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HYSTERESIS CHARACTERISTICS OF HIGH STRENGTH REINFORCED CONCRETE BEAMS

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SUMMARY

This paper studies the nonlinear load-deformation relation of RC beams using high strength materials. Published test results of RC beams under bending and shear were carefully examined. Those exhibiting flexural yielding before shear failure and bond-splitting failure were selected and the load-deformation relation were collected. In order to select a single yield point, an iterative procedure was used to idealize the observed relation into a tri-linear relation. To evaluate cracking point, yield point and ultimate moment, various methods were applied. As a result, (1) evaluated initial stiffness overestimates the test results, (2) a large variation was observed in evaluated cracking moments, (3) yielding and ultimate moments can be reliably evaluated by the flexural theory, and (4) a formula, which is based on the theoretical model, can provide a reasonable estimate of yielding deformation.

INTRODUCTION

Recently, many high-rise reinforced concrete (RC) buildings have been constructed using high-strength materials. As different characteristics were anticipated for high-strength RC members, the Ministry of Construction, Japanese Government, organized a national project for the “Development of Advanced Reinforced Concrete Buildings Using High-strength Concrete and Reinforcement (New RC Project)” from 1988 to 1993. The material strength ranged from 30 to 120 MPa for the concrete and from 400 to 1200 MPa for the longitudinal reinforcement. High-strength lateral reinforcement of yield stress ranging from 600 to 1300 MPa is commonly used in Japan. Methods to evaluate the force-deformation relationship of RC beams were examined in the project. The authors examined the hysteresis characteristics of RC beams for use in a nonlinear earthquake response analysis (Ref.1). Recently, short span beams were tested in laboratories to examine the behavior of beams in tube-type buildings. Furthermore, the performance-based design and engineering attracts attention in earthquake engineering community, and a more reliable evaluation of the force-deformation relation becomes necessary. This paper studies member end moment-rotation relations of RC beams using high-strength materials.

DATABASE OF RC BEAM TESTS

Test data of beams, which exhibited flexural yielding before shear and bond-splitting failure, were searched from literature (Ref. 2-30). These papers were published between 1982 and 1998 in Proceedings of Japan Concrete Institute (JCI), Summaries of Technical Papers of Annual Meeting of Architectural Institute of Japan (AIJ) and reports of research institutes of Japanese construction companies. The specimens must satisfy the following conditions; (a) rectangular cross section, (b) same amount of top and bottom longitudinal reinforcement, (c) width wider than 150mm, and (d) overall depth deeper than 225mm. Among 146 beam specimens studied, concrete strength ranged from 20 to 135 MPa; shear span-to-depth ratio from 0.88 to 3.55; tensile reinforcement ratio from 0.36 to 3.10 percent; yield strength of longitudinal reinforcement from 261 to 976 MPa. The load-

deformation relation curves under reversed cyclic loading were digitized for a sequence of parts where the load exceeded maximum load in preceding loading cycles. The error of digitization is less than 0.3 percent, on the average 0.2 percent, of the maximum load and displacement.

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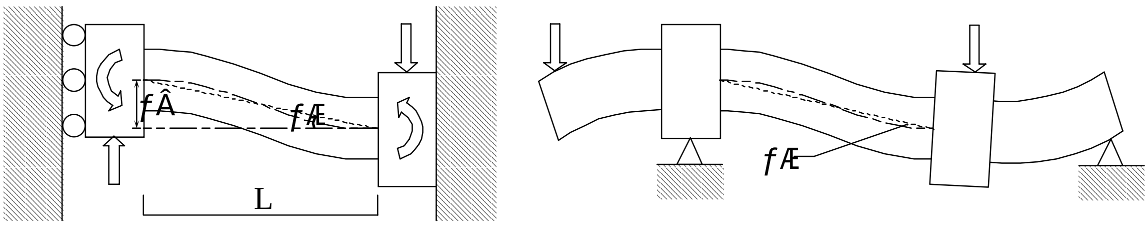
The specimens were tested under two types of loading methods (Fig.1) simulating the conditions during earthquake excitation. Type-A test is statically indeterminate beams with two stiff end stubs subjected to lateral displacement at the two ends maintaining the end stubs in parallel during loading. Type-B test is simply supported beams, normally with two loading stubs, subjected to point loading causing the point of inflection at the canter of a middle span. In Type-A test, moment distribution is not known due to statically indeterminacy; linear moment distribution is assumed with an inflection point at the mid-span. In Type-B test, the damage within the test span tends to concentrate at an end during the test. Simply supported specimens subjected to mid-span loading were not selected for the study.



