Stiffness and strength design of multistory frames

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Stiffness and Strength Design of Multistory Frames

Critères de rigidité et de résistance pour les cadres étagés

Steifigkeits- und Festigkeitskriterien beim Entwurf von Stockwerkrahmen

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Special Chapter

Subassemblage methods of analyzing unbraced multistory steel frames, although approximate in nature, have been proved to be simple and economical alternatives to conventional second-order elastic-plastic methods. One of the limitations of existing subassumblage techniques, however, is their inability to readily analyse the lowest story in a frame where the column bases are fixed. They also do not consider proportional loading cases.

The proposed treatment, which is based upon the subassemblage stiffness concept, develops simple algebraic expressions for representing the stiffnesses of both the intermediate and the lowest stories. By considering strength as a function of deteriorating stiffnesses, the entire elastic-plastic load-deformation characteristics of a story under proportional and nonproportional loadings can be generated. It is thus possible to determine the stiffness and strength of a story by the one approach.

As a design tool, the present technique eliminates the need for assuming sways and collapse mechanisms at failure, a feature generally present in existing subassemblage methods of design. Instead, it predicts the sway at collapse and chooses the mode of failure of a story. It, therefore, does not involve any iterations before the final design is reached.

The one important feature of the design method is that, from a preliminary design, it modifies the beams to satisfy strength. After selecting the beam sizes, the moments and axial loads on the columns are known. Consequently, these can be selected so as to force the story stiffness to become either negative or zero when hinges appear in them. In this way, it is possible to ensure that all stories fail at approximately the same pre-selected load factor, resulting in a frame which is not over-designed. Further, the stories have the same general load-deformation characteristics.

Following the proportioning for strength, the frame may be modified to satisfy stiffness constraints, again using the same basic stiffness equations. A typical design example is given to illustrate the principles involved. It is noted that, in agreement with previously observed trend, frames which are designed to meet stiffness constraints fail at load factors higher than those proportioned for strength only.

Introduction

Recognition of the fact that unbraced multistory steel frames are complex and highly redundant structures has led to the development of relatively simple plastic methods of analysis and design based upon the subassemblage concept. In these methods, a story unit is the basic substructure which is analyzed or designed without giving regard to the stress states of the other stories in the frame. Consequently, they are only approximate in nature.

Subassemblage techniques can be arbitrarily classified into 2 categories namely one which consists essentially of analytical methods and one which deals mainly with design. Obviously, analytical methods can be adapted to design. Those in the first group range from a pure manual method [5] to sophisticated computer systems [13]. The manual method [5] has since been simplified [3, 5], automated [7] and associated with optimization routines [14, 16]. The methods proposed by WRIGHT [15] and by POWELL and HAFEZ [3] also belong to the first class.

The second category of subassemblage techniques include those by HAFEZ and POWELL [8] and by EMKIN and LITLE [9, 10]. Failure mechanisms and sways are generally assumed in the design process, and several iterations may be necessary before calculations converge to the sway consistent with the idealized mechanism.

Purpose

This paper presents an alternative approximate method for analyzing and designing unbraced multistory steel frames. Its versatility is brought about by the use of algebraic expressions for describing elastic and inelastic behavior. As an analytical technique, it removes some of the shortcomings existing in current subassemblage methods such as their inability to investigate readily the strength of the lowest story in a frame.

In contrast to existing plastic design methods [5, 7, 8, 9, 10], the proposed treatment does not follow the usual procedure of assuming sway values and failure mechanisms. This obstacle is overcome by considering story stiffness as the prime variable. Once the stiffnesses have been evaluated, other dependent variables such as sways and moments can be readily found.

Apart from omitting the need for estimating sways and for idealizing failure mechanisms, it eliminates the use of auxiliary techniques to satisfy both stiffness and strength requirements, since strength is considered as a function of deteriorating stiffness. Besides, the story to be designed can be controlled to fail at pre-selected load-factors. Consequently, a balanced frame with no over-designed or under-designed stories results.

A two-stage development is presented. After introducing the story stiffness concept, the power of the method as an analytical tool is demonstrated. Comparison with a conventional, second-order, elastic-plastic method is carried out to indicate the degree of accuracy to be expected. Principles developed in the analysis part are subsequently employed in the design phase.

Assumptions

The following major assumptions are made herein. Only the in-plane behavior of rigidly-jointed frames with regular geometry is considered. All members are perfectly elastic-plastic and are initially straight. Bending is about their major axes. No story eccentricities exist. Panel zone deformations are neglected. Although applied loads can increase non-proportionally, only the proportional case is considered. Note that such a loading case has not been considered in any sub-assemblage techniques developed so far.

Basic Concepts

A typical load-deformation relationship for a story under proportional loads is shown in Figure 1 in which λ = load factor by which all loads are multiplied and Δ = sway. Also shown is the load-stiffness curve; the story stiffness $S_T = \frac{d\lambda}{d\Delta}$. Instead of studying story behavior from the $\lambda - \Delta$ relationship, it is convenient to study the $\lambda - S_T$ characteristic. The critical load-factor λ_C corresponds to the highest point on the $\lambda - S_T$ curve. After failure, a lower load factor, such as λ_B, is necessary for maintaining equilibrium. Note that at a load factor greater than λ_C, equilibrium cannot be satisfied and the computed story stiffness will be negative.

