Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

AIJ seismic design guidelines for RC buildings – Design of beam-column joints

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ABSTRACT: The Architectural Institute of Japan published "Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings based on Ultimate Strength Concept (draft)" in 1988 as a first attempt to develop an ultimate strength design procedure in Japan. The paper introduces the design requirement and background information for reinforced concrete beam-column joints adopted in the Design Guidelines. Based on experimental analyses, the shear strength of joint is evaluated using the product of effective joint shear area and factored compressive strength of concrete, and the amount of lateral reinforcement in the joint is required in order to keep the stiffness of joint after yielding of beam bars.

1 INTRODUCTION

Standard for Structural Calculation of Reinforced Concrete Structures issued by Architectural Institute of Japan (AIJ) consists of two design stages: the allowable stress design for considerable loads and the check of lateral load carrying capacity on the severest seismic forces. In this Standard, structural design for beam-column joints is not required at all, as one of reasons is that significant earthquake damage has not observed so many in reinforced concrete joints in Japan. However, the damage in joints may have hidden behind architectural coverage, or premature column failure may have reduced the action in the joints.

Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concept which was published as a draft from AIJ in 1988 and was formally approved in 1990 requires the structural design for beam-column joints.

The AIJ guidelines allow a building to be damaged by a severe earthquake to such a certain degree as the building structure does not perform over plastic deformations, brittle failures, and concentration of destruction in a limited part. In order to satisfy these design demand, the structure should be designed in two stages.

'Mechanism Design' of the first stage is a planning of desirable yield-hinge locations based on beam yielding mechanism and a design of sections of the yield hinge regions. Design bending moment can be calculated by a linear stress analysis using reduced member stiffness and modified by moment redistribution. Reliable strength, which is lower bound strength of a section or member calculated using specified material strength, shall be provided at the yield hinges.

'Assuring Design' of the second stage is a design of non-yielding regions to assure the performance of planned yield-hinge locations. The reliable strength shall be provided at the non-yielding regions for the design action determined by a nonlinear plastic analysis at the planned yield-hinge formation. Upper bound strength, which is flexural strength evaluated in consideration of all possible factors increasing the strength, shall be used at the yield hinges under remagnification with factors which account for dynamic effect and bi-directional response.

2 DESIGN CRITERIA OF BEAM-COLUMN JOINTS

A failure of beam-column joint in a structure prior to the formation of a yield mechanism should be avoided because it is difficult to design the joint for high ductility and also difficult to repair the damage. The diagonal compression strut of concrete is assumed as the shear resisting mechanism, and nominal shear stress in the joint is limited to prevent shear failure.

The beam-column joint must resist bending moment, shear, axial force, and torsional moment produced by columns and beams at a yield hinge mechanism without evident stiffness degradation. Shear cracking of joint concrete and bond deterioration after yielding of flexural reinforcement, however, can not be avoided in the joint, then the load vs. deformation characteristics with slippage are acceptable for the AIJ guidelines when those do not affect so much on an earthquake response. A recent study observed that the slipping hysteretic characteristics did not increase structural response in the displacement range considered in the AIJ guidelines; i.e., an story drift angle of 1/100 radian.

Yielding of lateral reinforcement in a joint and significant bond deterioration along the beam reinforcement within the joint result in a reduction in stiffness and energy dissipation capacity with increased deflection. The AIJ guidelines supposes the maximum story drift reaches 1/66 radian for a moment resisting frame and 1/75 radian for a frame-wall dual structure in the Assuring Design.

Therefore, the AIJ guidelines require that beam-column joints shall be designed to maintain its integrity to the assuring deformation after the structure forms a yield mechanism and to prevent a significant stiffness reduction and slip-type hysteretic behavior by load reversals. However, the design criteria for joints specified in beam-column joints is not so severe than that for beams, columns and shear walls because severe design requirements would disconnect the continuity of the design for joints between the previous design method and the AIJ guidelines, this continuity is important in Japan and make the scope of the AIJ guidelines narrow.

3 DESIGN FOR SHEAR

3.1 Deign principles

The AIJ guidelines require that reliable shear strength V_{ju} shall be greater than design shear V_j used in the Assuring Design.