(a) Type-A test (b) Type-B test

FIG. 1 LOADING METHODS IN LABORATORY

IDEALIZATION OF MOMENT-ROTATION RELATION

Although many specimens failed in shear or bond-splitting modes after flexural yielding, the initial stiffness may not be affected by the failure modes. Therefore, all 146 specimens were used in study of initial stiffness and flexural cracking moment. On the other hand, the yield deflection is increased by the damage associated with failure modes. Therefore, those specimens failing in shear or bond-splitting modes within a deflection equal to two times flexural yielding deflection were excluded from the examination of yield and ultimate points; 101 specimens were used to study the yielding deflection.

The initial stiffness in a test was defined as a secant slope at a load equal to one-half of the reported cracking load. If the cracking load was not reported, the cracking moment was calculated by assuming the tensile strength of concrete to be $0.56\sqrt{\sigma_B}$ (Ref. 31), where σ_B is concrete strength in MPa. Note that the cracking load is normally reported at a loading step when cracking is detected for the first time. As the reported cracking load is normally higher than the actual cracking load, the cracking point was determined from the shape of force-deformation relation by the method described in the following paragraph.

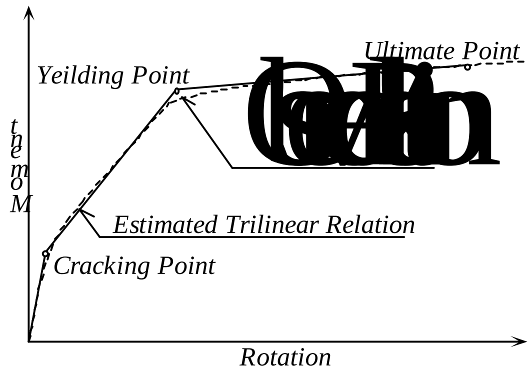


Fig.2 Idealization of observed moment-rotation relation

The stiffness of reinforced concrete section changes drastically at the yielding of tensile reinforcement. If tensile reinforcement is placed in double layers in a section, the stiffness changes at the yielding of the outer layer reinforcement and then of the inner layer reinforcement. In order to select a single yield point, the yield point and cracking moment were defined such that the energy stored at the ultimate deformation should be the same for the test and the model making the absolute difference in the energy to be minimum (Fig.2). Resistance at the ultimate point was taken as the observed maximum resistance. The determination of an ultimate deformation is an important but difficult issue; the ultimate deformation is not a unique value but is highly dependent on the progress of concrete deterioration dictated by loading history and failure modes. Small stiffness after yielding will not change appreciably by the choice of an ultimate deformation. Therefore, the ultimate deformation was selected to be an arbitrary deformation at a deformation ductility factor of four. An iterative procedure was used to define the yield point and cracking moment for the established initial stiffness and ultimate point.; i.e., (a) a trial yield displacement was assumed, (b) an ultimate displacement was selected at four times yield deformation, (c) post-yield stiffness was determined by connecting the ultimate point and a point on the observed curve at 2.5

times yield deformation, and (d) the cracking moment and yield deformation were determined for equal absorbed energy at the ultimate deformation and minimum absolute difference.