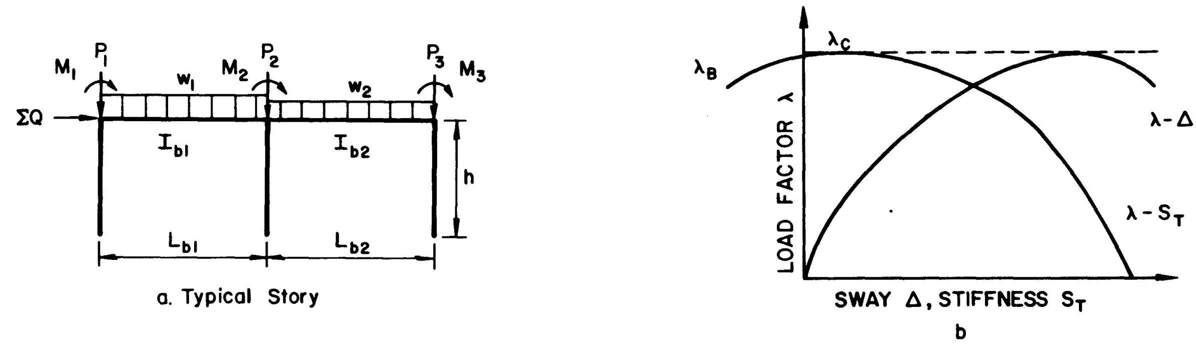

Fig. 1. Story Load-stiffness Characteristic.

The moments which are induced in a story can be approximated by two componnts, one caused by pure gravity loads and one by sway, $P - \Delta$ effect, Figure 2 (a). Gravity moments at the beams' ends are conservatively taken as the fixed-end moments, FEM = $wL^2/12$, which are shared equally by the columns at the top and at the bottom. Moments generated by sway are calculated once the story stiffness has been evaluated.

The story stiffness S_T is the sum of the stiffnesses, s, of the subassemblages whose number equals that of the columns in the story. Figure 2 (b) shows some typical subassemblages in an intermediate story. Each consists of one column and of one or two restraining beams. Points of inflection are assumed to develop at the centres of the members under sway conditions so that each subassemblage can be represented by the restrained column in Figure 2(c) – case (a). Cases (b) and (c) which are also depicted in Figure 2(c) refer to the columns of the lowest story in which points of inflection do not occur at mid-heights.

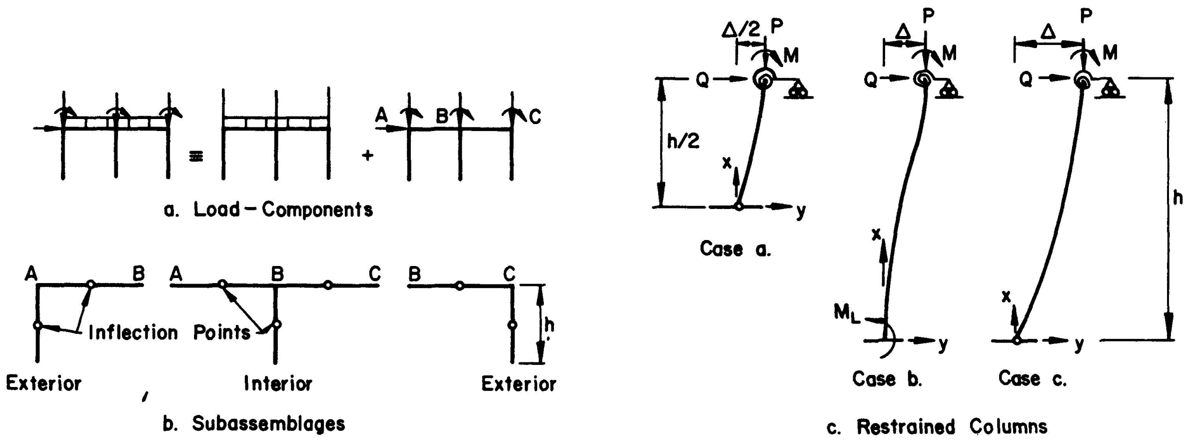

Fig. 2. Idealized Story Behavior.

Story Stiffness

The load-deformation relationships of these subassemblages can be derived algebraically as described in detail in Refs. 3 and 4. Briefly, these expressions are obtained in the following manner. Simple integration shows that the moment M_x in the columns is

$$M_x = A \sin(Kx) + B \cos(Kx) \tag{1}$$

in which A and B = constants of integration, $K^2 = \frac{P}{EI_c}$, P = column axial load, E = Young's Modulus, and I_c = column inertia. Denoting the rotational stiffnesses of the restraining beams in a subassemblage by $C = \Sigma\ (6EI_b/L_b)$ and satisfying compatibility of slope at the column's upper end, $\theta = \frac{dy}{dx}$, the stiffenesses of the restrained columns are obtained:

$$\text{Cases (a) and (c)}: s = \frac{Q}{\Delta} = \frac{P}{h}\frac{G}{1-G} \tag{2}$$

$$\text{in which } G = \frac{Kh}{2}\cot\frac{(Kh)}{2} - \frac{UPh}{2C} \tag{3}$$

$$\text{Case (b)}: s = \frac{Q}{\Delta} = P\left[\frac{\sin(Kh)}{K} - h + A_o - A_o\cos(Kh)\right]^{-1} \tag{4}$$

$$A_o = \left[1 - \cos(Kh) + \frac{UP}{KC}\sin(Kh)\right]\left[K\sin(Kh) + \frac{UP}{C}\cos(Kh)\right]^{-1} \quad (5)$$

One other quantity needed for case (b) is the moment M_L at the base,

$$M_L = A_o Q \quad (6)$$

In the above equations, the quantity U is the ratio of moments generated in the restraining beams to the column upper end moment. Based upon some simplifying assumptions, the following results have been obtained [3, 4]. For case (a), U = 2; for case (b), U is given by Eq. 7; and for case (c) U = 3/2,

$$U = \frac{4 - 4\cos(Kh)}{2 - 2\cos(Kh) - P(KC)^{-1}\sin(Kh)} > 2 \quad (7)$$

By using the first two terms in the expansion series of the trigonometric functions, the stiffness equations can be considerably simplified with only a minor loss of accuracy [3, 4].

