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PLASTIC DESIGN OF EARTHQUAKE-RESISTANT K-BRACED MULTI-STORY STEEL FRAMES

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SUMMARY

A plastic design procedure is proposed for K-braced multi-story steel frames subjected to seismic or wind lateral forces. This procedure gives a reasonable solution to a preliminary design problem how to assign the distribution of the resisting story-shear force to the bracings against the design lateral forces. Moreover, this paper introduces a numerical method, developed by authors, which is able to predict with good accuracy the elastic-plastic behavior of braced steel frames. The method is a combined geometrically and materially nonlinear one-dimensional finite element method with an incremental perturbation technique.

INTRODUCTION

Many previous studies (for example, Ref.1) on simple K-braced frames suggest that the most efficient means to improve the overall toughness of K-braced multi-story steel frames are (1) to avoid the formation of collapse mechanisms on the beams in the braced bays and (2) to prevent the early growth of yieldings on columns (Ref.2). In this paper, an efficient method of preliminary plastic design of multi-story K-braced steel frames is developed taking into account these two design requirements. A basic design factor β_j, the shear force ratio of bracings, is defined for the j-th story as the ratio of the story-shear-force to be assigned to the bracings to the prescribed design shear force of the story. Procedures of computing the vertical distributions of β_j are demonstrated. All the necessary informations for the preliminary plastic design can be easily derived from these β_j values. A design example of ten-story three-bay K-braced frames shows that the method is very simple and effective to provide reasonable member sections.

After some benchmark tests, the designed frame is analyzed its performance by the nonlinear F.E.M. (Refs.3,4) with an incremental perturbation method originally developed by Nakamura and Uetani (Refs.5,6).

PLASTIC DESIGN PROCEDURE

<u>Shear force ratio of bracings</u> A plane K-braced frame of f stories with s spans as shown in Fig.1 is to be designed so as to satisfy the design requirements. When the beams in the braced bays do not form collapse mechanisms, the story-shear force Q_{Bj} carried by braces in the j-th story is

$$Q_{Bj} = \beta_j Q_j = (1 + \gamma_{CTj})\, {}_{s}B_j N_{Tj} \cos\theta_j \qquad (1)$$

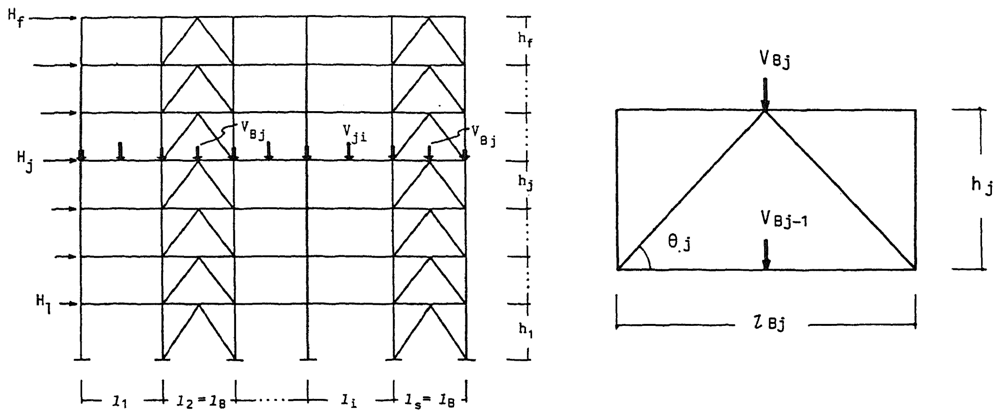


Fig.1 f-story s-bay K-braced Frame

where Q_j=prescribed design shear force, s_{Bj} =number of the braced bays, N_{Tj} =axial force in a tension brace at R_j, N_{Cj}=absolute value of axial force in a compression brace at R_j, R_j=prescribed design deflection angle, and $\gamma_{CTj} = N_{Cj} / N_{Tj}$.
The beam connected with braces is subjected to a transverse force equal to the sum of the vertical component of the axial forces of braces. The maximum transverse force P_j to be predicted is