ESTIMATION OF HYSTERESIS CHARACTERISTICS

Initial Stiffness

The initial elastic stiffness K_e was evaluated by the elastic theory of a lineal prismatic member considering flexural and shear deformation;

$$\frac{1}{K_e} = \frac{1}{K_f} + \frac{1}{K_s} \qquad (1)$$

where K_f: flexural stiffness ($= 6E_cI_e / L$), E_c: elastic modulus of concrete, I_e: moment of inertia of uncracked transformed section, L: member length, K_s: shear stiffness ($G_cAL / 2\kappa$), G_c: shear modulus of concrete ($=E_c/\{2(1+\nu)\}$), A: cross sectional area, κ: shape factor for shear deformation ($=1.2$), ν: Poisson's ratio of 0.20. Elastic modulus E_s of steel was assumed to be 206GPa. The initial stiffness was calculated using the observed elastic modulus of concrete and the clear span. If elastic modulus of concrete was not reported, the modulus was determined by Eq.2 (Ref.32) with $k_1=k_2=1.0$ and $\gamma = 2.4$.

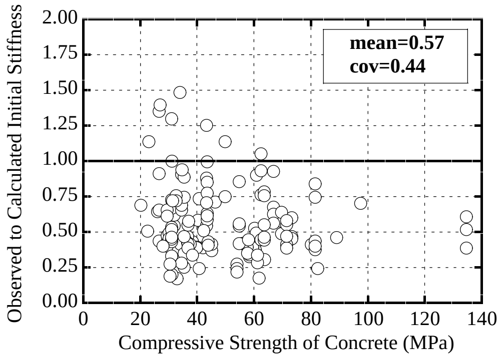


Fig.3 Reliability of initial stiffness

$$E_c = k_1 \times k_2 \times 3.35 \times 10^4 \times (\sigma_B / 60)^{1/3} \times (\gamma / 2.4)^2 \qquad (2)$$

in which k_1: factor representing type of coarse aggregates, k_2: factor representing kind of mineral admixture, σ_B : observed compressive strength of concrete(MPa), γ : unit density of concrete (ton/m^3).

Reliability of calculated initial stiffness is shown in Fig.3. The average ratio of the observed to the calculated initial stiffness was 0.57 with a coefficient (cov) of variation of 0.44 for all 146 specimens. The observed initial stiffness was notably low and the coefficient of variation was large. The discrepancy was probably attributed to (a) technical difficulty in measuring accurate initial stiffness in the test and (b) formation of accidental and shrinkage cracks prior to the test. In a real structure, flexural cracks under gravity loading, shrinkage cracks, cracks after medium intensity earthquake excitation may exist, and the initial stiffness for the analysis is difficult to estimate.

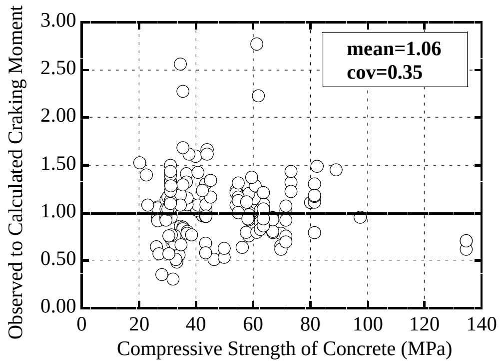


Fig. 4 Observed to calculated cracking moment

Cracking Moment

Cracking moment M_{cr} was calculated on the basis of the observed splitting tensile strength σ_{cr} of concrete and the section modulus Z_e of the uncracked transformed section and given by