$$\text{Case (a) – simplified: } s = \frac{Q}{\Delta} = \frac{12EI_c}{h^3(1+2\Psi)} - \frac{P}{h} \quad (8)$$

$$\text{Case (b) – simplified: } s = \frac{Q}{\Delta} = \frac{12EI_c}{h^3}\,\frac{3+\Psi}{3+4\Psi} - \frac{P}{h} \quad (9)$$

$$\text{Case (c) – simplified: } s = \frac{Q}{\Delta} = \frac{12EI_c}{h^3(4+1.5\Psi)} - \frac{P}{h} \quad (10)$$

The relationship between the load factor λ and the sway Δ is obtained by multiplying all Q's and P's by λ. For example, for case (a),

$$\lambda Q = \left[\frac{12EI_c}{h^3(1+2\Psi)} - \frac{\lambda P}{h}\right]\Delta \quad (11)$$

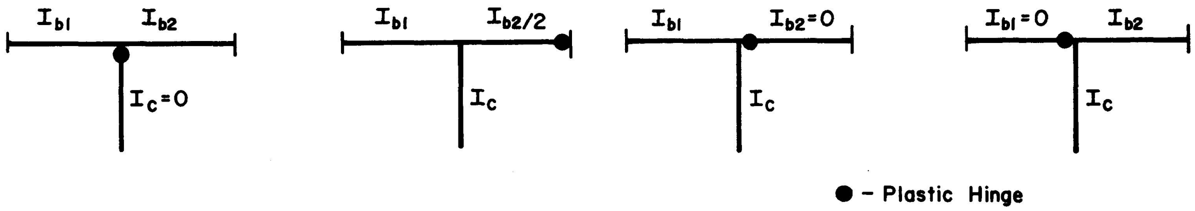

Fig. 3. Reduction in Member Stiffness.

It is convenient in this study to consider the story stiffness S_T as $\lambda\Sigma Q/\Delta$ where ΣQ = total wind loads on the story.

The quantity Ψ in Eqs. 8-11 is a measure of the ratio of column stiffness to restraining beam stiffnesses,

$$\Psi = \frac{I_c}{h\left[\frac{I_{b1}}{L_{b1}} + \frac{I_{b2}}{L_{b2}}\right]} \quad (12)$$

in which I_{b1} and I_{b2} are the beam inertias, and L_{b1} and L_{b2} are their respective lengths. In an exterior subassemblage, either I_{b1} or $I_{b2} = 0$. At the formation of plastic hinges, column inertias and beam inertias will change. Figure 3 illustrates the rules employed to evaluate these quantities for a given subassemblage.

Elastic Analysis

Deflections

The analytical aspect of the present treatment will be demonstrated on Frame A shown in Figure 4 and described in Table 1. For elastic deflections taking into account $P - \Delta$ effects, two sets of calculations are performed. First, the relative

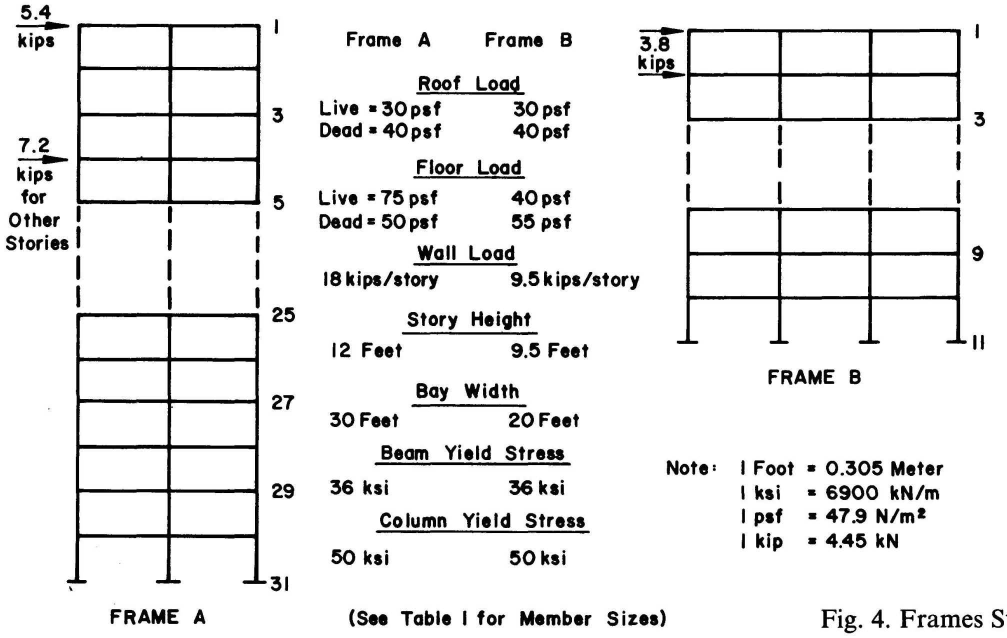

Fig. 4. Frames Studied.

Table 1: Members for frames in figure 4.