Beam-column joints should be designed against the most severe stress combination of bending moment and shear at the yield mechanism assuring design. Since the location of yield hinges have been determined at this design stage, the upper bound strength of the yield hinges and the compatible stress of the non-hinge members should be used for the design stresses of the joints.

3.2 Design Shear

Design shear, V_j, for the beam-column joint in a structure with beam hinges should be calculated by Eq. 1.

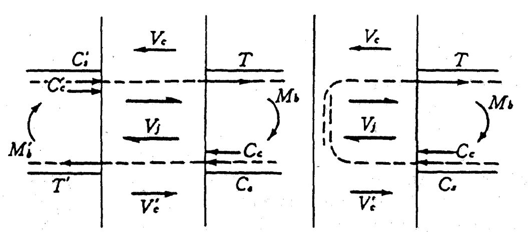

Fig.1 : Internal stress resultant of joint

$$V_j = T + C_s' + C_c' - V_c = T + T' - V_c \quad \text{(Eq. 1)}$$

where the symbols are explained in Fig. 1. In exterior beam-column joint, $C_s'+C_c'=T'=0$ is assumed. Column shear , V_c, may be taken as an average of the upper and lower column shears. Tension forces in the reinforcement, T and T', should be calculated by using the upper bound flexural strength of beam hinges, evaluating the effects of (1)actual strength increasing from nominal strength of reinforcement and (2)reinforcement in floor slab whose effective width extends with curvature of the beam hinges.

3.3 Reliable Shear Strength

3.3.1 Shear resisting mechanism

T.Paulay et al.(1978) proposed the shear resisting mechanism in beam-column joint consisted of a diagonal concrete strut action and a truss action as shown in Fig. 2. The diagonal concrete strut action is the stress transferring mechanism formed by a inclined compression strut of concrete diagonally connecting a pair of compression zones at the corners of joint produced by compression and shear of beams and columns. The truss action is the stress transferring mechanism formed by lateral and vertical joint reinforcement and by uniform diagonal compression struts of joint concrete, and the stress transfer between the reinforcement and the struts depends on the bond stress of beam and column longitudinal reinforcement within the joint.

As mentioned after, previous test data show that the joint shear strength depends on the concrete compressive strength remarkably, but the amount of lateral reinforcement in joints does not effect the shear strength.

Therefore, on the assumption that the diagonal strut action is a dominant shear resisting mechanism in the joint and the reliable shear strength V_{ju} of a joint is proportional to the compressive concrete strength f'_c, the AIJ guidelines require that the shear strength of joints should be evaluated by multiplying the factored concrete strength dependent on the shape of beam-column connection by the effective shear resisting area, given by Eq. 2.

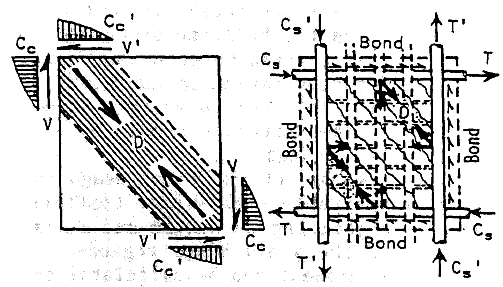

Fig. 2 : Idealized shear resisting mechanism

where, k: factor dependent on shape of a beam-column connection; i.e., 0.30 for an interior beam-column connection, and 0.18 for an exterior beam-column connection, b_j: effective joint width and D_j: effective joint depth.

3.3.2 Effective Joint Area

Average joint shear stress, v_{ju}, is evaluated using the effective joint shear area $b_j \times D_j$, in which b_j is effective joint width and D_j is effective joint depth.

(a) Effective joint width b_j

It need to be considered when the width of beam is very narrow against the width of column, or the beam connects eccentrically to the column, some part of column area apart from the beam in beam-column joint does not contribute to shear strength. The AIJ guidelines require the effective joint width should be evaluated by Eq. 3.

$$b_j = b_b + b_{a1} + b_{a2} \qquad \text{(Eq. 3)}$$

where, b_b: beam width, b_{a1}, b_{a2}: the smaller of one-quarter of column depth (e.g., shaded area on right side in Fig. 3) and one-half of distance between beam and column faces (e.g., shaded area on left side in Fig. 3) on either side of the beam.

