

Deterioration Mechanism of Shear-Resisting System in RC Beam Subjected to Reversed Cyclic Loading after Flexural Yielding

by H. Kinugasa and S. Nomura

Synopsis: Based on cyclic tests of RC beams that failed in flexural-shear without yielding of the transverse reinforcement, a mechanism controlling flexural shear failure is proposed. This mechanism, which is associated with ‘Error Catastrophe’ known as a theory of aging, was observed in the hinge region of the beams. The results of experimental testing indicate that a shear-resisting system forms in the flexural hinge region of a RC beam subjected to monotonic loading. Under reversed cyclic loading, the shear-resisting system temporarily disappears as cracks open and then is rebuilt as cracks close. A flexural shear failure occurs when the shear resisting mechanism is not rebuilt upon load reversal. What inhibits the rebuilding process and, ultimately, results in a failure to rebuild, is “errors” in the rebuilding process. These errors accumulate each time the shear-resisting system is rebuilt, and when the errors exceeded a certain tolerance, failure due to the malfunction of the rebuilding occurs.

Keywords: ductility; error catastrophe; failure mechanism; flexural shear failure; RC beam; reversed cyclic loading; shear-resisting system

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INTRODUCTION

For seismic design, it is important to precisely evaluate ductility capacity of beams and columns as well as their strength. It is known that beams subjected to reversed cyclic flexure-shear loading may exhibit significant strength degradation due to shear failure following flexural yielding. This failure mode is known as flexural shear failure. There have been many studies about flexural shear failure, and they have provided useful results. However, although various models have been proposed to predict the ductility capacity at which flexural shear failure develops, the accuracy of these models is not high and is much lower than models used to predict maximum strength. Currently, no method for predicting ductility capacity has been validated. The reason that no validated model exists is that very few studies have been made at the failure behavior of extremely damaged hinge regions and the failure mechanism that develops under large-deformation cyclic loading is not fully understood.

FAILURE DUE TO ERROR CATASTROPHE

The objective of this paper is to show experimentally the existence of a flexural shear failure mode for RC beams subjected to large-deformation cyclic loading, which is associated with 'Error Catastrophe'. Error catastrophe is a theory of aging and summarized as follows (see Fig1),

- ① Our cells are reproducing in our body.
- ② Errors in reproduction occur, causing damage to the reproduction function.
- ③ These errors accumulate each time cells reproduce.
- ④ Catastrophic failure due to malfunction of the reproduction occurs when certain tolerance of error is exceeded.

As a result of cyclic loading tests, a new flexural shear failure mode peculiar to RC beams subjected to reversed cyclic loading in large deformation range was observed. The failure behavior is described as follows (see Fig.2),

- ① Under reversed cyclic loading, a certain shear-resisting system repeats temporary disappearance and rebuilding due to opening and closing of cracks (①→②→③ in Fig.2).
- ② Errors in the rebuilding occur, causing damage to function of the rebuilding.
- ③ These errors accumulate each time the shear-resisting system is rebuilt.
- ④ Catastrophic failure due to malfunction of the rebuilding occurs when a certain tolerance of error is exceeded (②→⑤ in Fig.2).

In order to inhibit flexural shear failure under cyclic loading, RC beams must satisfy the following two conditions. Condition 1: The shear-resisting system is rebuilt after the temporary disappearance that occurs each time the loading direction is reversed (②→③ in Fig.2). Condition 2: The applied shear force does not exceed the shear strength of the rebuilt shear-resisting system (③→④ in Fig.2). The conventional flexural shear failure occurs when condition 2 is not satisfied (③→⑥ in Fig.2); however, here the observed failure mode occurred when condition 1 was not satisfied (②→⑤ in Fig.2).

However, this paper is not to say that the conventional flexural shear failure mechanism does not exist. The purpose of this paper is to show that there is a possibility of another new failure mode that develops in RC beams under large-deformation cyclic loading.

CYCLIC LOADING TESTS OF RC BEAM

Two kinds of cantilever RC beam specimens, denoted A and B, were constructed in order to investigate cyclic strength deterioration under large deformation demands beyond flexural yielding. The details of specimen-A and specimen-B are shown in Fig.3, and the test setup is shown in Fig.4. Specimens were rotated 90-degrees for testing. One end of the member was fixed against rotation and displacement while the other end was pinned. Lateral load was applied to the pinned end. The mechanical properties of the reinforcement are shown in Table 1, and the concrete strength, σ_B , of each specimen is shown in Fig.7. As shown in Fig.5, the deformation behavior in the hinge region and the strains of transverse reinforcement were measured in detail.

Specimen-A was designed so that it would exhibit a flexural failure. To ensure a flexural failure, the shear strength of Specimen-A was designed to be twice as large as the demand associated with flexural yielding. Specimen-B was designed so that it would fail in shear following flexural yielding. This was accomplished by reducing the volume of transverse reinforcement and increasing the longitudinal reinforcement, in comparison to Specimen-A. The shear strength of Specimen-B was designed to be slightly larger than the shear demand associated with flexural yielding.

Specimen-A was tested under monotonic loading and two different cyclic loading histories as shown in Fig.6. These specimens were named Am, A1 and A2 respectively. The purpose of the Specimen-A tests was to investigate the flexural shear failure mechanism. Since Specimen-A1 and A2 exhibited similar failure behavior, further

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analysis is done considering the observed response of Specimen-A1. Specimen-B was intended for comparison. Specimen-B was subjected to the same cyclic loading history as the Specimen-A1.

The observed load (P) versus drift ratio (R) relationships for the specimens are shown in Fig.7. The load (P) and drift ratio (R) are defined in Fig.4. As seen in Fig.7(1), specimen-A had a very large deformation capacity under monotonic loading. On the other hand, specimen-A, when subjected to reversed cyclic loading, failed in flexural shear at smaller drift ratios (See ▼ in Fig.7(2),(3)). Fig.8 shows a typical transverse steel strain versus drift ratio relationship for Specimen-A subjected to reversed cyclic loading. The strain was observed at the transverse reinforcement ST1 which was in the severely damaged region as shown in Fig.16. Since the strain was kept small (Fig.8), it is obvious that yielding of the transverse reinforcement did not occur despite the severe damage.