$$M_{cr} = \sigma_{cr} \cdot Z_e \qquad (3)$$

$$\sigma_{cr} = 3.74\sigma_B^{0.205} \qquad (4)$$

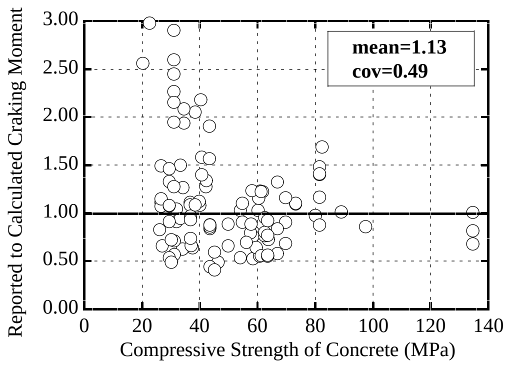


Fig. 5 Reported to calculated cracking moments

in which σ_B : compressive strength of concrete (MPa). Cracking tensile strength σ_{cr} of concrete was determined by dividing the cracking moment of the trilinear idealization of the moment-rotation relation. The ratio of the observed to the calculated cracking moment is compared in Fig.4. The average ratio was good at 1.06 with a significantly large coefficient of variation of 0.35. On the other hand, the ratio of reported to calculated cracking moments, evaluated by assuming the tensile strength of concrete to be $0.56\sqrt{\sigma_B}$ (Ref. 31), is compared in Fig.5. Note the difference between observed and reported cracking moments. The average ratio was 1.13 and the coefficient of variation was 0.49.

Yield Moment

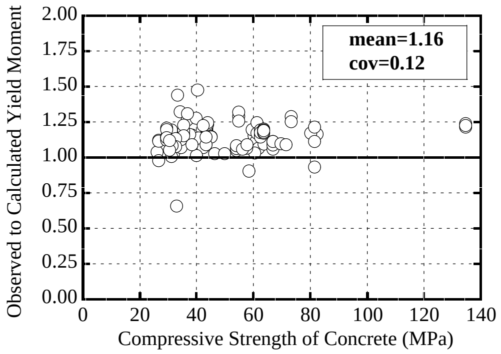
To investigate the yield moment, two evaluation methods were applied in this study. A method was an approximate equation proposed by Sugano (Ref.31);

$$M_y = \{p_t \sigma_y g_1\} \cdot B \cdot D^2 \quad (5)$$

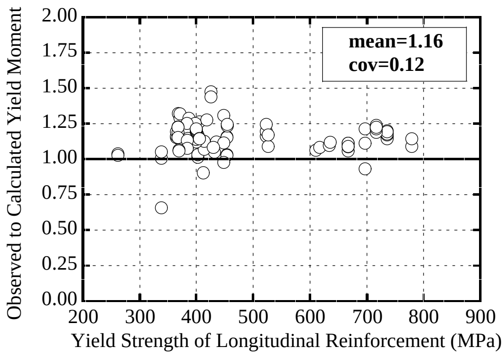
in which B: width of section, D: depth of section, g_1: distance from the centroid of tensile reinforcement to the centroid of compressive reinforcement, p_t: ratio of area of tensile reinforcement to area of section, σ_y :yield stress of tensile reinforcement. The other method was an analytical method using the fiber model, in which the yield moment at the critical section was calculated for yielding at an imaginary centroid of tensile reinforcement. The amount of tensile reinforcement is normally limited well below the balanced tensile reinforcement ratio. The stress-strain relation of concrete was a confined-concrete model proposed by Sakino (Ref.33) and that of main bar was an elasto-plastic model. In this study, the confining effect was ignored. In addition, the following assumptions were made in calculating yield moment, (a) plane section remained plane after deformation, and (b) the concrete in tension did not resist tensile stresses. The stress-strain curve was given by

$$\frac{\sigma}{\sigma_B} = \frac{AX + (D-1)X^2}{1 + (A-2)X + DX^2} \quad (6)$$

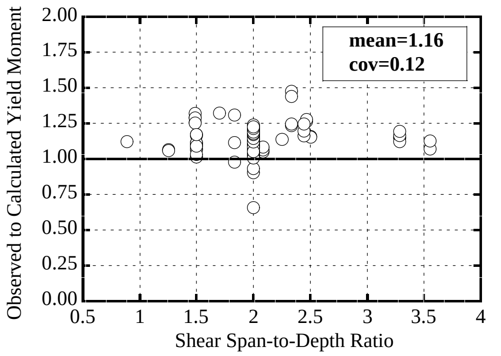
in which σ :stress of concrete (MPa), $A = E_c \cdot \varepsilon_0 / \sigma_B$, $X = \varepsilon / \varepsilon_0$, $D = 1.5 - 1.71 \times 10^{-3} \sigma_B$, ε :strain of concrete, ε_0: strain at compressive strength of concrete ($\varepsilon_0 = 0.93 \sigma_B^{1/4} \times 10^{-3}$), E_c: elastic modulus of concrete (Eq. 2), k : factor representing type of coarse aggregates, γ : unit density of concrete (ton/m^3), respectively. In both evaluation methods, the compressive strength of concrete and the yield stress of reinforcement were obtained from the reported material tests. In all 101 specimens, the calculated stress at the extreme compressive fiber did not exceed the compressive strength of concrete.