$$P_j = (N_{Pj} - N_{Cj}) \sin\theta_j = (1 - \gamma_{CPj}) N_{Pj} \sin\theta_j \qquad (2)$$

where N_{Pj} =axial yield strength of the brace, and $\gamma_{CPj} = N_{Cj} / N_{Pj}$. Therefore, the fully plastic moment B_{Bj} of the beams in the braced bays is required to satisfy,

$$B_{Bj} \geqq (P_j + V_{Bj}) l_{Bj} / 4 \qquad (3)$$

The floor moment M_j is

$$M_j = \zeta_j^T (1 - \beta_j) m_j + \zeta_{j+1}^B (1 - \beta_{j+1}) m_{j+1} \qquad (4)$$

where $m_j = Q_j h_j$ and $\zeta_j^T (\zeta_j^B)$=moment distribution factor assigned to the top (bottom) of the columns. If these two design requirements are satisfied, the formation of plastic hinges is restricted to take place at the both ends of the beams in the braced bays. On the other hand, in beams of unbraced bays, the following two cases of plastic hinge generations are possible :

$$\text{CASE 1} \qquad \frac{M_j - 2s_{Bj} B_{Bj}}{2(s - s_{Bj})} \geqq \frac{V_{ji} l_i}{4} \qquad \text{: both ends} \qquad (5a)$$

$$\text{CASE 2} \qquad \frac{M_j - 2s_{Bj} B_{Bj}}{2(s - s_{Bj})} < \frac{V_{ji} l_i}{4} \qquad \text{: middle and leeward end} \qquad (5b)$$

The relation between the sum of end moments of beams and the floor moment can be written as

$$M_j = 2s_{Bj} B_{Bj} + 2 (s - s_{Bj} + n_{qj}) B_j - \sum_q (V_{ji} l_i / 2) \qquad (6)$$

where B_j =required fully plastic moment of beams in the unbraced bays and n_{qj} =number of beams with plastic hinges in case 2. From Eqs.(1)-(4) and (6), the inequality equation on β_j is given by

$$\beta_j \leqq \frac{s_{Bj}}{s_{Bj}\zeta_j^T + \nu_j \Gamma_j} \{ \zeta_j^T + \zeta_{j+1}^B (1 - \beta_{j+1}) \frac{m_{j+1}}{m_j} - \frac{V_{Bj} l_{Bj} \nu_j}{2 m_j} + \frac{\Sigma (V_{ji} l_i / 2)}{m_j} \} \qquad (7)$$

where $\Gamma_j = (1 - \gamma_{CPj}) / (\gamma_{TPj} + \gamma_{CPj})$, $\nu_j = s_{Bj} + \rho_j (s - s_{Bj} + n_{qj})$,

$\gamma_{TPj} = N_{Tj} / N_{Pj}$, $\rho_j = B_j / B_{Bj}$.

Evaluation of γ_{TPj}, γ_{CPj} The coefficients γ_{TPj} and γ_{CPj} depend on the slenderness ratio of braces and the design deflection angle of the j-th story. Many tests on steel braces under repeated axial loading (for example, Ref.7) have been carried out. The γ_{TPj} and γ_{CPj} are possible to be evaluated by the empirical equations proposed in the papers on those tests. The empirical equation by Wakabayashi, Matsui and Mitani (Ref.8) is adopted here.

Example By adopting the upper bound values of β_j in Eq.(7), a K-braced frame shown in Fig.2 is designed here. The design story-shear forces are evaluated from Japanese Seismic Design Code. The design deflection angles of all stories are 0.01, the ratio ρ_j of B_j to B_{Bj} is 0.5, and the effective length of all braces for buckling is assumed to be $1/\sqrt{3}$ times the original length. Columns and beams are designed according to Guide of Plastic Design of Structures (AIJ, 1975). In designing individual members, the member sizes are treated as continuous quantity (Ref.9). Table 1 shows cross-sectional areas and moments of inertia of the members.