The observed load (P) versus drift ratio (R) relationship for Specimen-B is shown in Fig.7(4). Rapid strength degradation occurred from the 4th cycle just after flexural yielding. In the Specimen-B, yielding of the transverse reinforcement was observed at the 4th cycle.

TEMPORARY DISAPPEARANCE AND REBUILDING OF SHEAR-RESISTING SYSTEM

To investigate the failure behavior in the hinge region, rotation angle, θ , and lateral displacement, D, of the point “O” shown in Fig.9 were measured. It is noted that, as shown in Fig.10, a decrease in slope in the θ vs. D relationship represents an increase in the shear deformation component in the hinge region. Since it is impossible to directly observe the shear-resisting system in the hinge region, the deformation behavior caused by the shear-resisting system was observed instead using the θ vs. D relationship.

The θ vs. D relationship for Specimen-A subjected to monotonic loading is shown in Fig.11. As can be seen in this figure, an almost linear relationship, $\theta \doteq D/150$, was observed. This implies a shear-resisting system that provides a linear relationship, $\theta \doteq D/150$, formed in the hinge region. This shear-resisting system will be referred to as shear-resisting system of monotonic loading (SOM) hereafter. If the linear relationship, $\theta \doteq D/150$, is observed between θ and D, SOM is considered to be formed in the hinge region.

Fig.12(1) shows θ vs. D relationship for Specimen-A1 subjected to reversed cyclic loading. Fig.12(2) is the corresponding load vs. drift ratio relationship for the specimen. In Figure 12(1) and Fig. 12(2), the solid line shows response for one cycle before flexural shear failure occurs. For a portion of the cycle, a similar linear behavior to that of monotonic loading, $\theta \doteq D/150$, can be seen in Fig.12(1) (see ②③ and ⑤⑥ in Fig.

12 (1)). In other words, the formation of SOM occurs under cyclic loading, indicating that deformation behavior under cyclic loading is based on that under monotonic loading.

However, it is seen that the SOM does not always exist under cyclic loading (Fig. 12(1)). The slope of the θ vs. D relationship decreases temporarily in the low load region just after there is a change in the direction of loading (see ①② and ④⑤ in Fig.12(1)). It is considered that the SOM temporarily disappears due to reversed loading. The low load region just after load reversal (①② and ④⑤ in Fig.12(2)), where the disappearance of SOM is observed, is defined as the 'Slip Region', in which large shear deformation occurs due to temporary opening of cracks of both loading direction. These data indicate that SOM exhibits repeated temporary disappearance and rebuilding under reversed cyclic loading as a result of crack opening and closing. Thus, the rebuilding of SOM is necessary to maintain shear resistance.

FAILURE CAUSED BY MALFUNCTION OF THE REBUILDING

It is possible that the rebuilding of SOM does not always succeed due to an increase in damage in the hinge region. In this section, failure behavior under cyclic loading is examined in terms of SOM.

Fig.13(1) and 14(1) show the θ vs. D relationship for Specimen-A1 and A2 respectively. Fig.13(2) and 14(2) show the corresponding load vs. drift ratio relationship for the specimens. As can be seen in Fig.13(1) and 14(1), before the flexural shear failure (before ▼), for each loading process, the slope of the θ vs. D curve is approximately the same as peak displacement demand is approached. This suggests that SOM is rebuilt. However, once significant strength loss is observed (after ▼), the slope of the θ vs. D curve at peak displacement demand gradually decreases. This indicates that it becomes difficult to rebuild SOM. This suggests that the formation of SOM is necessary to maintain shear strength and that the malfunction of the rebuilding process causes strength degradation.

One question that must be answered is whether the SOM is destroyed by applied shear at the point where strength deterioration initiates. Conventional failure models explain that failure is caused by the destruction of the SOM due to a decrease in shear strength with increasing deformation. Fig.15(1) and (2) show the θ vs. D relationship and the corresponding load vs. drift ratio relationship for Specimen-B, respectively. If the SOM was destroyed by applied shear force, then a decrease in the slope of the θ vs. D relationship, such as that shown in Fig.10, should be observed during the loading process. This decrease in slope during loading results because shear strength decreases and shear deformation increases. The data in Fig.15(1) show a decrease in the slope ; thus destruction of the shear-resisting system occurs as a result of applied shear force for Specimen-B.

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On the other hand, the data in Fig.13(1) and 14(1) do not show the decrease in the slope of the θ vs. D history under loading that is shown in Fig.10 and Fig.15(1). Thus, for Specimen-A, the destruction of SOM does not occur during the loading process. Particularly in Fig.14(1) for Specimen-A2, the slope of the curves tend to increase rather than decrease (see curves after ▼), indicating that shear deformation tends to decrease rather than increase during loading. Again, this implies that SOM is going to be rebuilt, rather than destroyed, during loading.

There is no guarantee that SOM is always rebuilt successfully after the temporary disappearance. Evaluation of the θ vs. D relationship for Specimen-A indicates that the flexural shear failure of Specimen-A was not caused by destruction of SOM but by a malfunction of the rebuilding of SOM after its disappearance (see ②→⑤ in Fig.2).

ERRORS IN REBUILDING

The previous section discusses the disappearance and rebuilding of SOM (Shear-Resisting System of Monotonic Loading) and failure caused by malfunction of the rebuilding process. Rebuilding is not always perfect, and it is likely that some errors in the rebuilding process occur. These errors likely lead to malfunction of the rebuilding process and failure. In this section, the question of what causes malfunction of the rebuilding process is addressed.

In order to examine the failure behavior of the hinge region, the transverse strain in the hinge region (Fig.16) was measured using the measuring apparatus shown in Fig.5. The transverse strain was measured at the location of transverse reinforcement ST1 and ST2, which were located within the severely damaged region of the specimens (Fig.16). Since the transverse strain of ST1 was always a little larger than that of ST2, the strain values for ST1 are used. Although it is important to investigate the deformation of the core concrete in order to understand the failure mechanism of the hinge region, it is very difficult to measure concrete strain directly. Here, concrete behavior was observed indirectly from steel strains.