The AIJ guidelines recommend that a beam-column joint is designed to have beam width smaller than column width because (1)the maximum eccentricity between beams and columns can be limited to a half of column width and (2)all beam bars resulted to be passed in the column core.

When beams are connected to columns eccentrically, torsional moments acting on the beam-column joint and on the columns make torsional cracks in them, and reduce their stiffnesses. An large eccentricity of beams to columns reduces a force transferred to a potion away from the beams in the beam-column joint, and consequently shear capacity of the joint decreases due to the reduction of equivalent concrete volume resisting against shear force.

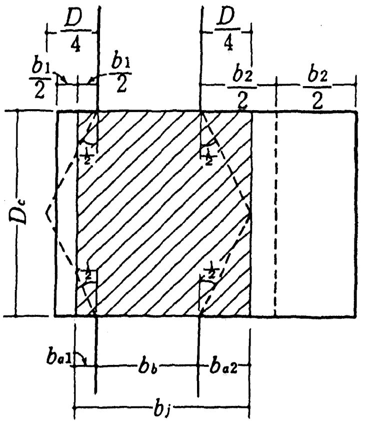

Fig. 3 : Effective width of beam-column joint

(b) Effective joint depth D_j

It is reasonable to take the effective depth for interior beam-column joint as the entire column depth because the dominant shear resisting mechanism is assumed to be the diagonal strut action in the AIJ guidelines. However, the horizontally projected length of a 90 degree hook or the horizontal length from the end of beam to the end of steel anchor plate in case of using mechanical embedment by steel anchor plate is used as the effective depth for exterior beam-column joint because the compression strut is assumed to start at the corner of the bar bend or the plate.

3.3.3 Shear Strength

Shear strengths v_{ju} resulted from sixty-eight interior joint specimens which have a cross shape without transverse beams nor slabs, tested from 1966 to 1988 in Japan and other countries, were compared with actual concrete strength s_B in Fig. 4.

Twenty-four test data showed shear failure of joint prior to the yielding of beams or columns (denoted as J-type, solid triangle symbols). The average ratio of v_{ju} to s_B is 0.345 and its deviation is 0.051. When the factored concrete strength as the lower bound shear strength is assumed to be $0.3 s_B$, percentage of specimens having shear strength larger than the lower bound shear strength is 80% in concrete compressive strength range from about 20 to 35 MPa.

Forty-four test data showed shear failure of joint after the yielding of beams (denoted as BJ-type, open circle symbols). The maximum shear stress of BJ-type dominated by the flexural strength of beams are lower than v_{ju} of J-type and shear failure occurred at the story drift angle greater than 1/50 radian beyond the Assuring Deformation. This means that shear failure up to the drift angle of 1/50 could be avoided by keeping the shear stress less than the lower bound shear strength.

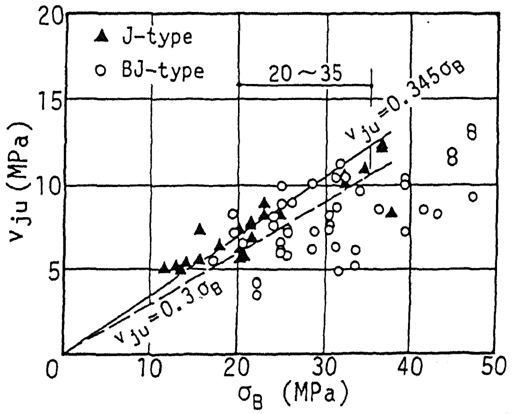

Fig. 4 : Average shear stress-concrete strength (interior joint)

The relations of shear strength and concrete compressive strength obtained from exterior beam-column joints which have a ⊢ shape without transverse beams nor slabs are summarized in fig. 5. There are only five specimens of J-type and seven specimens of BJ-type. The average ratio of v_{ju} to σ_B is 0.194 and its deviation is 0.013. The lower bound shear strength with excess probability of 80% is $0.18\sigma_B$ in J-type specimens.

The assumption in Eq. 2 as the shear strength is proportional to compressive strength of concrete is for practical use, and it should be noticed that the upper bound of the specified design compressive strength of concrete is 35MPa. That is the coefficient of k should not be used in the range of concrete compressive strength larger than 35 MPa.