(a) Effect of Compressive strength of concrete



(b) Effect of yield strength of longitudinal bars



(c) Effect of shear span-depth ratio

Fig.6 Reliability of Yield Moment

Figure 6 shows a relation between the observed yield moment and the yield moment calculated by Eq. 5. The ratio of the observed to the calculated yield moments is compared with respect to the compressive strength of concrete in Fig. 6(a), to tensile strength of the yield stress of reinforcement in Fig.6 (b), and to the shear span-to-depth ratio in Fig. 6(c), respectively. The average ratio of the estimated to the calculated yield moments was 1.16 with a coefficient of variation of 0.12 for Eq. 5. For the fiber analysis, the average ratio was 1.14 with a coefficient of 0.13. General tendency with respect to each factors was similar to Fig. 6. The effect of the compressive strength of concrete, the yield stress of reinforcement and the shear span-depth ratio was not indicated in both methods. In only four specimens the estimated yield moments were smaller than the calculated values. The yield moment at which the stiffness of an RC member changes drastically may be calculated conservatively by the two evaluation methods. Because there is hardly much difference in the two evaluation methods, the yield moment of beam is not sensitive to the shape of stress-strain relationship nor compressive strength of concrete. As the neutral axis depth is so small, the distance between the resultant compressive and tensile forces cannot change appreciably within the section.

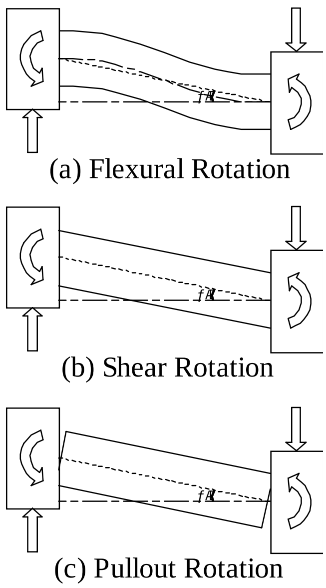


Fig.7 Concept of Rotation

Yield Rotation

Member end rotation at flexural yielding has been estimated by empirical stiffness degrading ratio α_y of secant stiffness at yielding to the initial stiffness. This empirical equation was proposed by Sugano (Ref.31);

$$R_y = \alpha_y \cdot K_f \cdot M_y \quad (7)$$

$$\alpha_y = \left(0.043 + 1.64n \cdot p_t + 0.043a / D + 0.33\eta\right) \cdot \left(d / D\right)^2 \quad (8)$$

in which M_y: yield moment calculated by (Eq. 5), K_f: initial flexural stiffness ignoring shear deformation, n : modular ratio of reinforcement to concrete, p_t: tensile reinforcement ratio, a: shear span of a beam, B: width of section, D: depth of section, η : the axial force ratio of 0.0, d: effective depth.

Recently, the theoretical equation estimating yield rotation was proposed. Yield rotation was defined as the sum of flexural rotation R_f , pullout rotation R_p of longitudinal reinforcement from the anchorage, and rotation R_s associated with shear deformation (Fig. 7). The authors proposed an empirical equation for yield rotation of RC beams and columns using high-strength materials in 1994. Shen and Kabeyasawa proposed a theoretical method for yield rotation of columns using high-strength materials in 1994 (Ref. 34). In the model, the shear rotation based on a truss mechanism and an arch mechanism of shear resistance mechanism was proposed. The expressions were given by

$$R_y = R_f + R_s + R_p \quad (9)$$

$$R_f = Q_y \cdot L^2 / 12E_c \cdot I_{cr} = \phi_y \cdot L / 6 \quad (9a)$$

$$R_s = Q_y \cdot \xi / \left\{E_c \cdot B \cdot j_t \cdot \left(x_n / j_t + \xi / \mu\right)\right\} \quad (9b)$$