Fig.17 shows the transverse strain vs. drift ratio relationships for Specimen-A subjected to reversed cyclic loading and monotonic loading. The transverse strain consists of strain due to “yielding” and “bending”(Fig.18). Since yielding of the transverse reinforcement was not observed in Specimen-A, it is considered that almost all the transverse strain for specimen-A was provided by “bending” (Fig.18). As can be seen in Fig.17, in specimen-A1 subjected to reversed cyclic loading, the transverse strain gradually accumulated with loading cycles and increased rapidly when the flexural shear failure occurred. On the other hand, in specimen-Am subjected to monotonic loading, the transverse strain was kept small, compared with that under reversed cyclic loading.

Fig.19(1) shows the transverse strain history and Fig. 19(2) shows the load versus drift ratio history for specimen-A1. It is obvious from Fig.19(1) that the transverse strain

increased in the slip region (see①②,④⑤) where the SOM temporarily disappeared. It is estimated that the increase in the transverse strain due to “bending” resulted from 3-dimensional shear deformation behavior shown in Fig.20. Since large shear deformation occurs in the slip region due to the disappearance of the shear-resisting system, it is quite likely that the shear deformation in such an extremely damaged hinge region is not 2-dimensional.

Consideration of the data in Fig.17 and Fig.19 results in the following observations:

- ① Transverse strain remained small under monotonic loading.
- ② Under reversed-cyclic loading, transverse strain accumulated during the period in which the SOM disappeared.
- ③ Transverse strain increased rapidly when flexural shear failure, resulting from malfunction of the SOM rebuilding process, occurred.

These observations suggest that transverse strain is an error in the rebuilding process, the error accumulates each time SOM is rebuilt under cyclic loading, and accumulated error results in malfunction of the SOM rebuilding process.

CONCLUSIONS

The results of large-deformation cyclic tests of RC beams that exhibit flexural shear failure without yielding of transverse reinforcement provide a basis for the following characterization of the failure mechanism in the hinge region.

- (1) A certain shear-resisting system (SOM) forms under monotonic loading. Under cyclic loading, the SOM repeats temporary disappearance and rebuilding due to opening and closing of cracks. Flexural shear failure can occur due to malfunction of the rebuilding process (see Fig.2).
- (2) What inhibits the rebuilding process and causes the malfunction is errors in the rebuilding. The errors accumulate each time the shear-resisting system is rebuilt, and when the errors exceed a certain tolerance, failure due to the malfunction of the rebuilding occurs.
- (3) Transverse strain (see Fig.16) is considered to be the error in the rebuilding process that finally causes the malfunction. It is estimated that the transverse strain is due to 3-dimensional shear deformation that occurs during the temporary disappearance of the SOM (see Fig.20). The occurrence of this failure mode is considered to be defined by certain tolerance of error, thus a value of the transverse strain.

REFERENCES

- (1) Priestley M.J.N., Verma R., and Xiao Y. (1994), “Seismic Shear Strength of Reinforced Concrete Columns”, Journal of Structural Engineering, Vol.120, No.8, August, pp.2310-2327.

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(2)Xiao Y., Esmaily-Ghasemabadi A. and Wu H. (1999), “High-Strength Concrete Short Beams Subjected to Cyclic Shear”, ACI Structural Journal, V.96, No.3, May-June, pp.392-399

(3)Ichinose T., Imai M., Okano T., and Ohashi K. (1999), “Three-Dimensional Shear Failure of RC Columns after Cyclic Loading, Seminar on Post-peak Behavior of RC Structure Subjected to Seismic Loads”, JCI, pp.171-179.

(4)Lee J.-Y., and Watanabe F. (2003), “Shear Deterioration of Reinforced Concrete Beams Subjected to Reversed Cyclic Loading”, ACI Structural Journal, V.100, No.4, July-August 2003, pp.480-489

Table 1 -- Mechanical Properties of Reinforcement

	Yield Strength	Tensile Strength	Young's Modulus
6 ϕ	366	463	2.04×10^5
D 10	361	509	2.02×10^5
D 13	352	510	2.06×10^5
D 16	402	559	2.02×10^5

(N/m m²)

