       Ml=Udl*Lambda*(Beamprop(1,B,Count)^2)/12
3195       Mr=Ml
3200     CASE =10
3205       Ml=-Beamom(1,B,Count)
3210       Mr=Udl*Lambda*(Beamprop(1,B,Count)^2)/8-Ml/2
3215     CASE =1
3220       Mr=Beamom(3,B,Count)
3225       Ml=Udl*Lambda*(Beamprop(1,B,Count)^2)/8-Mr/2
3230     CASE =20
3235       Ml=-Beamom(1,B,Count)
3240       Mr=Udl*(Lambda-Lambda1)*(Beamprop(1,B,Count)^2)/4+Fem2hr
3245     CASE =2
3250       Ml=Udl*(Lambda-Lambda1)*(Beamprop(1,B,Count)^2)/4-Fem2hl
```

```
3255        Mr=Beamom(3,S,Count)
3260      CASE =11
3265        Ml=-Beamom(1,S,Count)
3270        Mr=Beamom(3,S,Count)
3275      END SELECT
3280      Endmom(1,Count)=Ml
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          Mpl=Colprop(3,1,Count)*Fpc
3320        ELSE
3325          Py=Colprop(4,(S+1)-Loc,Count)*Fpc
3330          Mpl=Colprop(3,(S+1)-Loc,Count)*Fpc
3335        END IF
3340        SELECT Ccondition(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(Mpl),ABS(1.
18*Mpl*(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(Mpl),ABS(1.
18*Mpl*(1-Axial(Loc,Count)*Lambda/Py)))
3400        CASE =11
3405          Colmom(Loc,S+X,Count)=SGN(Colmom(Loc,S+X,Count))*MIN(ABS(Mpl),ABS(1.
18*Mpl*(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(Mpl),ABS(1.
18*Mpl*(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)-((Ccondition(1,S,1)=10)*Colmom(1,S+X,1))-((Ccondition(1,S
,1)=11)*Colmom(1,S+X,1))-((Ccondition(1,S,1)=1)*Basemom(1+X,1)/2)
3465    F(2)=F(2)-((Ccondition(2,S,1)=11)*Colmom(2,S+X,1))
3470    FOR Count=4 TO N+3
3475      Fi=-((Ccondition(1,S,Count-2)=10)*Colmom(1,S+X,Count-2))-((Ccondition(1,
S,Count-2)=11)*Colmom(1,S+X,Count-2))
3480      Fi=Fi-((Ccondition(1,S,Count-2)=1)*Basemom(1+X,Count-2)/2)-((Ccondition(
2,S,Count-2)=11)*Colmom(2,S+X,Count-2))
3485      IF Count=N+3 THEN
3490        F(Count)=-Endmom(2,Count-3)+Fi
3495      ELSE
3500        F(Count)=Endmom(1,Count-2)-Endmom(2,Count-3)+Fi
3505      END IF
3510    NEXT Count
3515    Hor1=0
3520    Hor2=0
3525    FOR Count=1 TO N+1
3530      Hor=((Ccondition(2,S,Count)=11)*Colmom(2,S+X,Count))/H2
3535      Hor2=Hor2+Hor
3540      Hor=(((Ccondition(1,S,Count)=10)*Colmom(1,S+X,Count))+((Ccondition(1,S,C
ount)=1)*Basemom(1+X,Count)))*3/(2*H1)-Hor
```

```
3545     Hor=Hor+((Ccondition(1,S,Count)=1))*(Colmom(1,S+X,Count)+Basemom(1+X,Cou
nt))/H1)
3550     Hor1=Hor1+Hor
3555   NEXT Count
3560   F(1)=Hor1+Load(S,N+1)*Lambda
3565   Shor=0
3570   IF S=1 THEN
3575     Shor=0
3580   ELSE
3585     FOR Count=1 TO S-1
3590       Shor=Shor+Load(Count,N+1)*Lambda
3595     NEXT Count
3600   END IF
3605   IF Cycle=2 THEN Shor=Shor*Slendratio1
3610   F(3)=Hor2+Shor
3615   RETURN
3620   !------------------------------------------------------------------------
4000   !Subroutine MOMENTS calculates the bending moments at for each load factor