$$R_p = 27.21\varepsilon_y \cdot d_b \cdot \sqrt[3]{E_s \cdot \varepsilon_y^2 / \sigma_B} / \left\{2 \cdot \left(d - x_n\right)\right\} \quad (9c)$$

in which Q_y: shear force at flexural yielding, L: clear span, ϕ_y : yield curvature considering confined effect of concrete, E_c: elastic modulus of concrete, I_{cr}: moment of inertia of uncracked transformed section, B: width of section, $\xi = 1 + \cot\phi$, $\mu = 4 + 1/\left(n \cdot p_w\right)$, j_t: distance between the centroid of compressive and tensile reinforcement, x_n: distance from neutral axis to the compressive extreme, ε_y :strain of longitudinal bar at yield stress, d_b: diameter of longitudinal bar, E_s: elastic modulus of reinforcement, σ_B : compressive strength of concrete (MPa), d: effective depth of section, respectively.

Figure 8 shows the reliability of Eq. 7. The average ratio (observed/calculated) was 1.52 with a coefficient of 0.22 for Eq. 7. The average ratio was 1.32 with a coefficient of 0.20 for Eq. 9. The yield rotation calculated by Eq. 7 overestimates the observed deformation. The error appears to be larger for higher strength concrete and reinforcement and for smaller span-depth ratio. Equation 7 was derived for RC member test data using normal strength materials and for shear span-to-depth ratio ranged from 2 to 5. Figure 9 shows the reliability of Eq. 9 in estimating yield rotation. An influence by the concrete strength and the strength of the longitudinal reinforcement isn't seen in that ratio. The calculated deformation overestimate the observed for smaller shear span-depth ratio. The effect of shear is larger in a member with small shear span-depth ratio. Additional deformation associated with shear cracking must be considered in estimating yield deformation.

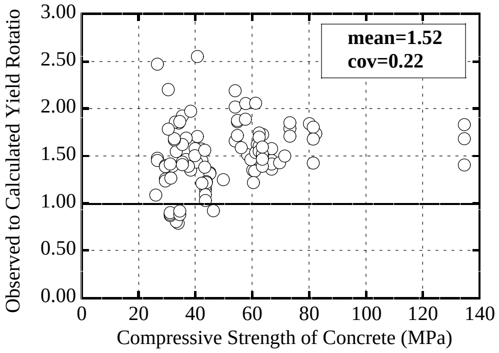
Shen (Ref. 34) proposed a method to estimate the yield deformation for column members using high strength materials. On the other hand, shear crack occurs before flexural yielding of beams. The compressive strength of cracked concrete is generally decreased by shear cracks. It is suggested in Ref. 35 to use effective compressive strength of concrete $\nu\sigma_B$ in estimating shear strength of reinforced concrete members before flexure yielding in order to consider the decay in compressive strength caused by inclined cracking. In this study, effective elastic modulus *Ec'* of concrete was used to consider the effect of cracking on shear deformation;

$$E_c' = \nu\sigma_B / \varepsilon_0 \tag{10}$$

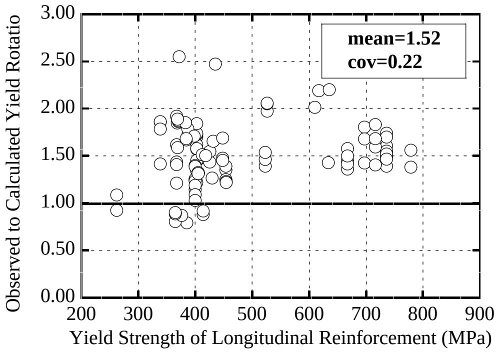
in which $\nu\sigma_B = 1.7\sigma_B^{0.667}$, σ_B :compressive strength of concrete (MPa), $\varepsilon_0 = 0.93\sigma_B^{1/4} \times 10^{-3}$. The effective modulus *Ec'* is used in Eq. 9b. The yielding curvature in Eq. 9a was calculated by the fiber analysis at the critical section. Figure10 shows the reliability of yield rotations calculated by Eq. 9 using the effective modulus and the yield curvature from the fiber analysis. The average ratio (observed / calculated) was improved to 1.08 with a coefficient of 0.22. The reliability is not influenced by the strength of concrete and reinforcement, nor by the shear span-to-depth ratio.