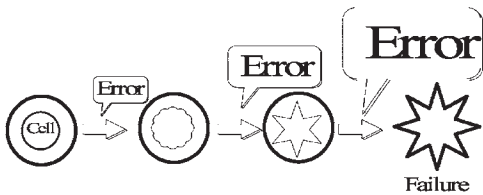


Figure 1 -- Error Catastrophe

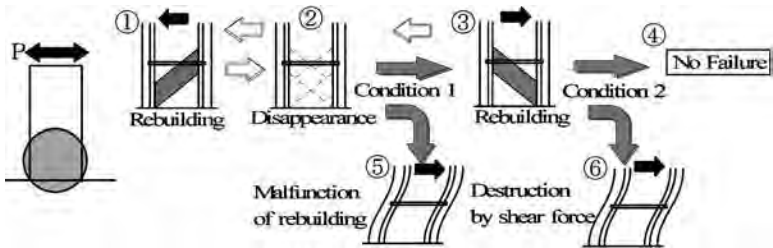


Figure 2 -- Flexural Shear Failure Mechanism in Hinge Region

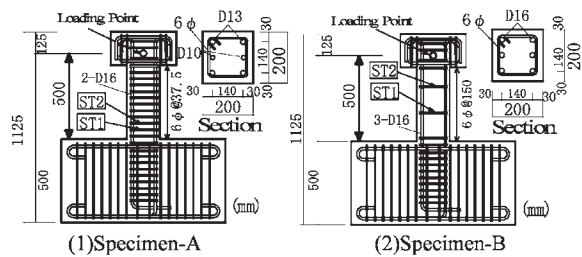


Figure 3 -- Geometry and Reinforcing Arrangement

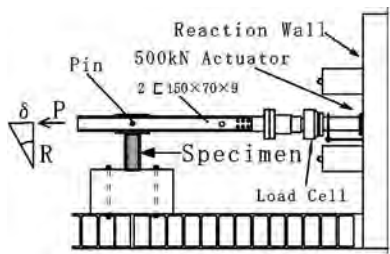


Figure 4 -- Test Setup

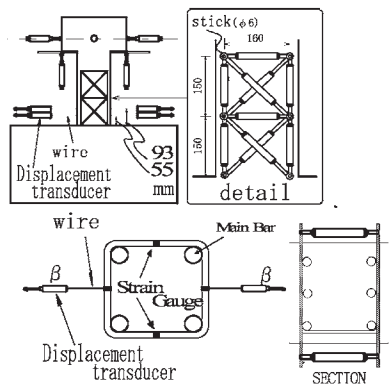


Figure 5 -- Measuring Apparatus

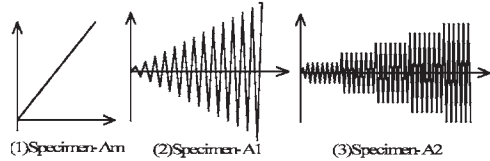


Figure 6 -- Loading Histories

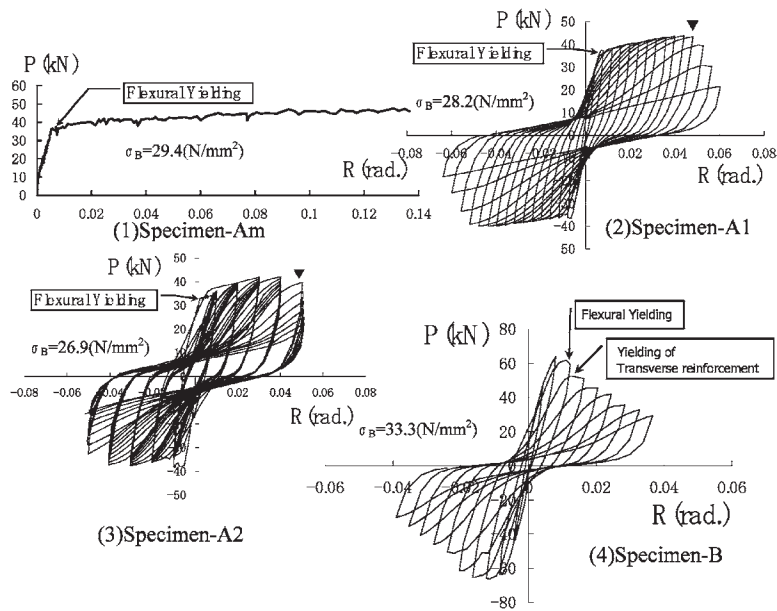


Figure 7 -- Load (P) vs. Drift ratio (R) Relation

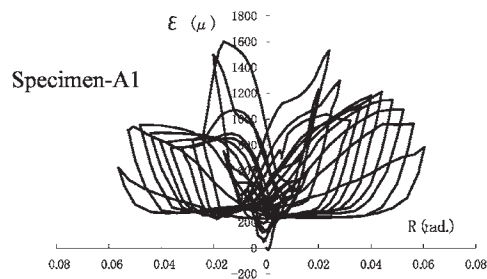


Figure 8 -- Strain of Transverse Reinforcement ST1 for Specimen-A subjected to Reversed Cyclic Loading

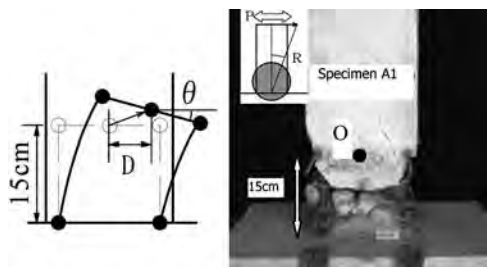


Figure 9 -- Rotation Angle θ and Lateral Displacement D

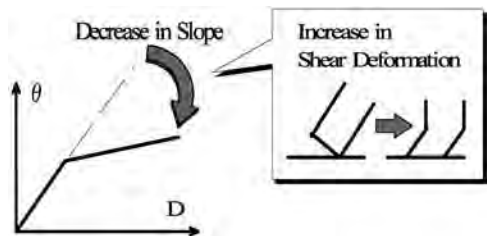


Figure 10 -- Decrease in Slope caused by Shear Failure

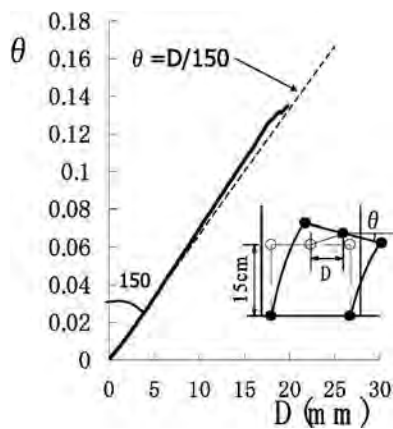


Figure 11 -- Deformation Behavior under Monotonic Loading

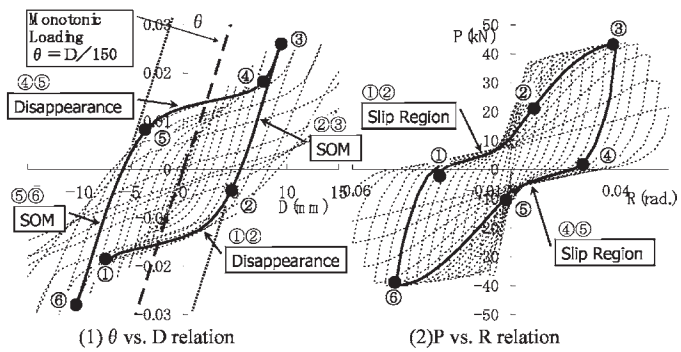


Figure 12 -- Deformation Behavior under Reversed Cyclic Loading (Specimen-A1)

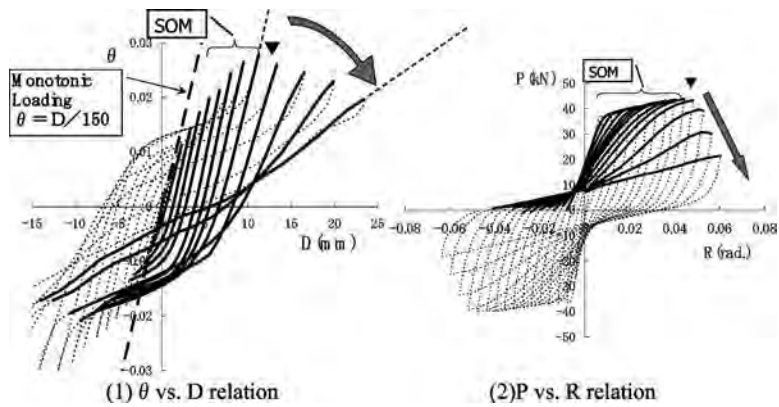


Figure 13 -- Deformation Behavior in Loading Process (Specimen-A1)