4005   !------------------------------------------------------------------------
4010 Moments:      !
4015   MAT Df= Kinv*F
4020   FOR Count=1 TO N
4025     Ks=Beamprop(2,S,Count)*E/Beamprop(1,S,Count)
4030     SELECT Bcondition(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       Beamom(1,S+X,Count)=M1
4080       Beamom(3,S+X,Count)=M3
4085       Beamom(2,S+X,Count)=-(-M1+M3)/2+(Load(S,Count)*Lambda/8)*(Beamprop(1,S
,Count)^2)
4090       IF Counter=0 THEN
4095         Hmombm(1,Count)=M1
4100         Hmombm(2,Count)=-(-M1+M3)/2
4105         Hmombm(3,Count)=M3
4110       END IF
4115       IF Lambda=1 THEN
4120         Servmombm(1,Count)=M1
4125         Servmombm(2,Count)=Beamom(2,S+X,Count)
4130         Servmombm(3,Count)=M3
4135       END IF
4140     CASE =10
4145       M3=(3*Df(Count+3))*Ks+Endmom(2,Count)
4150       Beamom(1,S+X,Count)=Beamom(1,S,Count)
4155       Beamom(3,S+X,Count)=M3
4160       Beamom(2,S+X,Count)=(Load(S,Count)*Lambda/8)*(Beamprop(1,S,Count)^2)-(
-Beamom(1,S,Count)+M3)/2
4165     CASE =20
4170       M3=Endmom(2,Count)
4175       Beamom(1,S+X,Count)=Beamom(1,S,Count)
4180       Beamom(3,S+X,Count)=M3
4185       Beamom(2,S+X,Count)=Beamom(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       Beamom(1,S+X,Count)=M1
4225       Beamom(3,S+X,Count)=Beamom(3,S,Count)
4230       Beamom(2,S+X,Count)=(Load(S,Count)*Lambda/8)*(Beamprop(1,S,Count)^2)-(
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-M1+Beamom(3,S,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,S+X,Count)=M1
4270       Beamom(3,S+X,Count)=Beamom(3,S,Count)
4275       Beamom(2,S+X,Count)=Beamom(2,S,Count)
4280     CASE =11
4285       M2=(Load(S,Count)*Lambda/8)*(Beamprop(1,S,Count)^2)
4290       Beamom(1,S+X,Count)=Beamom(1,S,Count)
4295       Beamom(3,S+X,Count)=Beamom(3,S,Count)
4300       Beamom(2,S+X,Count)=M2-((Beamom(3,S,Count)-Beamom(1,S,Count))/2)
4305     END SELECT
4310   NEXT Count
4315   FOR Count=1 TO N+1
4320     Ks=Colprop(2,S,Count)*E/H1
4325     SELECT Ccondition(1,S,Count)
4330     CASE =0
4335       IF Count=1 THEN
4340         Mlo=(4*Df(2)-(6*Df(1)/H1))*Ks
4345         Mbase=(2*Df(2)-(6*Df(1)/H1))*Ks
4350       ELSE
4355         Mlo=(4*Df(Count+2)-(6*Df(1)/H1))*Ks
4360         Mbase=(2*Df(Count+2)-(6*Df(1)/H1))*Ks
4365       END IF
4370       Colmom(1,S+X,Count)=Mlo
4375       Basemom(1+X,Count)=Mbase
4380       IF Counter=0 THEN
4385         Hmomcol(1,Count)=Mlo
4390         Hmombase(Count)=Mbase
4395       END IF
4400       IF Lambda=1 THEN
4405         Servmomcol(1,Count)=Mlo
4410         Servmombase(Count)=Mbase