FRAME A

Level	Beam	Level	Columns Ext:	Columns Int:
1	W21 x 44	1 - 3	W10 x 54	W10 x 33
2 - 4	W21 x 55	3 - 5	W14 x 68	W14 x 68
5	W24 x 61	5 - 7	W14 x 95	W14 x 95
6 - 7	W24 x 68	7 - 9	W14 x 119	W14 x 127
8 - 10	W24 x 76	9 - 11	W14 x 142	W14 x 158
11 - 12	W24 x 84	11 - 13	W14 x 167	W14 x 193
13 - 14	W27 x 84	13 - 15	W14 x 193	W14 x 219
15 - 17	W27 x 94	15 - 17	W14 x 219	W14 x 246
18 - 21	W30 x 99	17 - 19	W14 x 246	W14 x 287
22 - 24	W30 x 108	19 - 21	W14 x 287	W14 x 314
25 - 30	W30 x 116	21 - 23	W14 x 314	W14 x 342
		23 - 25	W14 x 342	W14 x 398
		25 - 27	W14 x 370	W14 x 426
		27 - 29	W14 x 398	W14 x 455
		29 - 31	W14 x 500	W14 x 550

FRAME B

Level	Beam	Level	Columns Ext:	Columns Int:
1 - 2	W12 x 22	1 - 3	W8 x 13	W8 x 13
3 - 4	W14 x 22	3 - 5	W8 x 20	W8 x 20
5 - 8	W14 x 26	5 - 7	W8 x 28	W8 x 28
9 - 10	W16 x 26	7 - 9	W8 x 35	W8 x 35
		9 - 11	W8 x 40	W8 x 40

story sways are calculated using Eq. 8 and Eq. 9. These deflections are then used to calculate the moments and shears at the beams' ends. Additional sways [1] caused by axial deformations are subsequently evaluated.

Some results are plotted in Figure 5 together with those obtained from a conventional second-order, elastic-plastic method [12]. Good agreement is observed between the two methods.

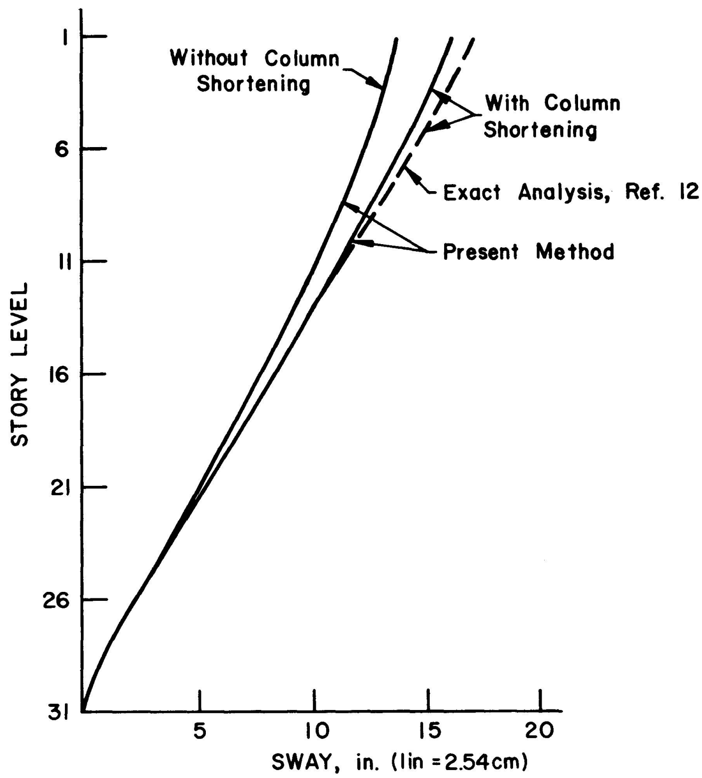

Fig. 5. Load-deformation Relationship of Frame a at $\lambda = 1.0$.

Increase In Member Sizes

The story stiffness concept was used in Refs. 3 and 4 to provide simple guidelines for improving frame stiffnesses. It was explained why increases in beam sizes has a more significant effect than column sizes in reducing lateral sway in conventional multistory frames in which $\Psi > 0.5$. However, column axial shortening was not considered. To fill the gap, some further examples are given.

The relative sway index, including the effects of column deformations, of the story below Level 26 of Frame A (Fig. 4 and Table 1) is 0.0041 at $\lambda = 1.0$. When all the columns are changed to W14X500, the sway index becomes 0.0039. Thus, a 50% increase in column inertias causes only a 5% decrease in sway. On the other hand, when the beams are changed to W33X130, the deflection index improves to 0.0029, i.e., an increase in beam inertias of about 35% causes a sway decrease of about 30%. Note that when Ψ is not much larger than 0.5, increasing both column and beam sizes may be more desirable [3, 4].

In the upper stories where column shortening has a more pronounced influence, similar results are obtained. For example, when the columns in the story below level 4 of Frame A are changed to W14X68, I_C increases by 35% and the sway decreases by 10%. Note that an increase in column area will ensure a reduction in chord drift, but this is only a minor component in the sway of unbraced frames.

Bending Moments

Other quantities which are of interest to the designer are the bending moments which occur at working loads. In an intermediate story, the moment M_C which acts at the top of a column is the sum of moments caused by the sway and beam gravity loads,

$$M_c = \frac{P\Delta}{2} + \frac{Qh}{2} \pm \frac{(FEM)_c}{2} \tag{14}$$

Since $Q = s\Delta$, Eq. 14 can be simplified to

$$M_c = \frac{6EI_c\Delta}{h^2(1+2\Psi)} \pm \frac{(FEM)_c}{2} \tag{15}$$

in which $(FEM)_c$ = resultant beam fixed-end moments on the joint. Similarly, the moments M_b which occur at the beam's ends are

$$M_b = R\,\frac{12EI_c\Delta}{h^2(1+2\Psi)} \pm FEM \tag{16}$$

in which R = ratio of stiffness I_{b2}/L_{b2} of the leeward beam to the total beam stiffnesses $\frac{I_{b1}}{L_{b1}} + \frac{I_{b2}}{L_{b2}}$. In an exterior subassemblage, $R = 1$. When calculating the above quantities, Δ to be used is the relative sway excluding column shortening influence. Moreover, in an interior subassemblage, beam moments are calculated for the leeward beam only.

