AN EMPIRICAL EVALUATION OF INELASTIC BEHAVIOR OF STRUCTURAL ELEMENTS IN REINFORCED CONCRETE FRAMES SUBJECTED TO LATERAL FORCES

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SYNOPSIS

Test results of reinforced concrete framing elements, i.e., beams, columns, shear walls and beam-column connections were reviewed to derive equations for skeleton curves of restoring force characteristics. Based on some discussions of influences of geometry and materials on test results, empirical equations of strength and inelastic stiffness of the elements were proposed by statistical method.

INTRODUCTION

A practical method to evaluate inelastic seismic deflection of reinforced concrete frames based on the test results of elements was proposed[1)] and was further developed[2),3)]. For this mehtod as well as general earth quake resistant design of frames, existing data of the elements should be systematically and statistically reviewed to predict more realistic restoring force characteristics.

For the first step of the study, the authors investigated skeleton curves of restoring force characteristics which can generally be idealized into tri-linear model. 22 papers were reviewed[4)~25)] and test data of more than 250 rectangular beams and columns, 120 framed shear walls without opening and 40 beam-column connections were studied with respect to strength and inelastic stiffness which define the tri-linear relation in Figs. 1 to 3. Influences of geometry and materials on the behavior were discussed with regression analysis and set of empirical or approximate equations were proposed. Reliability and application of the equations were also discussed.

It was observed that yield strength and stiffness at yielding of the elements could be estimated with fair accuracy by the proposed equations. On the other hand, measured cracking strength had wide scatter to the proposed equations. However error in estimating cracking strength is not very critical in frame analysis because the idealized tri-linear relation is not very sensitive to the cracking strength.

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INELASTIC BEHAVIOR OF BEAMS AND COLUMNS FAILING IN BENDING

The tri-linear relation of beams and columns in Fig. 1 is defined by cracking moment M_c, yield moment M_y and stiffness reduction at yielding α_y. They are estimated by following set of equations.

According to elastic beam theory, cracking moment M_c is expressed by

$$M_c = ({}_c\sigma_t + N/A)\, Z \qquad \ldots\ldots 1)$$

where Z and A, transformed uncracked section modulus and area, and N, axial compression on the section. ${}_c\sigma_t$ which normally corresponds to modulus of rupture of concrete is found as follows from eq. 1) and measured M_c

$${}_c\sigma_t = 1.8\sqrt{F_c} \qquad (\mathrm{kg/cm^2}) \qquad \ldots\ldots 2)$$

where F_c is compressive strength of concrete.

Instead of analyzing yield moment M_y rationally, the authors attempted to estimate M_y with much simpler approach. Assuming tension and compression steel yield, we can obtain following equation,

$$M_y = (g_1 p_t \sigma_y / F_c + 0.5\eta_o (1-\eta_o))\, F_c b D^2 \qquad \ldots\ldots 3)$$

where g_1 is the distance between tension and compression steel divided by depth of section, p_t and σ_y, ratio and yield strength of tensile steel and η_o=N/(bDF_c). However eq. 3) is applicable even if compressive steel does not yield and test results show satisfactory agreement, i.e., 90% of data has within 20% of error.

Stiffness reduction at yielding α_y is influenced by tensile steel np_t (n: modular ratio), span to depth ratio a/D, axial force η_o and effective depth ratio d/D. An empirical equation is obtained as follows,

$$\alpha_y = (0.043 + 1.64np_t + 0.043a/D + 0.33\eta_o)\, (d/D)^2 \qquad \ldots\ldots 4)$$

and test results show good agreement, i.e., 90% of data has within 30% of error. This equation may be used for members with p_t=0.4~2.8%, a/D=2~5 and η_o=0~0.55.

INELASTIC BEHAVIOR OF FRAMED SHEAR WALLS WITHOUT OPENING

Cracking shear stress τ_c, yield shear stress τ_y and stiffness reduction at yielding β_y define the tri-linear relation of shear walls in Fig. 2. Here shear stress is expressed by average, i.e., $\tau=Q/A_w$; Q is shear force and A_w is section area of wall.

Effect of axial force on τ_c is not considered here because it is usually transmitted to columns and wall panel relatively free of axial force. It appears that τ_c is influenced by F_c and stress due to bending, and is expressed by empirical equation in terms of F_c and A_g/A_w,

$$\tau_c = (4.3\ A_g / A_w + 0.05)\ F_c \qquad (kg/cm^2) \qquad .....5)$$

where A_g is total area of steel in a column.

The authors attempted to estimate yield stress τ_y introducing "truss effect" of concrete in wall panel. Assuming that concrete diagonal member would crush at yielding, τ_y can be estimated as the sum of strength of truss and wall reinforcement if the effective width of diagonal member B_e is known

$$\tau_y = B_e \cdot F_c / \sqrt{\ell^2 + h^2} + p_w \cdot {}_w\sigma_y \cdot h/\ell \qquad .....6)$$

According to test results, first term of eq. 6) can be estimated by following equation,

$$B_e \cdot F_c / \sqrt{\ell^2 + h^2} = \left\{ A_g \cdot \sigma_y / A_w + p_w \cdot {}_w\sigma_y (1-h^2/\ell^2)/2 + N/(2A_w) \right\} \ell/h$$

$$\leqq 5.6t \cdot F_c / \sqrt{\ell^2 + h^2} \qquad .....7)$$

where ℓ,h,t, are length, height and thickness of wall respectively and p_w and ${}_w\sigma_y$ are steel ratio and yield strength of wall reinforcement. Eqs 6) and 7) show good agreement, i.e., 80% of test data has within 20% of error.