4415       END IF
4420     CASE =1
4425       IF Count=1 THEN
4430         Mlo=(3*Df(2)-(3*Df(1)/H1))*Ks+Basemom(1,Count)/2
4435       ELSE
4440         Mlo=(3*Df(Count+2)-(3*Df(1)/H1))*Ks+Basemom(1,Count)/2
4445       END IF
4450       Colmom(1,S+X,Count)=Mlo
4455       IF Counter=0 THEN Hmomcol(1,Count)=Mlo
4460       IF Lambda=1 THEN Servmomcol(1,Count)=Mlo
4465     CASE =10
4470       Mbase=(-3*Df(1)/H1)*Ks+Colmom(1,S,Count)/2
4475       Basemom(1+X,Count)=Mbase
4480     CASE =11
4485     END SELECT
4490     IF S>1 THEN
4495       IF Ccondition(2,S,Count)=1 THEN
4500         IF Count=1 THEN
4505           Mup=(3*Df(2)-(3*(Df(3)-Df(1))/H2))*Colprop(2,S-1,1)*E/H2
4510         ELSE
4515           Mup=(3*Df(Count+2)-(3*(Df(3)-Df(1))/H2))*Colprop(2,S-1,Count)*E/H2
4520         END IF
4525         Colmom(2,S+X,Count)=Mup
4530         IF Counter=0 THEN Hmomcol(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           Mpl=SGN(Mom1)*Beamprop(3,S,Count)*Fpb
5060         ELSE
5065           Mpl=-1*SGN(Mom1)*Beamprop(3,S,Count)*Fpb
5070         END IF
5075         Lambdap=((Lambda2-Lambda1)/(Mom2-Mom1))*Mpl+(Lambda1*Mom2-Lambda2*Mo
m1)/(Mom2-Mom1)
5080         IF Lambdap<Lambda THEN
5085           Lambda=Lambdap
5090           Beamno=Count
5095           Hingeloc=Loc
5100           Bmplastic=Mpl
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         Axl=Axial(1,Count)
5155       ELSE
5160         Py=Colprop(4,(S+1)-Loc,Count)*Fpc
5165         Axl=Axial(Loc,Count)
5170       END IF
5175       Mom1=Colmom(Loc,S,Count)
5180       Mom2=Colmom(Loc,S+1,Count)
5185       IF Ccondition(Loc,S,Count)<>11 THEN
5190         IF Ccondition(Loc,S,Count)<>10 THEN
5195           IF SGN(Mom2-Mom1)=SGN(Mom1) THEN
5200             IF S=1 THEN
5205               Mpl=SGN(Mom1)*Colprop(3,1,Count)*Fpc
5210             ELSE
5215               Mpl=SGN(Mom1)*Colprop(3,(S+1)-Loc,Count)*Fpc
5220             END IF
5225           ELSE
5230             IF S=1 THEN
5235               Mpl=-1*SGN(Mom1)*Colprop(3,1,Count)*Fpc
5240             ELSE
5245               Mpl=-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=((1.18*(Lambda2-Lambda1)*Mpl*Py)+(Lambda1*Mom2-Lambda2*Mom
1)*Py)/((Mom2-Mom1)*Py+1.18*(Lambda2-Lambda1)*Mpl*Axial(Loc,Count))
5280           IF Lambdap<Lambda THEN
5285             Lambda=Lambdap
5290             Colno=Count
```

```
5295              Hingeloc=Loc
5300              Colplastic=SGN(Mpl)*MIN(ABS(Mpl),ABS(1.18*Mpl*(1-Axial(1,Count)*
Lambdap/Py)))
5305              IF Cycle=1 THEN
5310                Location(1,S,Hinge+1)=0