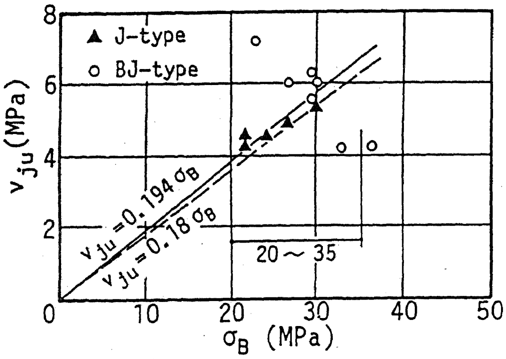

Fig. 5 : Average shear stress-concrete strength (exterior joint)

3.3.4 Effect of Transverse Beams

The effect of unloaded transverse beams on increase of the shear strength and stiffness of joint is well known. M.Ohwada(1981) revealed the relationship between a cover ratio L defined by Eq. 4 and a strength ratio B of the joint with transverse beam(s) to the joint without them. The solid parabolic line could follow its relationship as shown in Fig. 6.

In an existing building, however, a beam-column joint can not be confined so much by the transverse beams, because the joint has yield hinges at all faces subjected to an bi-directional earthquake motion. Consequently, the effect of transverse beams on strengthening may be reduced. However, the AIJ guidelines recommend that only when the transverse beams are assured not to yield at the column faces, the shear strength could be increased by the factor β expressed by Eq. 4.

$$\beta = 1 + 0.3\,\lambda \qquad \text{(Eq. 4)}$$

$$\lambda = \Sigma\,(\,b_{Lb}\cdot D_{Lb}\,)\ /\ 2(\,D_b\cdot D\,)$$

where b_{Lb} : width of transverse beam, D : depth of column, D_b : depth of longitudinal beam, D_{Lb} : transverse beam, but limited by the depth of longitudinal beam.

For a joint with a transverse beam on only one side, the strength increment should not be considered. The equation above gives the lower bound prediction to scattered test data as shown in Fig. 6.

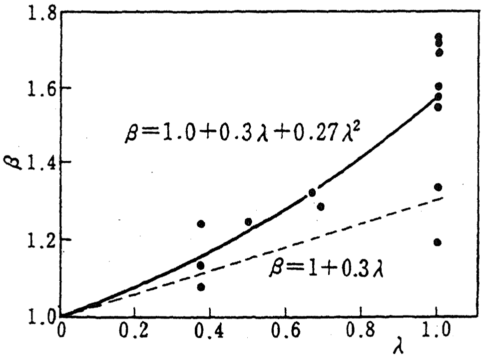

Fig. 6 : Shear strength magnification by transverse beams

3.3.5 Bi-directional Performance

K.Minami et al.(1988) reported the test results on three dimensional interior beam-column joints failed in joint shear suffering from the two directional load reversals as shown in Fig. 7. The shear strengths in inclined directions were larger than that in the main direction (beam axial direction) under zero axial load, but were not larger under an axial stress of 0.2 times the concrete strength.

Loading type	axial stress ratio σ_N/σ_B	
	0	0.2
Two-dimensional	□ ■	◇ ◆
Three-dimensional	○ ●	△ ▲

Open mark: forward loading
Solid mark: backward loading
—·— square lines and a circular arc interpolated between shear strength in two main directions(0-degree) calculated by Eqs. 2 and 3

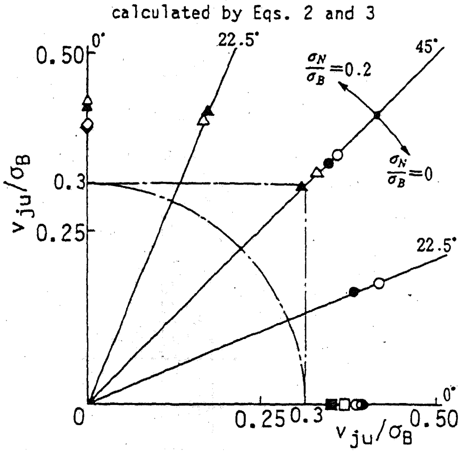

Fig. 7 : Bi-directional Shear Interaction

In three-dimensional beam-column joint tests carried out by T.Kusakari et al.(1984) in which the joint failed in shear after developing flexural yielding at beam ends, a specimen subjected to alternate longitudinal and transverse load reversals was similar to a specimen subjected to diagonal load reversals in the resistance and deformation capacity.