Table 2 shows typical results obtained for Frame A. Comparison of bending moments with the second-order, elastic-plastic method [12] shows good agreement. The discrepancy in the upper levels arises partly because gravity beam moments are over-estimated and partly because the columns do not bend in exact double curvature.

Table 2: Comparison between present and exact methods at working loads.

	Beam Leeward-end Moments kip-in				Column Maximum Moments kip-in					
	A - B		B - C		A		B		C	
Level	Present	Ref.12	Present	Ref.12	Present	Ref.12	Present	Ref.12	Present	Ref.12
4	3740	3341	3657	3313	726	492	-923	-940	-1870	-1700
15	6992	6393	7034	7056	-900	-731	-3936	-4387	-3496	-3612
26	10198	9610	10546	10051	-2503	-2542	-6915	-7337	-5099	-5115

Note: 1 kip-in = 113N-M

Axial Loads

Provided the bending moments at the beams' ends are known with sufficient accuracy, column axial forces can be predicted with a reasonable degree of precision. However, since bending moments change as the load factor increases, prediction of column forces appears complicated. To simplify design, idealizations are necessary.

Some column forces for Frame A are shown in Figure 6. They vary almost linearly with λ even in the inelastic range. Note that at $\lambda = 1.35$ there are 24 hinges present and that at $\lambda = 1.40$, 48 hinges have developed. Similar studies made on the frames of Ref. 11 show that the deviation from linearity is small especially when plastic hinges appear less in the columns. At collapse, however, there is a sudden unloading in the windward columns and a corresponding loading in the leeward columns. For design purposes, the nonlinearity may be neglected.

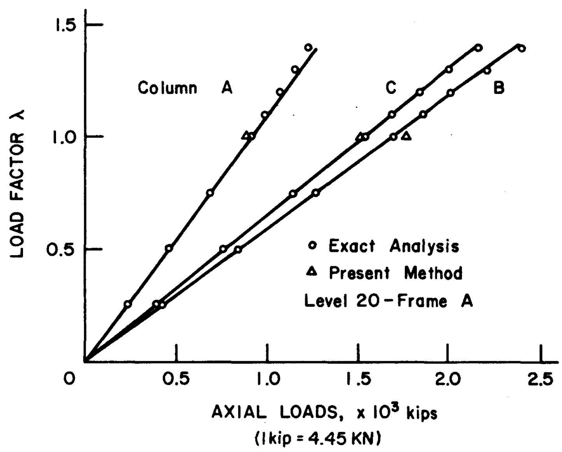

Fig. 6. Prediction of Column Loads.

With this idealization, the following relationship may be postulated,

$$P_\lambda = \lambda P_W \tag{17}$$

in which $P_\lambda =$ column load at a load factor λ, and $P_W =$ column load at $\lambda = 1.0$. Shown in Figure 6 are some values of P_W obtained using the present technique and the $P - \lambda$ distribution of Frame A given by the "exact" analysis [12]. The good agreement reached suggests that it is sufficiently accurate to employ Eq. 17 provided P_W is known. This procedure finds an important use in design where axial forces have to be predicted quickly and with sufficient accuracy.

Inelastic Behavior

Intermediate Stories

Prediction of the formation of a hinge at the leeward end of a beam in an intermediate story is carried out using the following expressions:

$$S_T = \frac{12E}{h^3}\sum_1^m\left[\frac{I_c}{1+2\Psi}\right] - (\lambda_p + \delta\lambda)\frac{\Sigma P}{h} \tag{18}$$

$$\delta\Delta = \frac{\delta\lambda \cdot \Sigma Q}{S_T} \tag{19}$$

$$(M_b)_p + R\,\frac{12EI_c}{h^2(1+2\Psi)}\delta\Delta + \delta\lambda\ \text{FEM} = M_p \tag{20}$$

$$\lambda_n = \lambda_p + \delta\lambda \tag{21}$$

in which $\delta\Delta$ = sway increment caused by a load factor increment $\delta\lambda$, λ_p = load factor in previous cycle = zero in the first cycle, λ_n = load factor in present cycle, $(M_b)_p$ = total moment on the beam in previous cycle = zero in first cycle, and R = beam stiffness ratio which changes according to the hinge formation, Figure 3.

The procedure works in the following manner. Assume that there are no hinges present in the story. Set $(M_b)_p = \lambda_p = 0$ for all beams. Substitute Eqs. 18 and 19 into Eq. 20 and calculate $\delta\lambda$ as the load factor increment required for a hinge to form in each beam. The smallest $\delta\lambda$ corresponds to the first hinge in the story. Calculate $\delta\Delta$ from Eq. 19. The increment in the leeward moment in each beam δM_b caused by $\delta\Delta$ is calculated,

$$\delta M_b = R\,\frac{12EI_c}{h^2(1+2\Psi)}\delta\Delta + \delta\lambda\ \text{FEM} \tag{22}$$

Add δM_b to $(M_b)_p$ of the previous cycle to give the actual total moment of the beam. Recalculate beam stiffnesses and repeat above process.

Hinges forming in the windward ends of the beams are predicted in a similar way except that FEM has to be re-determined when a leeward hinge exists and a negative sign must be associated with it. Furthermore, the right hand term in Eq. 20 is the lesser value of M_p or M_{pw},

$$M_{pw} = \lambda\,\frac{wL^2}{2}\left[\left(\frac{16M_p}{\lambda wL^2}\right)^{1/2} - 1\right] - M_p \tag{23}$$

Collapse occurs when $S_T \leq 0$.