5315                Location(2,S,Hinge+1)=Colno+Hingeloc/10
5320              END IF
5325            END IF
5330          END IF
5335        END IF
5340        IF Axl*Lambda/Py>=1 THEN
5345          Lambda=Py/Axl
5350          IF Lambdap<Lambda THEN
5355            Lambda=Lambdap
5360            Colno=Count
5365            Hingeloc=5+Loc
5370            IF Cycle=1 THEN
5375              Location(1,S,Hinge+1)=0
5380              Location(2,S,Hinge+1)=Colno+Hingeloc/10
5385            END IF
5390            GOTO 615
5395          END IF
5400        END IF
5405      NEXT Loc
5410      IF S=Storey-1 THEN
5415        IF Base=0 THEN
5420          Mom1=Basemom(1,Count)
5425          Mom2=Basemom(2,Count)
5430          IF Ccondition(1,S,Count)<>11 THEN
5435            IF Ccondition(1,S,Count)<>1 THEN
5440              IF SGN(Mom2-Mom1)=SGN(Mom1) THEN
5445                Mpl=SGN(Mom1)*Colprop(3,S,Count)*Fpc
5450              ELSE
5455                Mpl=-1*SGN(Mom1)*Colprop(3,S,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 Lambdap<Lambda THEN
5475                Lambda=Lambdap
5480                Colno=Count
5485                Hingeloc=3
5490                Colplastic=Mpl
5495                IF Cycle=1 THEN
5500                  Location(1,S,Hinge+1)=0
5505                  Location(2,S,Hinge+1)=Colno+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 Colno=0 THEN
5550      IF Bcondition(S,Beamno)=2 THEN
5555        IF Cycle=1 THEN Hinges(S)=Hinge+1
5560        GOTO 770
5565      END IF
5570      IF Bcondition(S,Beamno)=20 THEN
5575        IF Cycle=1 THEN Hinges(S)=Hinge+1
5580        GOTO 770
5585      END IF
5590      IF Bcondition(S,Beamno)=11 THEN
5595        IF Cycle=1 THEN Hinges(S)=Hinge+1
5600        GOTO 770
5605      END IF
5610      IF Bcondition(S,Beamno)=0 THEN
```

```
5615        IF Hingeloc=1 THEN
5620          Bcondition(S,Beamno)=10
5625        ELSE
5630          Bcondition(S,Beamno)=1
5635        END IF
5640      ELSE
5645        IF Bcondition(S,Beamno)=10 THEN
5650          IF Hingeloc=2 THEN
5655            Bcondition(S,Beamno)=20
5660            Udl=Load(S,Beamno)
5665            Fem2hr=(Beamom(3,S+1,Beamno)*(Udl*Lambda-Udl*Lambda1)-Beamom(3,S,B
eamno)*(Udl*Lambda-Udl*Lambda2))/(Udl*Lambda2-Udl*Lambda1)
5670          ELSE
5675            Bcondition(S,Beamno)=11
5680          END IF
5685        ELSE
5690          IF Hingeloc=2 THEN
5695            Bcondition(S,Beamno)=2
5700            Udl=Load(S,Beamno)
5705            Fem2hl=(Beamom(1,S+1,Beamno)*(Udl*Lambda-Udl*Lambda1)-Beamom(1,S,B
eamno)*(Udl*Lambda-Udl*Lambda2))/(Udl*Lamuda2-Udl*Lambda1)