Therefore, the AIJ guidelines suggest that a joint might be designed for the two principal directions independently, although it is desirable to design the joint to resist simultaneous bi-directional earthquake loading conditions, in which the joint input shear becomes larger than that under uni-directional earthquake loading.

3.4 Joint Shear Reinforcement

3.4.1 Lateral Reinforcement

Fig. 8 shows the relationship between the strength ratio of shear strength to concrete compressive strength v_{ju}/σ_B and the product of lateral reinforcement ratio and its yield strength, $p_{jh} \cdot \sigma_y$ for J-type internal joint specimens. Points linked by solid lines mean the specimens which were tested as a series by the same researchers and consisted of same details except only the value of $p_{jh} \cdot \sigma_y$. A few results shows an increase in shear strength with the amount of lateral reinforcement, but a majority of test results does not show it. The roles of lateral reinforcement in a joint can be recognized (1)to avoid diagonal tension failure of the joint, (2)to improve the ductility of the joint by confining the cracked core concrete, and (3)to protect column corner bars from bond splitting failure. Therefore, the AIJ guidelines require that lateral reinforcement ratio p_{jh} of a beam-column connection shall be not less than 0.002, and shall satisfy Eq. 5.

$$p_{jh} > 0.003\ (V_j\ /\ V_{ju}) \qquad \text{(Eq. 5)}$$

in which V_j: design joint shear, V_{ju}: reliable joint shear strength given by Eq. 2.

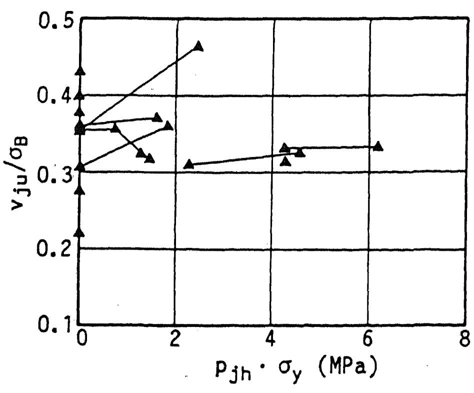

Fig. 8 : Effect of lateral reinforcement in interior joint

3.4.2 Vertical Reinforcement

Design for vertical reinforcement in a joint is not required in the AIJ guidelines, because at least one intermediate column bar is arranged in each face of a column. The effect of vertical reinforcement on the shear resistance of joint is unknown, however test results of exterior joints reported by T.Kaku (1988) are noteworthy such as a proper arrangement of lateral and vertical reinforcement prevented a shear failure of joints especially under a large axial load .

4 ANCHORAGE OF BEAM AND COLUMN REINFORCEMENT

4.1 Anchorage Method

Longitudinal reinforcement of beam normally passes through an interior joint and is anchored with a 90 degree hook in an exterior joint. Yielding of the beam longitudinal reinforcement is liable to penetrate into the joint. Consequently bond resistance deteriorates along the reinforcement in the joint and also hysteretic energy dissipation deteriorates. The bond deterioration may be delayed in the interior joint by passing the beam reinforcement in the confined concrete core, and by limiting the bond stress level in design. The beam reinforcement should be anchored in the joint core of exterior joint.

Joint cover concrete adjacent to a loaded beam may spall out or fail in punching shear with cone-shape by tension of beam reinforcement, resulting in a reduction in anchorage length. However, these behaviors are not so clear that the critical section for anchorage of the beam reinforcement may be taken at the face of the column.

The AIJ guidelines require that longitudinal reinforcement of beams with an yield hinge shall pass through the column core of the joint or be anchored in the column core with a 90-degree hook. Development length of a bar shall be calculated at the critical section at the column face for beam reinforcement or at the top and bottom of beam for column reinforcement.

4.2 Anchorage with 90 degree Hook

There are a few kinds of anchorage failure modes: spalling of side cover concrete around the bend of only corner beam bars, crushing locally of inside concrete at the bend of any beam bars, splitting along the beam bar layer, and spalling of outside cover concrete around the tail of beam bars. It is often difficult to distinguish a shear failure and anchorage failures for tests of exterior joints because some failure modes are similar in concrete crack pattern and occur complexly. In some cases anchorage failure results in a sudden loss of resistance and poor hysteretic energy dissipation characteristics.