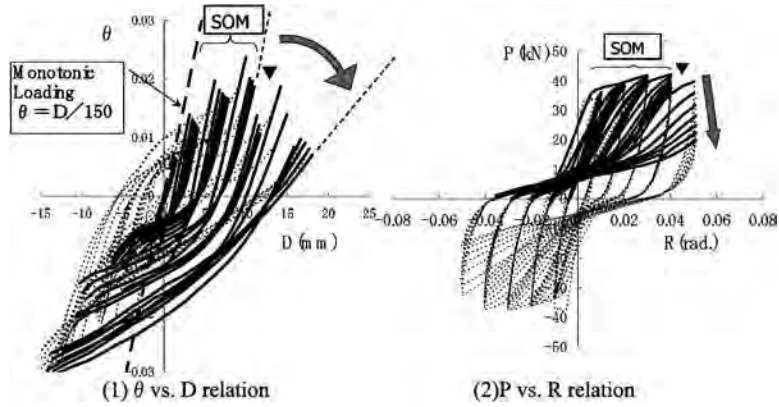


Figure 14 -- Deformation Behavior in Loading Process (Specimen-A2)

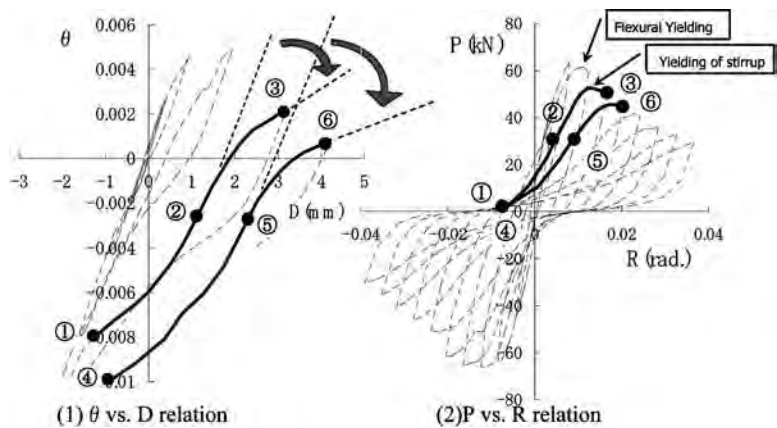


Figure 15 -- Deformation Behavior in Loading Process (Specimen-B)

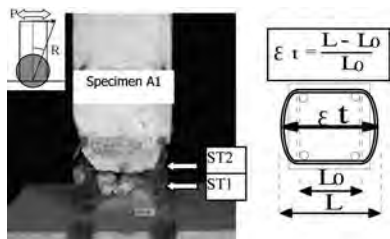


Figure 16 -- Transverse Strain ϵ_t

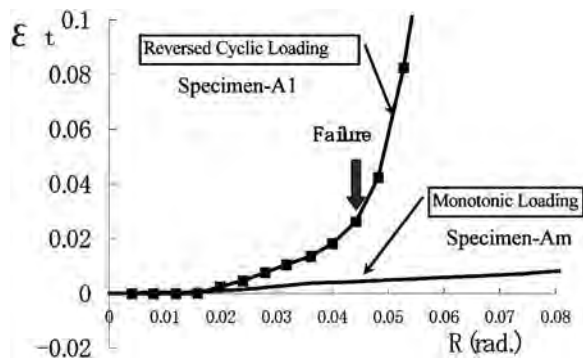


Figure 17 -- Increase in Transverse Strain ϵ_t

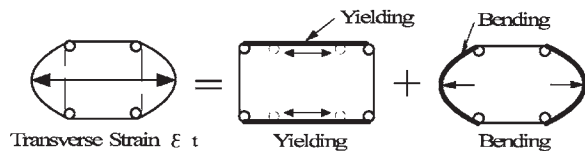


Figure 18 -- Transverse Strain due to Yielding and Bending

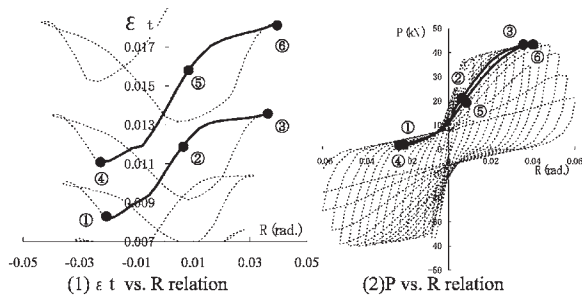


Figure 19 -- Increase Behavior of Transverse Strain ϵ_t for Specimen-A1

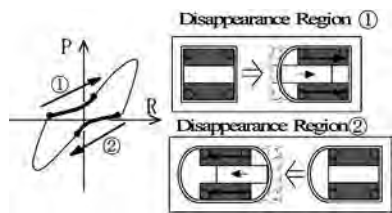


Figure 20 -- Transverse Strain ϵ_t caused by 3-Dimensional Shear Deformation in Slip Region

Consideration of Scale Effect in Reinforced Concrete Members Subjected to Shear Loading

by M. Terai and K. Minami

Synopsis: In most structural members, strength tends to decrease as the member size increases. This phenomenon is known as scale effect. Many experiments investigating the impact of scale effects on RC structures have already been conducted. However, since few laboratories have the capacity to test large-scale specimens, few experimental investigations have tested near full-scale RC members. This paper investigates the scale effect in reinforced concrete members subjected to shear loading. Two different sized test specimens were prepared and tested. The results clearly indicate that the member strength decreases as its size increases. The Japanese building code, an empirically-based code with no scale effect parameter in the shear formulas, generates overly-conservative predictions for full-scale members. Using plane concrete monotonic compression strength data, a formula defining the shear strength of reinforced concrete members is proposed. Experimental results show that the proposed formula for computing ultimate strength agrees better with the experimental data than existing formulas.