5710          ELSE
5715            Bcondition(S,Beamno)=11
5720          END IF
5725        END IF
5730      END IF
5735      Beamom(Hingeloc,S,Beamno)=Bmplastic
5740      Beamom(Hingeloc,S+1,Beamno)=Bmplastic
5745    ELSE
5750      IF Hingeloc=1 THEN
5755        IF Ccondition(1,S,Colno)=0 THEN
5760          Ccondition(1,S,Colno)=10
5765        ELSE
5770          Ccondition(1,S,Colno)=11
5775        END IF
5780        Colmom(1,S,Colno)=Colplastic
5785        Colmom(1,S+1,Colno)=Colplastic
5790      END IF
5795      IF Hingeloc=2 THEN
5800        Ccondition(2,S,Colno)=11
5805        Colmom(2,S,Colno)=Colplastic
5810        Colmom(2.S+1,Colno)=Colplastic
5815      END IF
5820      IF Hingeloc=3 THEN
5825        IF Ccondition(1,S,Colno)=0 THEN
5830          Ccondition(1,S,Colno)=1
5835        ELSE
5840          Ccondition(1,S,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 Result:    !
6015    PRINT
6020    IF S=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,5("-"),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 S
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,/,16("-")
6390     PRINT
6395     PRINT USING 6400
6400     IMAGE "Elastic/Plastic Collapse Load Factors",/,37("-")
6405     PRINT USING 6410
6410     IMAGE "Storey No.",15X,"Plastic",8X,"Elastic",8X,"Elastic/Plastic",/,10(
"-"),15X,7("-"),8X,7("-"),8X,15("-")
6415     S=1
6420   END IF
6425   PRINT USING 6430;S,Lambdap1,Lambdao1,Fail
6430   IMAGE 4X,DD,20X,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     Us=(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*Us*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+Us*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   Lambdao=Pstorey/Axtotal
7155   RETURN
7160     !------------------------------------------------------------------
8000     !Subroutine INTERACTION calculates the Elastic/Plastic failure load.
8005     !------------------------------------------------------------------
8010 Interaction:       !
8015   Slendratio=1000
8020   Mag=1/(1-((.4*Pstorey)/Pstorey))
8025   MAT Hmombm= Hmombm*(Lambdao*.4*Mag)
8030   MAT Hmomcol= Hmomcol*(Lambdao*.4*Mag)
8035   MAT Hmombase= Hmombase*(Lambdao*.4*Mag)
8040   Lambdau4=0
```

```
8045  FOR Count=1 TO N
8050     Sigmom=MAX(ABS(Hmombm(1,Count)),ABS(Hmombm(2,Count)),ABS(Hmombm(3,Count)
))
8055     Lambda=(Beamprop(1,S,Count)/Fpb)*(Sigmom/Beamprop(2,S,Count))*(Beamprop(
5,S,Count)/SQR(Beamprop(2,S,Count)/Beamprop(4,S,Count)))
8060     IF Lambda>Lambda04 THEN Lambda04=Lambda
8065  NEXT Count
8070  FOR Count=1 TO N+1
8075     Subt=SQR(1+4*Fpc*Axial(1,Count)*.4*Lambdao*Colprop(2,S,Count)/(Hmomcol(1
,Count)^2))*.5+.5
8080     Lambda=(ABS(Hmomcol(1,Count))/Colprop(2,S,Count))*(Colprop(1,S,Count)/Fp
c)*Subt
8085     IF Lambda>Lambda04 THEN Lambda04=Lambda
8090     IF S=Storey-1 THEN
8095       IF Base=0 THEN
8100         Subt=SQR(1+4*Fpc*Axial(1,Count)*.4*Lambdao*Colprop(2,S,Count)/(Hmomb
ase(Count)^2))*.5+.5
8105         Lambda=(ABS(Hmombase(Count))/Colprop(2,S,Count))*(Colprop(1,S,Count)
/Fpc)*Subt
8110         IF Lambda>Lambda04 THEN Lambda04=Lambda
8115       END IF
8120     END IF
8125  NEXT Count
8130  Alpha=.4/(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