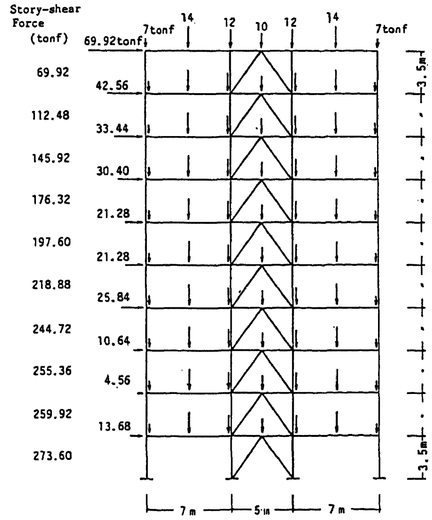


Fig.2 Design Frame

Table 1 Cross-Sectional Areas and Moments of Inertia

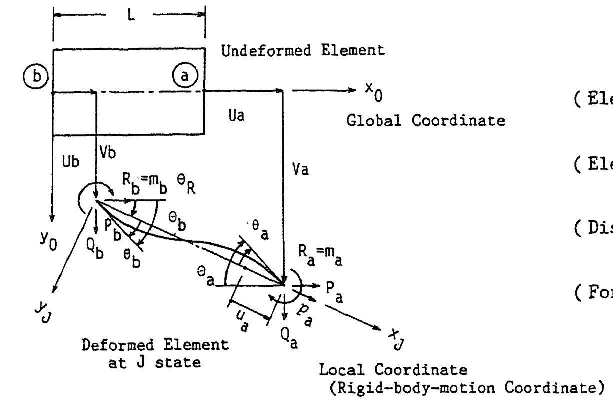
story	β	Brace A	Brace I	Internal Beam A	Internal Beam I(×10³)	External Beam A	External Beam I(×10³)	Internal Column A	Internal Column I(×10³)	External Column A	External Column I(×10³)
10	0.182	11.4	46.0	98.5	28.5	62.0	11.3	181	32.9	87.2	7.60
9	0.259	22.7	184	130	49.4	81.6	19.6	193	37.4	90.0	8.09
8	0.360	36.5	473	160	75.1	101	29.8	230	53.1	104	10.7
7	0.386	44.7	710	174	89.5	110	35.5	279	77.7	120	14.5
6	0.423	52.3	972	186	102	117	40.4	321	103	132	17.5
5	0.440	58.2	1200	194	111	122	44.0	372	138	146	21.4
4	0.462	63.5	1430	200	118	126	47.0	421	177	158	24.9
3	0.474	69.1	1700	207	126	130	49.9	480	230	172	29.6
2	0.494	72.1	1850	210	129	132	51.4	531	282	181	32.8
1	0.428	67.4	1610	205	124	129	49.0	637	406	218	47.5

A:Cross-sectional Areas (cm^2) I:Moment of Inertia (cm^4)

NUMERICAL ANALYSIS

Numerical method In 1973, Nakamura, Ishida et al.(Ref.3) proposed the numerical method to simulate the static and dynamic collapse behavior of steel plane frames. In this method, a local moving coordinate system called "Rigid-body-motion coordinate" is applied, which enables to separate the real deformation of each element from its rigid body displacement. And a transfer matrix technique in a form extended so as to incorporate the effect of accumulated large deflection is applied to the member as a subsystem consisting of one-dimensionally connected elements. Recently, this method is called FERT(Finite Element method with Rigid-body-motion coordinates and Transfer matrix technique). The FERT adopts the linearized incremental governing equations. The stiffness values evaluated at the starting point of a step have been used for the linearized incremental stiffness values by assuming that the variations of the stiffness values would be small during the step so far as a sufficiently small step length has been chosen. The decision of the step length is the most significant matter particularly in numerical investigations of critical behaviors and post-buckling behaviors.