Formation of hinges in the columns is calculated in a similar way, but with Eq. 20 replaced by Eq. 24 and Eq. 22 by Eq. 25,

$$(M_c)_p + \frac{6EI_c}{h^2(1+2\Psi)}\delta\Delta + \frac{\delta\lambda}{2}(\text{FEM})_c = 1.18\,(M_p)_c\left[1 - (\lambda_p + \delta\lambda)\frac{P_w}{P_y}\right] \tag{24}$$

$$\delta M_c = \frac{6EI_c\,\delta\Delta}{h^2(1+2\Psi)} + \frac{\delta\lambda}{2}(\text{FEM})_c \tag{25}$$

in which $(M_c)_p$ = total moment applied to column in previous cycle = zero in first cycle, $(M_p)_c$ = full plastic moment of column, P_y = column yield load and δM_c = increment in column moment caused by $\delta\lambda$. A test is always made in the calculations to determine whether hinges form in the beams or in the columns and subassemblage stiffnesses are changed accordingly.

Frame A was analyzed by the method of Ref. 12 and the weakest story was found to be the one below level 26. This story was then analyzed by the present technique. Comparison of results in Figure 7(a) shows good agreement. In order to investigate the effect of omitting column shortening in the present treatment, the story below level 26 was deliberately strengthened using W14X500 columns, and the story below level 4 weakened using W10X66 columns and W21X44 beams. Results of analyses are shown in Figure 7(b). If is observed that column shortening effects may be neglected when computing ultimate strength. A similar observation was made by PARIKH [12].

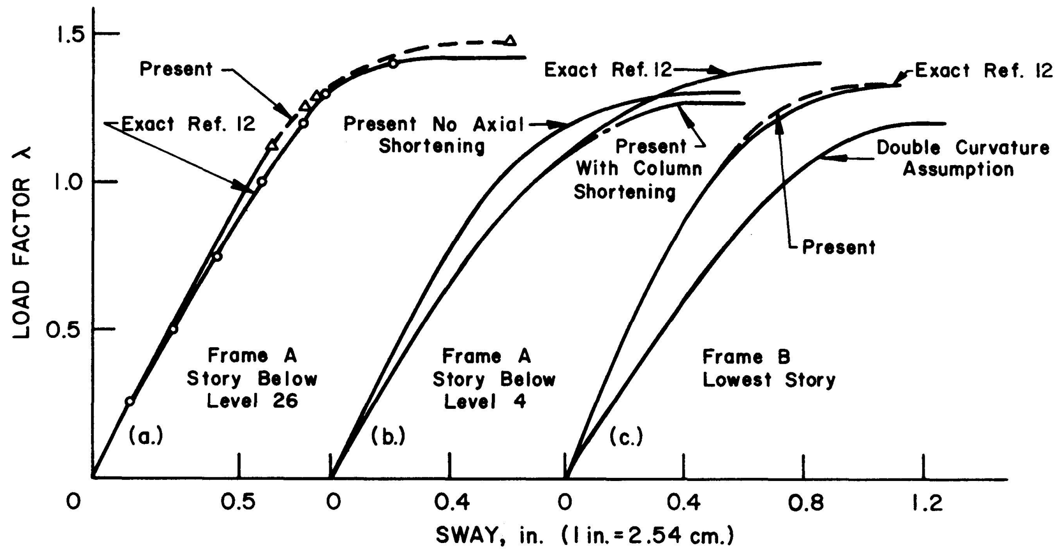

Fig. 7. Comparison of Methods.

Lowest Story

Expressions similar to Eqs. 18-24 can be derived for the lowest story. However, a computer program was written to remove the tediousness associated with keeping track of hinge formations and changing member stiffnesses. Note that a hinge will form at the fixed base of a column before appearing at the top since $M_L > M_C$, and that when the hinge has formed, the subassemblage stiffness is no longer given by Eq. 9, but by Eq. 10. The lowest story of Frame B which is described in Table 1 and Figure 4 was analyzed and the results are plotted in Figure 7(c). The conservativeness which follows from assuming exact double curvature bending in the lower columns is also demonstrated in Figure 7(c).

It must be remarked that when the number of bays is few and when $\Psi > 1.0$, Eq. 9 tends to be unconservative in the elastic range, but sufficiently accurate in predicting ultimate strengths.

Design

It has been demonstrated how relevant quantities such as bending moments, sway and axial forces can be evaluated up to the collapse load factor once story stiffnesses have been computed. It has also been shown that changes in column sizes do not affect story stiffnesses to any significant extent provided the ratio of column stiffness to beam stiffnesses $\Psi > 0.5$. In fact, they do not affect story strengths to any appreciable degree if the columns stay elastic.

This information can be fully exploited in design by forcing the columns to remain elastic until the point of collapse and by modifying the beams to satisfy story strength requirements. A simple design procedure is now proposed. However, the following fixed quantities must be given: frame dimensions, applied gravity and wind loads, required failure load factor under pure gravity loads λ_g, and required failure load factor under combined lateral and vertical forces λ_F.

A typical set of design steps would be:

STEP 1 – Obtain a preliminary design using λ_g and simple plastic theory, neglecting secondary effects [5].

STEP 2 – Calculate story stiffness S_T and sway Δ at $\lambda = 1.0$. Calculate required plastic moment of beams using Eq. 16 by replacing M_b by M_p. This step ensures that no hinges form in the beams at working load level.

STEP 3 – Analyse the story assuming that the columns remain perfectly elastic. The load factors at which the leeward beam hinges form will be greater than 1.0, but may be less or greater than λ_F. Ensure that the windward hinges develop at $\lambda > \lambda_F$ by using Eqs. 20 and 23. Increase beam sizes if necessary.

STEP 4 – From step 4, all column axial forces and bending moments are known for load factors up to values slightly larger than λ_F. Choose sections sizes so that the columns fail at a load factor slightly larger than λ_F.

STEP 5 – If stiffness constraints are imposed, calculate required story stiffness. Use Eq. 8 directly to compute required size of new girders. [See also Refs. 3 and 4.]