Ultimate Moment

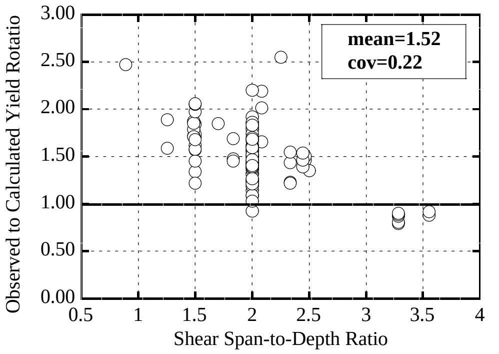
The ultimate moment was calculated by the analysis using stress block indicated ACI 318 (Ref.37), in which the flexural mechanism was assumed to form by the yielding of tensile reinforcement followed by the compressive failure of concrete.



(a) Effect of compressive strength of concrete

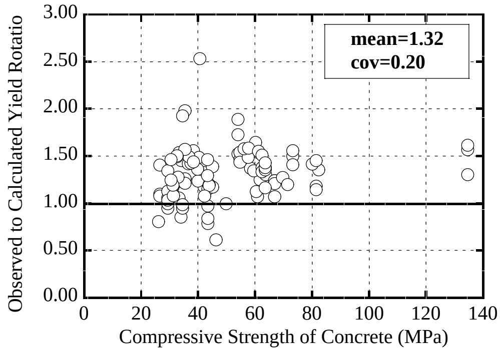


(b) Effect of yield strength of longitudinal bar

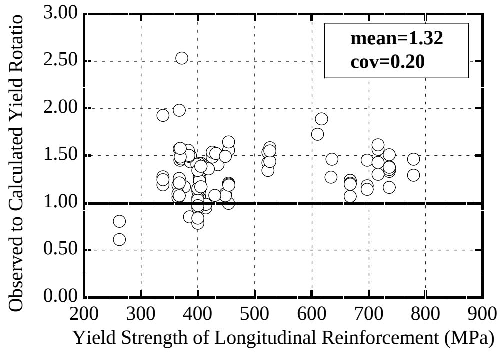


(c) Effect of shear span-to-depth ratio

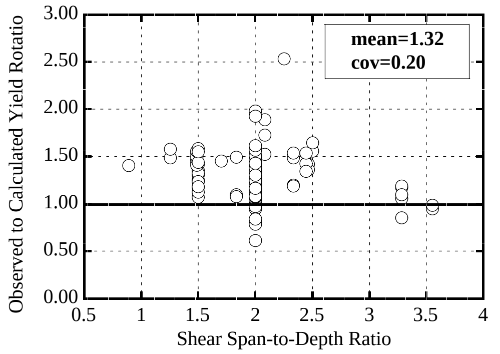
Fig.8 Reliability of Yield Rotation (Eq.7)