Keywords: arch-truss mechanism; column; compressive strength; reinforced concrete; scale effect; shear strength

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1. INTRODUCTION

Scale effect has been recognized since the early 19th century and is extremely important in the design of structures to assure safety and adequate resistance to earthquake forces because it appears in fracture phenomena associated with compression, flexural and shear failure modes. The development of advanced computational analysis tools in the last decade and theoretical approaches that employ fracture mechanics to concrete structures have improved understanding of the behavior of materials and structural members (JCI 1993). In addition, many experimental investigations on the scale effect in Reinforced Concrete (RC) structures have been carried out. However, since laboratory load devices often do not have the capacity to test full-scale specimens, researchers tend to carry out tests on relatively small-scale models. As a result, the Japanese building code has no specific provisions for the scale effect in the structural performance of RC structures.

A shear loading test on two geometrically similar specimens, having diameters of $D=300$ and 600 mm, was conducted in the present study (TERAI et al. 2002) to investigate the scale effect in large-scale RC members. Then, a rational shear strength design formula that takes into account the scale effect is formulated by applying “equivalent strength” to the current Architectural Institute of Japan (AIJ formula) design formula (AIJ 1990). Finally, the validity of the proposed formula is verified by comparing the predicted values with the existing test results of 30 RC members with D values ranging from 300 to 800 mm.

2. EXPERIMENTAL METHODS AND TEST RESULTS

2.1 Test specimens and parameters

The laboratory test specimens were representative of perimeter columns that could be expected to experience large variations in axial forces under earthquake loading. To study the impact of scale effect on the shear failure of RC structural components, a full-scale RC column specimen was designed based on a small-scale model of the section, sized 300 x 300 mm, that had been tested in the authors' laboratory in 1996 (YAMASHITA et al. 1997).

Figure 1 and Table 1 present the specimen details and the structural description, respectively. Two specimens were tested: a full-scale specimen and a 1/2-scale specimen. The cross-sections of the full-scale and half-scale specimens are 600 mm x 600 mm and 300 mm x 300 mm, respectively. The total height (H) of the specimen is 3.0D, where D is the total depth (width) of the specimen.

The test parameters are the specimens' dimensions. Three differences exist between the full-scale and half-scale specimens. The strength of longitudinal bars was SD685 for the full-scale specimen and SD345 for the half-scale specimen. The axial loading was varied for the full-scale specimen, but held constant for the half-scale specimen. Finally, joint parts were not installed at the ends of the column for half-scale specimen, as shown in Figure 1.

2.2 Loading method

The test setup for the specimens is shown in Figure 2. The axial and horizontal forces are applied by a 10 MN oil jack and a 5 MN oil jack, respectively. These jacks are controlled by closed-loop hydraulic control apparatus. A pantagraph was used to keep both end stubs parallel during loading to simulate typical beam behavior in a framed structure subjected to horizontal loads. Reversed-cyclic lateral loading and varying axial loading were applied. The lateral load was applied cyclically until the final failure of the specimen. The cyclic load history consisted of two cycles of gradually increasing

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maximum drift demands up to a maximum drift angle of 1/25 rad. by 1/200 rad.

2.3 Test Results

Shear force-deformation curves for the two specimens are shown in Figure 3. Figure 4 provides the hysteric shear stress-drift angle curves in the positive loading direction for the two specimens to facilitate comparisons. From a comparison of the full-scale and half-scale model data in Figure 4, the impact of scale effect can be observed in the peak shear stress that was generated at the same drift angle, such as R=2.0% rad.

Figure 5 shows the plot of $\tau/\sqrt{\sigma_B}$ obtained from a test against the D value. The values calculated using the AIJ formula are also plotted in the figure. Figure 5 reveals that the scale effect is clearly observed in these test results.

3. PROPOSED FORMULA FOR SHEAR STRENGTH OF RC MEMBERS

The authors are attempting to introduce a scale effect law into the AIJ formula, as N. Shirai derived the shear strength prediction formula for RC beams by introducing it into the AIJ formula, N. Shirai et al. (1998). The AIJ formula is selected to maintain continuity and practicality of the current design formula and due to the fact that the AIJ formula is basically a rational theoretical formula derived on the basis of the lower bound theorem of plastic analysis.

3.1 Formulation of shear strength design formula and verification

In the current AIJ Design Guideline, AIJ (1990), the ultimate shear strength is based upon a strut and tie model. The ultimate shear strength of reinforced concrete members with shear reinforcement V_u has generally been expressed with the following equation.

$$V_u = V_a + V_t \quad (1-1)$$

V_t : design shear capacity carried by shear reinforcement steel and obtained from the following equation.

$$V_t = b \cdot j_t \cdot p_w \cdot \sigma_{wy} \cot \phi \quad (1-2)$$

V_a : design shear capacity by the strength of concrete, shear span ratio and the amount of

flexural reinforcement, representing the contribution of concrete and obtained from the following equation.

$$V_a = \tan \theta (1 - \beta) b D \nu \sigma_B / 2 \quad (1-3)$$

$$\text{in which, } \tan \theta = \{ \sqrt{[(L/D)^2 + 1]} - L/D \} \quad (1-4)$$

$$\beta = \{ (1 + \cot^2 \phi) p_w \cdot \sigma_{wy} \} / (\nu \sigma_B) \quad (1-5)$$

$$\nu = 0.7 - \sigma_B / 200 \quad (1-6)$$

In this equation, σ_B is the compressive strength of concrete (MPa), p_w is the shear reinforcement ratio, and σ_{wy} is the yield strength of shear reinforcement (MPa). For $\sigma_{wy} > 25\sigma_B$, $\sigma_{wy} = 25\sigma_B$ and for $p_w \cdot \sigma_{wy} > \nu \sigma_B / 2$, $p_w \cdot \sigma_{wy} = \nu \sigma_B / 2$. j_t and L are the distance between the centers of the longitudinal bars in the tension and compression sides and the shear span length, respectively. ν is the effectiveness factor of the compressive strength of concrete, and ϕ is the angle of the compressive concrete strut in the truss mechanism and is defined as $\cot \phi = \min\{2.0, j_t / (D \tan \theta), \sqrt{\nu \sigma_B / (p_w \cdot \sigma_{wy})} - 1.0\}$.