ag))*Alpha*(1/(1-Alpha)-1))
8150       Ratio=1/((Beamprop(5,S,Count)/Fpb)*(Mtotal/Beamprop(2,S,Count)))
8155       IF Ratio<Slendratio THEN Slendratio=Ratio
8160     NEXT Loc
8165  NEXT Count
8170  FOR Count=1 TO N+1
8175     FOR Loc=1 TO 2
8180       IF S=1 THEN
8185         Mtotal=ABS(Alpha*Lambdao*Servmomcol(1,Count)+(Hmomcol(1,Count)/(.4*M
ag))*Alpha*(1/(1-Alpha)-1))
8190       ELSE
8195         Mtotal=ABS(Alpha*Lambdao*Servmomcol(Loc,Count)+(Hmomcol(Loc,Count)/(
.4*Mag))*Alpha*(1/(1-Alpha)-1))
8200       END IF
8205       Subt=SQR(1+4*Fp*Lambdao*Axial(1,Count)*Colprop(2,S,Count)/((Colprop(5,
S,Count)/SQR(Colprop(2,S,Count)/Colprop(4,S,Count)))*Mtotal)^2)
8210       Ratio=1/((Colprop(5,S,Count)/Fpc)*(Mtotal/Colprop(2,S,Count))*Subt)
8215       IF Ratio<Slendratio THEN Slendratio=Ratio
8220     NEXT Loc
8225     IF S=Storey-1 THEN
8230       IF Base=0 THEN
8235         Mtotal=ABS(Alpha*Lambdao*Servmombase(Count)+(Hmombase(Count)/(.4*Mag
))*Alpha*(1/(1-Alpha)-1))
8240         Subt=SQR(1+4*Fp*Lambdao*Axial(1,Count)*Colprop(2,S,Count)/((Colprop(
5,S,Count)/SQR(Colprop(2,S,Count)/Colprop(4,S,Count)))*Mtotal)^2)
8245         Ratio=1/((Colprop(5,S,Count)/Fpc)*(Mtotal/Colprop(2,S,Count))*Subt)
8250         IF Ratio<Slendratio THEN Slendratio=Ratio
8255       END IF
8260     END IF
8265  NEXT Count
8270  IF Cycle=1 THEN
8275     Alpha1=Alpha
8280     Lambdao1=Lambdao
8285     Lambdap1=Lambdap
8290     Angle=Alpha1*Lambdao/Lambdap
8295  END IF
8300  IF Cycle=1 THEN
8305     Slendratio1=Slendratio
8310     Y.axis=Angle*Slendratio1
```

```
8315  ELSE
8320    Y_axis=(Alpha1*Lambdao/Lambdap)
8325  END IF
8330  IF Angle>1.5 THEN
8335    Io=.76*(.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,S,Count)=Beamprop(1,S,Count)/Slendratio1
8410      Colprop(1,S,Count)=Colprop(1,S,Count)/Slendratio1
8415    NEXT Count
8420    FOR Count=1 TO N+1
8425      IF Count=N+1 THEN
8430        Load(S,Count)=Load(S,Count)*Slendratio1
8435      ELSE
8440        Load(S,Count)=Load(S,Count)*(Slendratio1^2)
8445      END IF
8450      Thrust(S,Count)=Thrust(S,Count)*Slendratio1
8455      Axial(2,Count)=Axial(2,Count)*Slendratio1
8460      Holdax1(Count)=Axial(1,Count)
8465    NEXT Count
8470    Cycle=2
8475    FOR Count=1 TO N
8480      Bcondition(S,Count)=0
8485    NEXT Count
8490    FOR Count=1 TO N+1
8495      FOR Loc=1 TO 2
8500        IF S=Storey-1 THEN
8505          IF Base=0 THEN
8510            Ccondition(1,S,Count)=0
8515            Ccondition(2,S,Count)=1
8520          ELSE
8525            Ccondition(Loc,S,Count)=1
8530          END IF
8535        ELSE
8540          Ccondition(Loc,S,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,S,Count)=Beamprop(1,S,Count)*Slendratio1
8590      Colprop(1,S,Count)=Colprop(1,S,Count)*Slendratio1
8595    NEXT Count
8600    FOR Count=1 TO N+1
8605      IF Count=N+1 THEN
8610        Load(S,Count)=Load(S,Count)/Slendratio1
8615      ELSE
8620        Load(S,Count)=Load(S,Count)/(Slendratio1^2)
8625      END IF
8630      Thrust(S,Count)=Thrust(S,Count)/Slendratio1
```

```
8635      Axial(2,Count)=Axial(2,Count)/Slendratiol
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 Graphic:      !