The incremental perturbation method proposed by Nakamura, Uetani et al.(Refs.5,6) applies the perturbation procedure not only to the non-linear strain-displacement relations and equilibrium equations, but also to the



(Element-end Displacements in Global Coordinate)

$\{ D \} = \{ U_a \ \ V_a \ \ \Theta_a \ \ U_b \ \ V_b \ \ \Theta_b \}^T$

(Element-end Forces in Global Coordinate)

$\{ P \} = \{ P_a \ \ Q_a \ \ R_a \ \ P_b \ \ Q_b \ \ R_b \}^T$

(Displacements in Local Coordinate)

$\{ d \} = \{ u_a \ \ \theta_b \ \ \theta_a \}^T$

(Forces in Local Coordinate)

$\{ p \} = \{ p_a \ \ m_b \ \ m_a \}^T$

Fig.3 Local and Global Coordinates for an Element

constitutive equations in the terms of rate equations. Therefore all the governing equations can be satisfied to any desired accuracy at every instantaneous configuration in between the starting and terminal points of an incremental step. Moreover the method provides means of finding, to a desired accuracy, every point on an equilibrium path of a discrete system at which a new element will start yielding or unloading. The computational step lengths are automatically determined by the termination conditions,i.e. truncation error, yielding or unloading, etc.

By the application of the incremental perturbation method to the FERT, a refined numerical program FERT-P is developed here. Fig.3 shows the finite element and the two coordinate systems. All the variables are expanded in Taylor series of t about the known configulation J at which t=0: for example,

$$\{ D(t) \} = \sum_{M=0}^{\infty} \{ D^{(M)} \} \, t^M \tag{9}$$

Finally, the following perturbation equation may be derived:

$$[K^{(0)}] \{ D^{(M)} \} = \{ P^{(M)} \} - \{ \hat{P}^{(M)} \} \tag{10}$$

The matrix $[K^{(0)}]$ in Eq.(9) can be evaluated from quantities at the starting point of a step and the vector $\{\hat{P}^{(M)}\}$ contains only those terms lower than the M th order. The $\{\hat{P}^{(M)}\}$ which is of special importance in the FERT-P relates the transformation of the two coordinates shown in Fig.3 (Ref.11).

Fig.4 shows the comparison of the analysis prediction by the FERT-P with the experimental result by Wakabayashi et al.(Ref.7) on the behavior of a steel bar under repeated axial loading. In Fig.4, a dot or a round mark is plotted at every incremental step. The solid line in Fig.5 predicts the cyclic behavior of two-story one-bay K-braced steel frame tested by Igarashi et al. (Ref.10). The hysteretic stress-strain relation (Ref.1) illustrated in Fig.4 is adopted in these numerical tests and the following example. A cross-section is idealized as an equivalent four-material-point section. With a very good accuracy, the FERT-P program predicted these experimental results.

Static Behavior under Lateral Loads Proportional to Design Seismic Loads The lateral loads proportional to the design seismic loads have been applied to the example frame in Fig.2 and increased under the constant gravity loads. Fig.5 shows the base shear-structural rotation θ_{ST} diagram. The curve has been obtained by the displacement-controlled incremental analysis (Ref.12). Fig.5 includes the deformed configuration and the extent of the plastic region at θ_{ST}=0.0102. The displacement scale is two times the scale of the frame geometry. These numerical results suggest that the example frame sufficiently satisfies the design requirements and holds the overall structural toughness.