Comments

The above-listed routine actually recommends the design to follow curve AB in Figure 8. All plastic hinges are initially compelled to develop in the beams which are selected so that $\lambda_c \geq \lambda_F$, where λ_c = critical load factor based upon the premises that the columns do not yield. Usually, the columns selected in the preliminary design will have yielded before λ_F is reached. By a judicious choice of larger column sections, hinges are forced to form in the columns at a load factor not less than λ_F. As soon as the hinges appear in the columns, a drastic decrease in story stiffness occurs and the design follows path CD. Considering curve ACD, it is appreciated that λ_F has now become the failure load factor of the story designed.

Frame B shown in Figure 4 was designed for $\lambda_g = 1.7$ and $\lambda_F = 1.3$, with no stiffness constraints imposed at $\lambda = 1.0$. The member sizes are those given in Table 1. This frame was analyzed by the method of Ref. 12 which considers it in its entirety. Some results are plotted in Figure 9. It can be observed that all the lower stories have almost the same ultimate strengths. They fail at a load factor slightly above 1.3, thereby indicating a well balanced frame. Note that the frame and its stories have the same load-deflection characteristics.

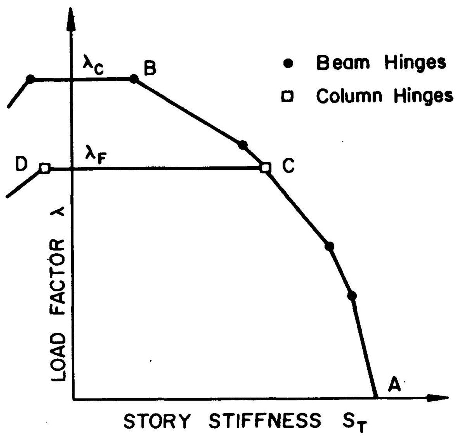

Fig. 8. Load-stiffness Path of Typical Design.

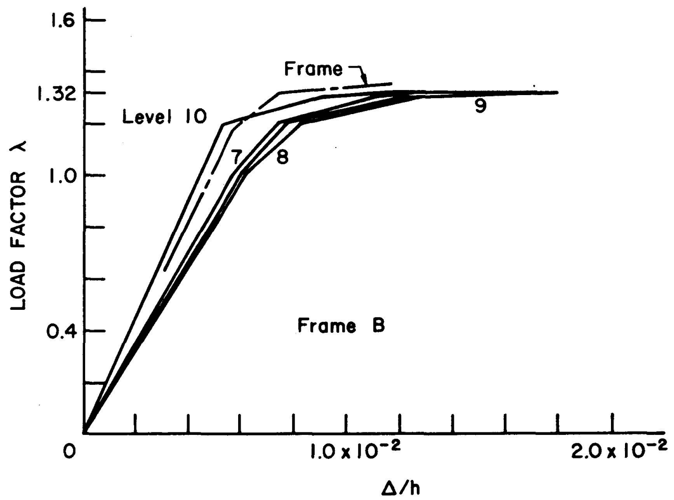

Fig. 9. Load-Deformation Curves of Frame B Stories.

For further illustration of the application of the present method, Frame B is proportioned to meet a second-order deflection constraint of 0.003h per story. Rapid calculations of the ratio of column stiffness to beam stiffnesses, Eq. 12, shows that the quantity Ψ is of the order of 0.5 for the interior subassemblages. As explained in Refs. 3 and 4, changing beam sizes, in this particular case, is not the most effective way of reducing sway since the design will result in very large girders and relatively small columns.

One procedure that may be adopted is to increase all member sizes in the same proportion. The final frame dimensions are given in Figure 10. Obviously, this frame has an ultimate load factor greater than 1.3 since it has larger member sizes than Frame B given in Table 1.

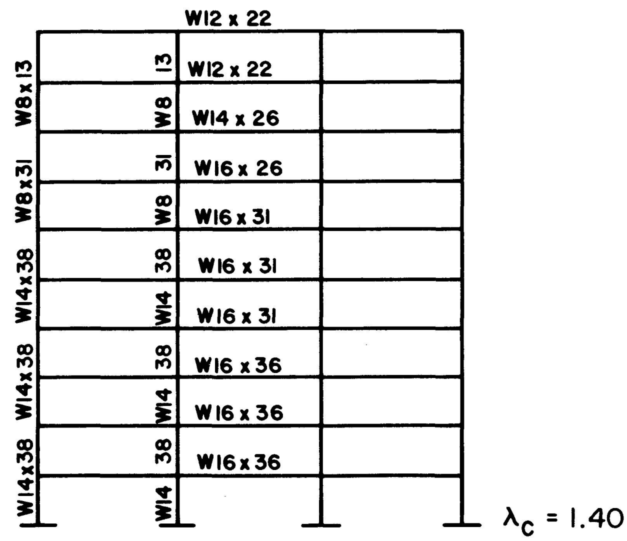

Fig. 10. Frame B Designed for Stiffness ($\Delta/h = 0.003$).

It may be pointed out that, generally, in most unbraced multistory steel frames $\Psi > 0.5$ so that only beam sizes need be increased to satisfy stiffness limits. A typical example can be found in Refs. 3, 14 and 16.

Note further that the above-designed frames illustrate the advantages of using subassemblage techniques as design tools. Consider, for instance, the conventional allowable stress method [2]. Neither does it aim at satisfying strength nor does it steer the design to neet deflection constraints, except through the use of additional auxiliary techniques. In the present treatment, however, the sway and strength of each story can be controlled individually.