(a) Effect of compressive strength of concrete

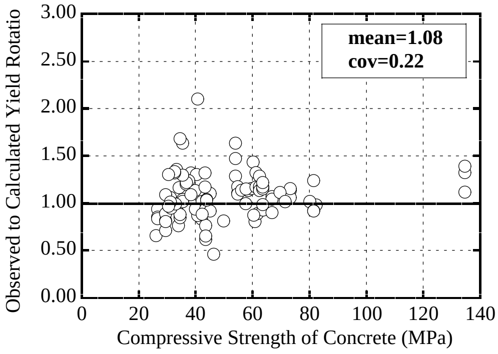


(b) Effect of yield strength of longitudinal bar

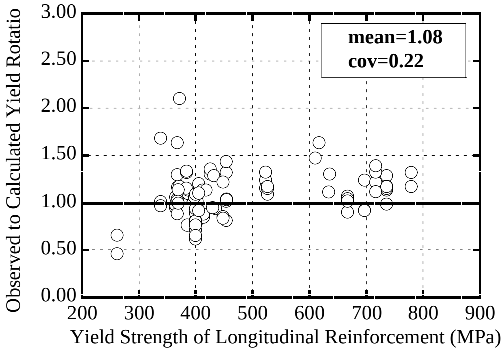


(c) Effect of shear span-to-depth ratio

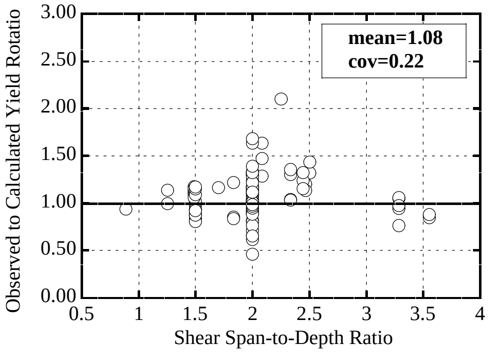
Fig.9 Reliability of Yield Rotation (Eq.9)



(a) Effect of compressive strength of concrete



(b) Effect of yield strength of longitudinal bar



(c) Effect of shear span-to-depth ratio

Fig.10 Reliability of Yield Rotation by Proposed Method

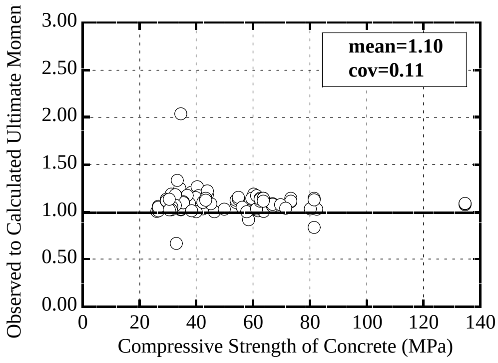


Fig.11 Reliability of Ultimate Moment

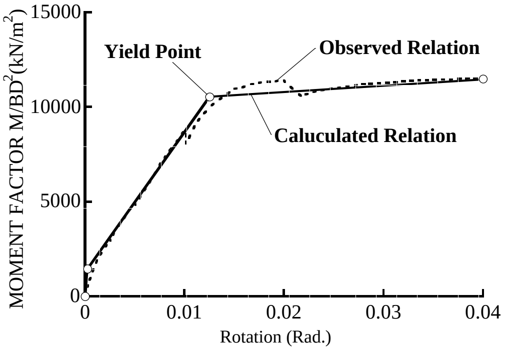


Fig.12 Application Example of Moment-Rotation Relation for Specimen SB6

The ratio of the observed to the calculated ultimate moments is shown in Fig. 11 with respect to compressive strength of concrete for 101 specimens. The average ratio is 1.10 with a coefficient of variation of 0.11.

APPLICATION EXAMPLE

Figure 12 compares an observed envelope curve with a calculated trilinear relation for specimen SB6 tested by Nagai (Ref. 27). The ultimate rotation is assumed to be 0.04. The specimen has the following characteristics; 250mm width, 400mm depth, shear span-to-depth ratio of 1.5, tensile reinforcement ratio p_t of 3.1 percent, concrete strength of 58 MPa and reinforcement yielding stress of 527 MPa. Although the calculated initial stiffness overestimated the observed stiffness, the calculated yield moment, ultimate moment and yield rotation represent the corresponding observed values.

CONCLUSION

Force-deformation relationship of reinforced concrete beams using high strength materials was idealized by a trilinear relation. Methods to evaluate the characteristic points were proposed. The reliability of the proposed method was examined with respect to the observed test data.

This paper presents the following concluding remarks.

(1) Calculated initial stiffness is shown to significantly overestimate the observed value.
(2) A large coefficient of variation was observed in the evaluation of cracking moment.
(3) Yielding and ultimate moments could be reliably estimated by the flexural theory.
(4) An evaluation method based on the theory was developed to evaluate yield deformation.

ACKNOWLEDGMENT

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