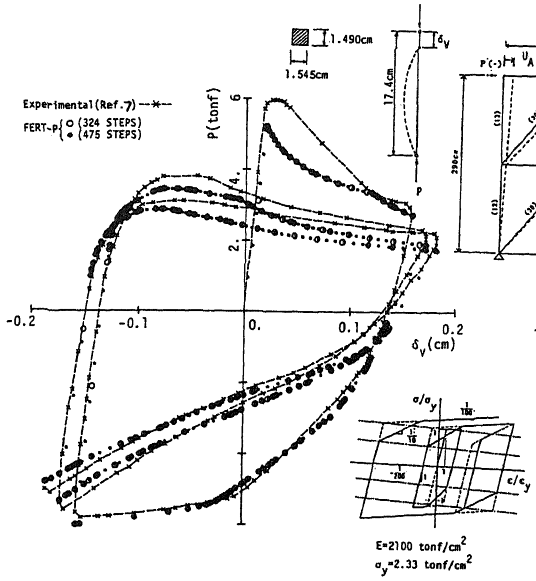


Fig.4 Behavior of a Steel Bar under Repeated Axial Loading

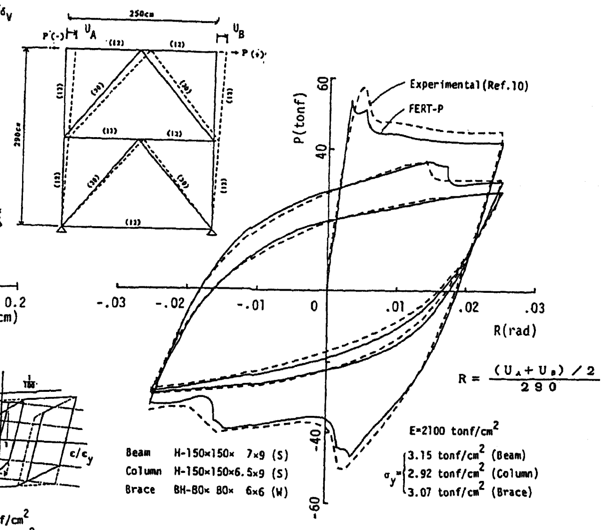


Fig.5 Behavior of Two-story One-bay K-braced Steel Frame under Repeated Lateral Loading

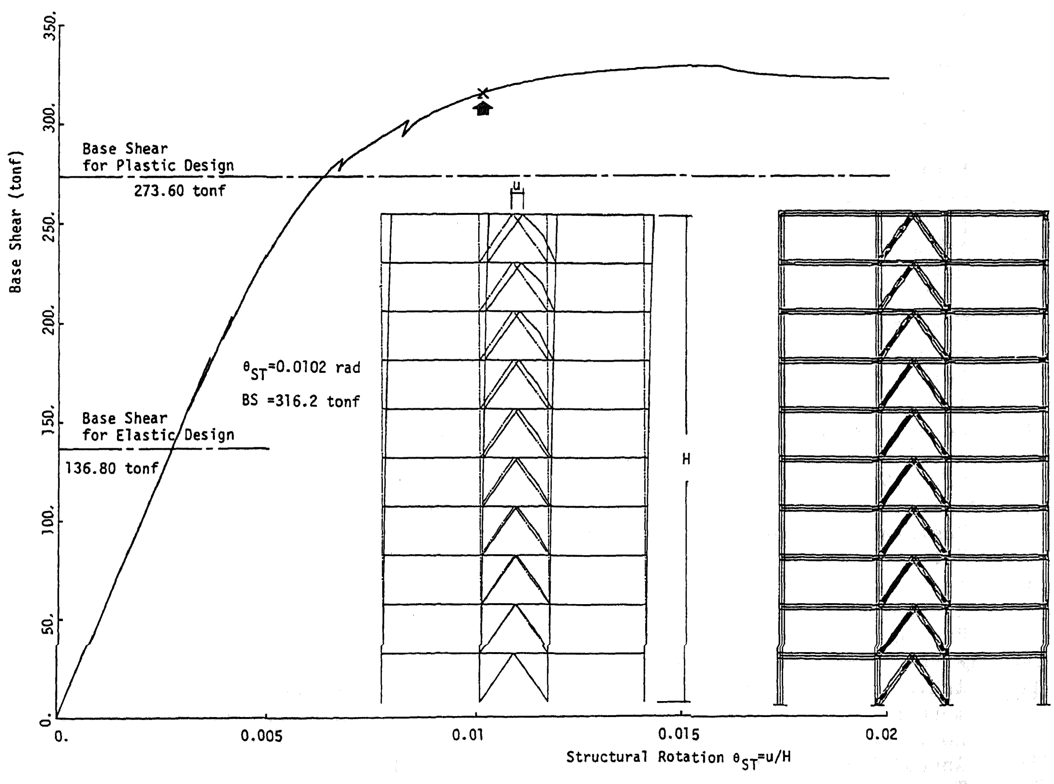


Fig.6 Base Shear - Structural Rotation Diagram for Design Frame

CONCLUSIONS

1) An efficient method of preliminary plastic design of K-braced multi-story steel frames is proposed.
2) Through an example design of K-braced multi-story frame, the method is verified to be very simple and effective to provide reasonable member sections.
3) The structural performance of the example frame is analyzed by a refined numerical program FERT-P developed here.
4) The numerical results suggest that the frame satisfies the design requirements and holds the sufficient overall structural toughness.

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