3.2 Proposed formula for shear strength of reinforced concrete members

As shown in Eq. (1-1), the AIJ formula calculates the shear strength as the sum of the truss and arch mechanism contributions. However, the shear strength is primarily due to the truss mechanism, as shown in Fig. 6. The arch mechanism is additionally considered only in the case where there are few shear reinforcements. Therefore, to accurately discuss how the scale effect can be applied to prevent brittle shear failures in RC members, it is necessary to have specific provisions that account for scale effect not only in the arch mechanism, but also in the truss mechanism. To facilitate the development of a model in which this is accomplished, the angle of the concrete strut for both the arch and the truss mechanism is assumed to be identical, as is the width of the cross-section b that is incorporated in both the arch and the truss mechanisms.

The AIJ formula assumes that the failure criterion is satisfied when the superposition of the resulting stress induced in the compressive concrete strut for the truss and arch mechanisms reaches $\nu \cdot \sigma_B$. In the present study, the nominal strength defined by the

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concrete compressive strength will be applied to this criterion; that is, the failure criterion is satisfied when $\lambda(\nu \cdot \sigma_B) = \sigma_N$. Thus, after replacing $\nu \cdot \sigma_B$ in Eq. (1-3) with σ_N , V_a can be redefined as follows:

$$V'_a = \tan \theta (1 - \beta) b D \sigma_N / 2 \quad (2)$$

where λ is the strength reduction rate, which is dependent on the characteristic length of the specimen. From monotonic compression test data for plane concrete, which were obtained in author's laboratory (TERAI et al. 2001), λ was determined to be

$$\lambda = -0.11 \ln(d) + 1.48 \quad (3)$$

where d is the characteristic length of the specimen in mm. Figure 6 shows the experimental data characterizing the scale effect for compressive strength testing of concrete cylinders and cubes. The compressive strength of plain concrete was tested using specimens with diameters varying from 100 mm to 400 mm, both circular and square cross sections, and height-to-diameter ratios (H/D) of 1, 2 and 3. Note that the predicted value by the approximation type is also plotted.

The truss mechanism contains both flexural and shear reinforcements and a concrete diagonal strut, as shown in Fig. 7. Under the proportioned stress condition, the compressive strut strength of concrete ${}_c\sigma_t$ is defined by Eq. (4):

$${}_c\sigma_t = (1 + \cot^2 \phi) p_w \cdot \sigma_{wy} \quad (4)$$

After replacing ${}_c\sigma_t$ in Eq. (4) with σ_N , the following is obtained:

$$(1 + \cot^2 \phi) p_w \cdot \sigma_{wy} = \lambda(\nu \cdot \sigma_B) \quad (5)$$

Therefore, after replacing $p_w \cdot \sigma_{wy}$ in Eq. (1-3) with $\sigma_N / (1 + \cot^2 \phi)$, V_t can be redefined as follows:

$$V'_t = b \cdot j_t \cdot \sigma_N \cot \phi / (1 + \cot^2 \phi) \quad (6)$$

3.3 Evaluation of the proposed formula

The authors have proposed a formula for predicting the shear strength of RC members based on an experimental study of the monotonic compressive strength of concrete in authors' laboratory. In Japan, many formulas have been previously proposed to estimate the shear strength of RC members. These include the well-known JSCE recommendation (JSCE formula), JSCE (2002) and the AIJ formula. These formulas will be compared

with the proposed formula. The validity of the proposed formula is verified through a comparison with the test results of 27 RC specimens failing in shear. The collected data is for specimens with $D=75\sim600$ mm. Figure 9 displays plots of the ratio between the test values and the predicted values against the test values for the AIJ formula, the JSCE formula, and the proposed formula, respectively. For the JSCE formula, the shear strength ratio increases gradually with increasing tested shear strength, and is much larger than those computed using the other formulas. The data in Figure 9 show that the scatter of the shear strength ratio for the JSCE formula and the AIJ formula is much larger than that of the proposed formula. Figure 10 shows the ratio of tested to calculated shear strength as a function of d value for previously tested members that experienced shear failure. The scatter of the data for the previously proposed model is somewhat larger than that of the proposed formula. As shown in Figure 9 and 10, the proposed formula that accounts for the impact of scale effect on the strength of concrete is more accurate than the other formulas.

4. CONCLUSIONS

The following conclusions are drawn from the results obtained in this study. First, a comparison of the full-scale and half-scale experimental results indicates that the scale of the specimen has a significant impact on its strength. A formula defining the strength of reinforced concrete members is proposed using the test data defining the monotonic compression strength of plane concrete. Using data from previous experimental tests, it was shown that the proposed model provides better prediction of observed strength than previously proposed formulas.

5. REFERENCES

- Architectural Institute of Japan (1990) Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concept, Maruzen, Tokyo
- Japan Concrete Institute (1993) Applications of Fracture Mechanics to Concrete

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Structures, Maruzen, Tokyo

Japan Society of Civil Engineers (2002) Standard Specifications for Concrete Structures -2002-, Structural Performance Verification, Maruzen, Tokyo

Shirai, N., Moriizumi, K. And Tamura, M. (1998) Formulation of Shear Strength Design Formula for Reinforced Concrete Beams Considering Size Effect, *Proceedings of FRAMCOS-3*, Vol.2, pp.1407-1418

Terai, M., Nagatomi, Y. and Minami, K. (2001) Experimental research about Scale effect on Compressive Strength of Concrete with No Confinement, *Proceedings of AIJ Tyugoku Chapter Architectural Research Meeting*, Vol.24, pp.45-52 (in Japanese)

Terai, M., Nagatomi, Y. and Minami, K. (2002) Study on The Elasto-Plastic Behavior of Large Dimension RC Columns Subjected to Varying Axial Loading, *Proceedings of the fib 2002 Osaka Congress*, Vol.5, pp.461-466

Yamasita, Y., Kuwata, Y. and Minami, K. (1997) Experimental research about Shear failure of Reinforced Concrete Columns with high strength (8000kgf/cm² class) reinforcement (No.1-5), *Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan*, C-2, pp.1-10 (in Japanese)

Table 1--Material properties of concrete and steel

Specimen	Concrete Compressive Strength(MPa)	Shear Reinforcement			Longitudinal Bar		Axial Loading Ratio N/σ _B bD
		Type	s(mm)	σ _y (MPa)	Type	Pt(%)	
Full-Scale Model	27.2	4-S10 KSS785	100	904	16-D25 SD345	385	Comp.:0.6 Tens.:0.16
Harf-Scale Model	39.8	4-D13 SBPR785	50	930	16-D16 SD685	685	0.3(const.)