Based on recent studies, strength of

anchorage depends on the horizontal projection length of a beam bar including the bend, but it does not relate to the vertical extension of the beam bar beyond 12 times bar diameter. Exterior joint specimens with beam bottom bars bent downward into the lower column normally exhibited smaller flexural resistance under positive bending than that under negative bending.

Therefore, the AIJ guidelines require that a straight portion of beam longitudinal reinforcement before the bend of 90-degree hook should be as long as possible, the hooks shall be extended at least beyond the mid-depth of column, and extension of the 90 degree hook shall be placed in the connection.

4.3 Bars Passing through Joint

When longitudinal reinforcement of beams or columns with intended yield hinges at both faces of a joint passes through the joint, bar size to member depth ratio shall be determined not to cause significant stiffness reduction or slip-type hysteretic behavior under load reversals.

Average bond stress τ_a of beam longitudinal reinforcement passing through a joint is expressed by Eq. 6;

$$\tau_a = a_s \cdot \Delta\sigma_s / D \cdot \phi_s \qquad \text{(Eq. 6)}$$

where a_s and ϕ_s: area and perimeter of a beam bar, $\Delta\sigma_s$: stress difference of a beam bar at both the joint faces, and D: depth of column. The area and perimeter are expressed by bar diameter d_b in Eq. 7, then

$$\tau_a = \Delta\sigma_s \cdot d_b / 4D \qquad \text{(Eq. 7)}$$

If the bond strength is assumed to be proportional to the square root of concrete compressive strength, and the stress difference $\Delta\sigma_s$ is determined for simultaneous yielding in tension and compression at both the faces as the considerable maximum value, Eq. 8 is obtained as a design equation;

$$D / d_b > \sigma_y / \mu \ \mathrm{sqrt}(f'_c) \qquad \text{(Eq. 8)}$$

where, σ_y: yield strength of the beam bar. The guidelines do not specify the value of mu in Eq. 8. It is hard to prevent the bond deterioration for the concrete and reinforcement strengths, bar sizes, and column dimensions usually used in Japan.

The bond deterioration along the beam reinforcement causes the following problems; (1)reduction in hysteretic energy dissipation capacity results in increased earthquake response, (2)pulling out of beam bars from the joint is increased prior to beam bar yielding, (3)large beam end rotation accelerates concrete crushing at the critical section, and (4)repair of the bond deterioration in a beam-column connection, especially interior connection, is difficult. However, on condition that the beam critical region is properly confined and the beam reinforcement is anchored in the opposite side beam across the joint, the loss of bond in the joint may not lead to a sudden reduction in the beam resistance.

K.Kitayama et al.(1986) revealed from a study of earthquake responses with different hysteresis models that the largest response is not sensitive to the hysteretic energy dissipation capacity. The authors proposed a value of mu in Eq. 8 of 10 to maintain the hysteretic energy dissipation capacity at a story drift angle of 1/50 radian. It should be noted, however, that the maximum response amplitude may not be influenced by the decay in the hysteretic energy dissipation, but the number of large amplitude oscillations certainly increases with the decay.

Restriction on the value of μ may be further relaxed because (1)the increase in structural resistance due to the development of the upper bound strength at yield hinges will decrease the structural response, (2)the trouble caused by bond deterioration is less significant than that caused by shear failure, (3)the bond deterioration in a limited number of joints does not have ill effect on the structural response as long as the majority of joints can maintain their stiffness and resistances. For the time being, the value of μ is allowed to take 1.25 for practical use.

The AIJ guidelines require that the bar size to member depth ratio shall be determined not to cause significant stiffness reduction nor slip-type hysteretic behavior under load reversals where longitudinal reinforcement is planned to yield at both faces of the joint.

5 CONCLUSIONS

The outline of design method on reinforced concrete beam-column joints in the AIJ guidelines is mentioned above. There are still many unclear factors left in the design on reinforced concrete beam-column joints, then further investigations are necessary.

ACKNOWLEDGMENT

The AIJ Design Guidelines was drafted by Seismic Design Sub-committee of Reinforced Concrete Committee, AIJ. The authors wish to acknowledge the work by the committee members.

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