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 65,96
9270    LORG 6
```

```
9275     LABEL "STOREY NO.";S;":Collapse mechanism"
9280     MOVE Xstart,Ystart
9285     DRAW Xstart+125,Ystart
9290     LINE TYPE 4
9295     DRAW Xstart+128,Ystart
9300     MOVE  ..rt-5,Ystart-50
9305     DRAW ...t,Ystart-50
9310     LINE TYPE 1
9315     DRAW (Scale*Xsum)-120,Ystart-50
9320     VIEWPORT 0,132,0,100
9325     FRAME
9330   END IF
9335   IF S>1 THEN
9340     MOVE Xsta     tart+(Scale*Colprop(1,S-1,1)/2)
9345     LORG 4
9350     LABEL "o"
9355     IMOVE 0,6
9360   ELSE
9365     MOVE Xstart,Ystart
9370   END IF
9375   IF S=Storey-1 THEN
9380     LINE TYPE 10
9385     DRAW Xstart,Ystart-(Scale*Colprop(1,S,1))
9390     LINE TYPE 1
9395   ELSE
9400     DRAW Xstart,Ystart-(Scale*Colprop(1,S,1)/2)
9405     IMOVE 0,2
9410     LORG 6
9415     LABEL "o"
9420   END IF
9425   Xi=0
9430   FOR Beam=1 TO Bays(S)
9435     CSIZE 5
9440     Xi=(Scale*Beamprop(1,3,Beam))+Xi
9445     Xincr=Xstart+Xi
9450     IF Beam<=10 THEN
9455       IF S>1 THEN
9460         MOVE Xincr,Ystart+(Scale*Colprop(1,S-1,1)/2)
9465         LORG 4
9470         LABEL "o"
9475         IMOVE 0,6
9480       ELSE
9485         MOVE Xincr,Ystart
9490       END IF
9495       IF S=Storey-1 THEN
9500         LINE TYPE 10
9505         DRAW Xincr,Ystart-(Scale*Colprop(1,S,1))
9510         LINE TYPE 1
9515         MOVE Xincr-(Scale*Beamprop(1,S,Beam)/2),Ystart-(Scale*Colprop(1,S,1)
)-4
9520       ELSE
9525         DRAW Xincr,Ystart-(Scale*Colprop(1,S,1)/2)
9530         IMOVE 0,2
9535         LORG 6
9540         LABEL "o"
9545         MOVE Xincr-(Scale*Beamprop(1,S,Beam)/2),Ystart-(Scale*Colprop(1,S,1)
/2)-4
9550       END IF
9555       IF Bays(S)<=8 THEN
9560         CSIZE 4
9565       ELSE
9570         CSIZE 3
9575       END IF
9580       LABEL "BAY";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 "o"
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 "o"
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 "o"
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 "o"
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(S) =8 THEN
9795          CSIZE 4
9800        ELSE
9805          CSIZE 3
9810        END IF
9815        LABEL "BAY";Beam
9820      END IF
9825    NEXT Beam
9830    FOR No=1 TO Hinoos(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
7880          Xincr=Xi+Xincr
9695        NEXT Beam
9890      END IF
9895      IF INT(Number)<=10 THEN
9900        IF Bays(S)<=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(1,8,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*Beamprop(1,8,INT(Number
))/2),0
9955      IF INT(FRACT(Number)*10.5)=3 THEN IMOVE (Scale*Beamprop(1,8,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(1,8,INT(Num
ber)))
9980      IF INT(FRACT(Number)*10.5)=6 THEN IMOVE 0,2-(Scale*Colprop(1,8,INT(Num
ber))/2)
9985      IF INT(FRACT(Number)*10.5)=7 THEN IMOVE 0,(Scale*Colprop(1,8,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(1,8,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$="Y" THEN
10160   IF Bays(8)<=8 THEN
10165     DUMP GRAPHICS #701
10170   END IF
10175   IF Bays(8)>8 THEN
10180     DUMP DEVICE IS 701,EXPANDED
10185     DUMP GRAPHICS
10190   END IF
10195 END IF
10200 GCLEAR
10205 RETURN
10210 !--------------------------------------------------------------------------
10215 END
11000 DEF FNClear$
11005   OUTPUT 2;K";
11010   RETURN Dummy$
11015 FNEND
```

Imbed Trace

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