Safety and Economy

The use of plastic theory in the design of unbraced multistory steel frames has been advocated because of the economic advantages it offers over allowable stress concepts. The penalty for employing plastic theory, however, is the difficulty in accounting for the secondary moments which have a pronounced influence on the failure modes and the computed collapse loads. Existing plastic design methods attempt to overcome this problem by assuming failure mechanisms and sway values at collapse. This procedure generally requires several iterations in analysis before agreement is obtained between assumed sways and actual ones.

The present technique eliminates the need for the iterations by choosing stiffness as the main variable instead of sway. After all, sway and ultimate strengths are merely functions of story stiffness. Furthermore, the one concept presented is amply satisfactory for both strength and stiffness design, requiring no additional sub-routines. This is a feature not available in existing design techniques.

Rapidity in analysis and design is realized because of the simple nature of the working algebraic equations. In the advent of the unavailability of a computer, the method is very suitable for hand calculations.

The weight of the structure designed for strength tends to be close to optimum because the member sizes are so selected as to just satisfy strength. Moreover, all stories are forced to fail at the same pre-selected load factor. Consequently, the stories are of almost equal strength and the frame is a well balanced one.

The accuracy of the method has been well substantiated by comparison with a conventional second-order elastic-plastic method which considers the frame in its entirety. The present treatment has a tendency to err slightly on the conservative side. As a consequence, it produces safe designs.

Conclusion

The power of using story stiffness as the basic independent variable in the analysis and design of unbraced multistory steel frames has been demonstrated. Unlike previously proposed techniques, the present treatment can analyze without difficulty the lowest story in a frame and can predict with sufficient accuracy frame behavior in the elastic and inelastic range under proportional loading conditions. As a design tool, it eliminates the need to assume failure mechanisms and sways at ultimate strength. Neither does it require auxiliary sub-programs to satisfy strength and stiffness requirements, nor does it nêcessitate the use of iterations to reach the final design. Moreover, while it ensures that all stories remain elastic at the working load level, it also proportions them to fail at about the same load factor, at a value somewhat higher than a pre-selected one. Consequently, the resulting design is well balanced. The accuracy of the method has been confirmed by comparison with a conventional second-order, elastic-plastic method.

Notations

A, B	constants of integration.
E	Young's Modulus.
FEM	beam fixed-end moments.
$(FEM)_c$	resultant FEM at a joint.
I_{b1}, I_{b2}	inertias of beams in a subassemblage.
I_c	column inertia.
K^2	P/EI_c.
L_{b1}, L_{b2}	lengths of beams in a subassemblage.
M_b	leeward-end beam moment.
M_c	upper-end column moment.
M_L	moment at lower end of column with fixed base.
M_P	full plastic moment of beam.
M_{pw}	applied moment causing hinge formation at windward end of beam.
M_x	moment at any point x in column.
$(M_b)_p$	beam moment in previous cycle of calculation.
$(M_c)_p$	column moment in previous cycle of calculation.
$(M_p)_c$	column full plastic moment.
P	column axial load.
P_w	column force at working loads.
P_λ	column force at load factor λ.
ΣP	total gravity loads on a story.
Q	column shear force.
ΣQ	total shear on a story.
R	ratio of leeward beam stiffness to windward beam stiffness.
S_T	story stiffness.
U	ratio of restraining beam moments to column moment.
h	story height.
s	subassemblage stiffness.
w	distributed floor load.
Δ	story sway.
δM_b	increment in beam leeward moment due to $\delta\Delta$.
δM_c	increment in column moment due to $\delta\Delta$.
$\delta\Delta$	increment in story sway caused by $\delta\lambda$.
$\delta\lambda$	increment in load factor.
λ	load factor.
λ_B	load factor less than λ_c.
λ_c	maximum load factor.
λ_g	required failure load factor under pure gravity forces.
λ_F	required failure load factor under combined loads.
λ_n	load factor in present cycle of calculations.
λ_p	load factor in previous cycle of calculations.
Ψ	ratio of column stiffness to restraining beam stiffnesses.

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Summary

The use of subassemblage stiffness as the main variable can facilitate the satisfaction of both strength and stiffness constraints in the design of unbraced multistorey steel frames. Strength is merely considered as a function of detoriorating stiffness. As a design tool, the present treatment overcomes the limitations associated with the assumption of sways and collapse mechanism at failure, a feature generally present in conventional plastic design technics. It can control the individual stories to fail at pre-selected load factors resulting in a frame which is neither over-designed nor under-designed.

Résumé

Le choix de la rigidité de sous-ensembles comme variable principale dans le calcul des ossatures métalliques en cadres non contreventés peut permettre de remplir plus facilement les conditions de résistance et de rigidité. La résistance y est considérée uniquement comme fonction de la diminution de rigidité. Comme outil de calcul, la méthode proposée permet d'éviter les limites associées à l'adoption de mécanismes de déplacements et de ruine, c'est-à-dire une caractéristique générale du calcul plastique conventionnel. On peut ainsi contrôler à la ruine chaque étage en admettant des facteurs de pondération choisis au préalable de façon à obtenir une ossature qui ne soit ni sur- ni sous-dimensionnée.

Zusammenfassung

Die Wahl der Steifigkeit von Rahmeneinheiten als Hauptvariable beim Entwurf von seitlich verschieblichen stählernen Stockwerkrahmen kann die Erfüllung sowohl der Festigkeits- als auch der Steifigkeitsbedingungen vereinfachen. Die Festigkeit wird dabei lediglich als Funktion einer Steifigkeitsverminderung betrachtet. Als Entwurfsgrundlage vermeidet die vorliegende Behandlung die Begrenzungen, die mit der Annahme von Verschiebungs- und Versagensmechanismen gekoppelt sind, eine im normalen Traglastverfahren meistens vorkommende Erscheinung. Die einzelnen Stockwerke werden für im voraus gewählte Lastfaktoren auf Versagen so bemessen, dass der Rahmen weder über- noch unterbemessen ist.

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