Stiffness reduction at yielding β_y is expressed by following equation in terms of $p_w \cdot {}_w\sigma_y$ and F_c,

$$\beta_y = 0.46 \cdot p_w \cdot {}_w\sigma_y / F_c + 0.14 \qquad .....8)$$

INELASTIC BEHAVIOR OF BEAM-COLUMN CONNECTIONS

Cracking shear stress τ_c, yield shear stress τ_y and yield shear strain γ_y define the tri-linear relation in Fig. 3. Here shear stress is expressed as follows,

$$\tau = (1-u-v) \cdot Q \cdot h / (tu\ell vh) \qquad .....9)$$

where Q,h and ℓ are column shear force, story height and span and t,uℓ and vh are thickness, width and height of connection panel respectively.

A rational expression for cracking shear stress τ_c is

$$\tau_c = \sqrt{{}_c\sigma_t^{\,2} + {}_c\sigma_t \cdot \sigma_n} \qquad (kg/cm^2) \qquad .....10)$$

where σ_n is normal stress due to column load (compressive positive). ${}_c\sigma_t$ which normally corresponds to tensile strength of concrete is found as follows from eq. 10) and measured τ_c

$${}_c\sigma_t = 0.205 \cdot F_c - 0.0004 \cdot F_c^{\,2} \qquad (F_c < 420\ kg/cm^2) \qquad .....11)$$

It appeared in the test results, that yield shear stress τ_y is little affected by axial stress σ_n. Assuming that τ_y is expressed by equation 12),

$$\tau_y = \tau_{conc} + 2.7\sqrt{p_w \cdot {}_w\sigma_y} \qquad .....12)$$

where p_w and ${}_w\sigma_y$ are steel ratio and yield strength of shear reinforcement, the relation between τ_{conc} and F_c is expressed by following empirical equation.

$$\tau_{conc} = 0.51 \cdot F_c - 0.001 \cdot F_c^2 \qquad (F_c < 420\ kg/cm^2) \qquad \ldots\ldots 13)$$

All of 31 test results which failed at connections show good agreement within 20% of error. Yield shear strain γ_y is generally greater than 0.4%, and

$$\gamma_y = 0.5\ \% \qquad \ldots\ldots 14)$$

will result in the best representation of tri-linear relation in Fig. 3.

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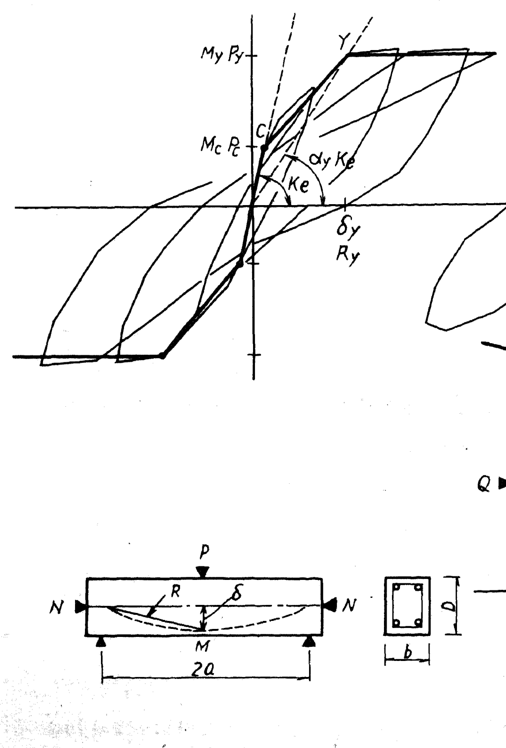

Fig. 1. Beams and Columns

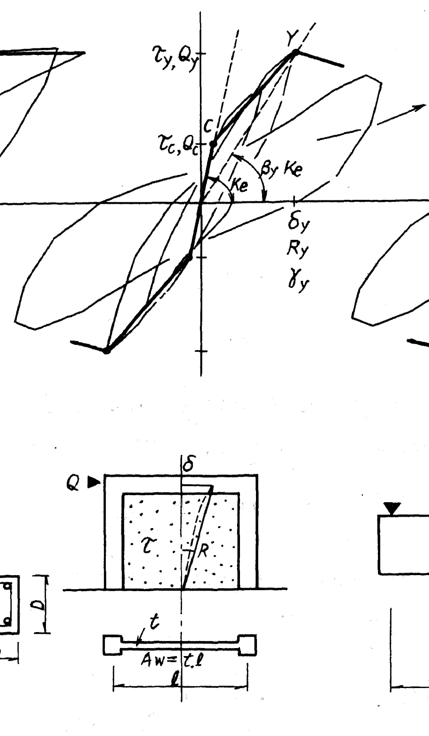

Fig. 2. Shear Walls

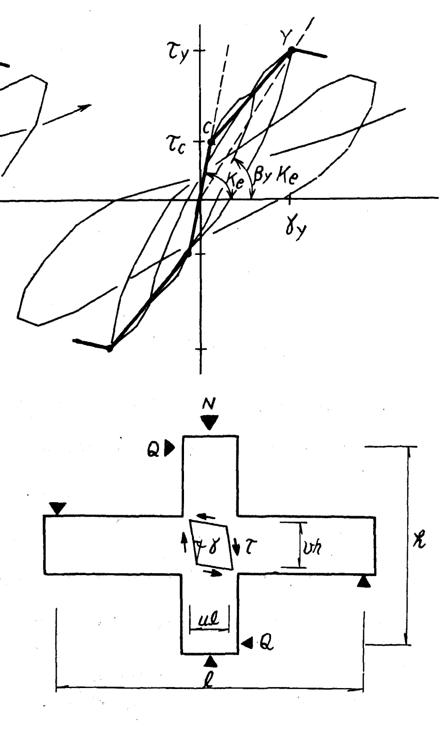

Fig. 3. Beam-Column Connections