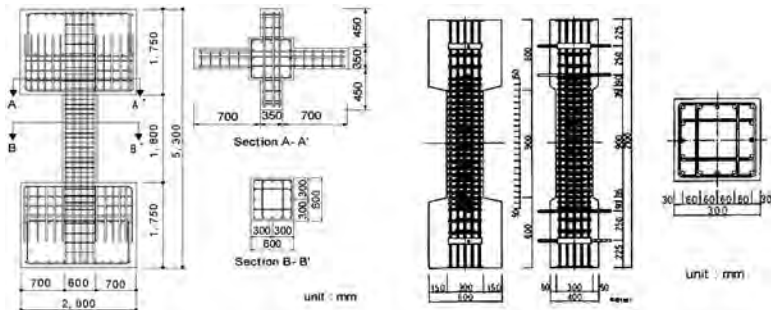


Figure 1--Specimen details

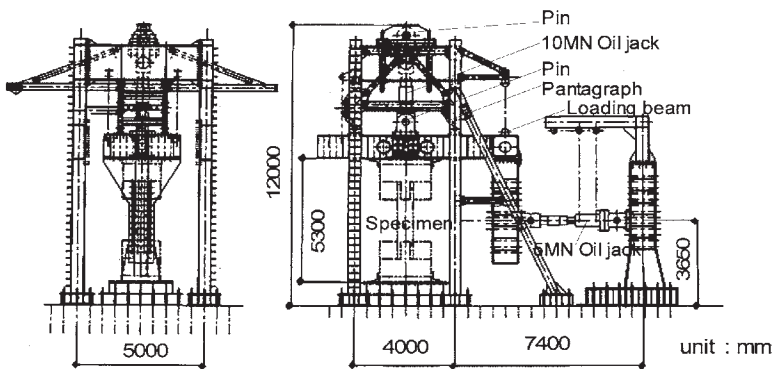
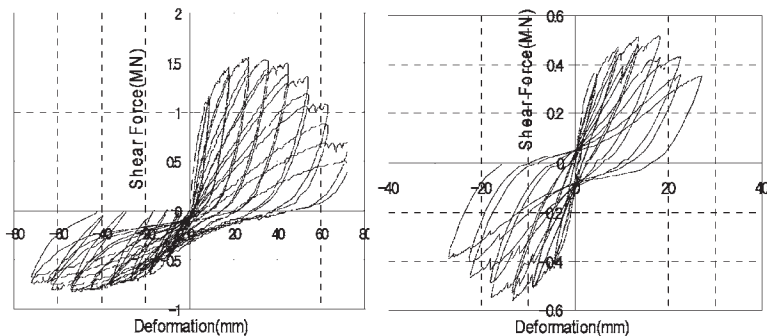


Figure 2--Load device



(a) Full-scale model

(b) Half-scale model

Figure 3--Hysteretic shear force-deformation curves

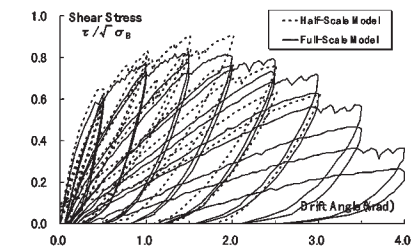


Figure 4--Hysteric shear stress-drift angle curves in positive direction

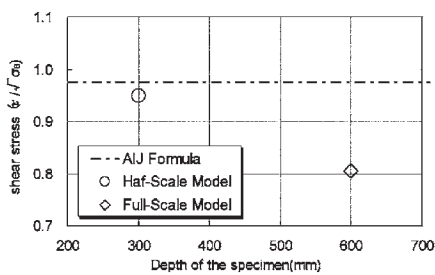


Figure 5--Shear stress and depth

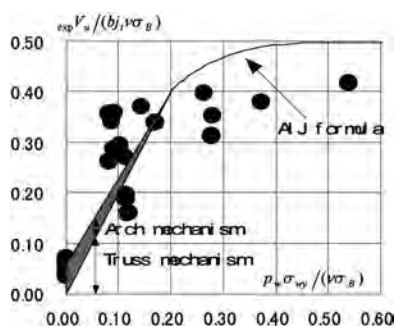


Figure 6--Contribution of shear reinforcements to shear strength

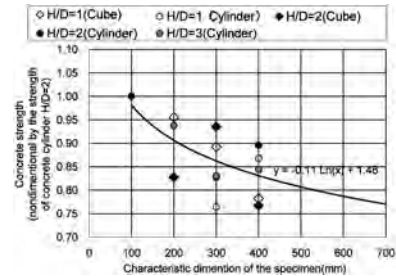


Figure 7--Scale effect on compressive strength of concrete

Experimental Study of Recovered Properties of Retrofit (Repaired) Concrete Members

by K. Suzuki and A. Fujii

Synopsis: Two interior column (RC)-beam (PPC) specimens were tested. The specimens were first subjected to reversed, repeated cyclic loading to maximum rotational angles of $1/30$ and $1/16$, respectively. The specimens were then repaired using resin injection and mortar covering. Mechanical properties obtained following repair were almost the same as those obtained during the initial loading, probably, due to the confined concrete used in the beam ends. However, a reduction in initial stiffness of approximately 30% was observed.

Keywords: confined concrete; reinforced concrete; repair; retrofit; seismic performance

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Atsushi Fujii: Graduated from Science Univ. of Tokyo. Currently working at Konoike Gumi Co. Ltd. (construction company), belonging to research section on building structures. Working and research areas: development of structural system of high-rise building , base-isolation systems, seismic retrofit, earthquake observation, vibration tests of structures, simulation of structural dynamics.

PREFACE

Many buildings exhibited significant damage as a result of the 1995 Hyogoken-Nambu Earthquake. In many cases, it was necessary to destroy and rebuild those buildings. Most buildings were not repaired. The main reason that repair was not done was because it was unknown how to quantify the extent to which repair methods restored seismic performance.

In this study, the effect of repair on damaged members was evaluated by laboratory tests. Test specimens were two cruciform-shaped column (RC) and beam (PPC) joints. Beams did not have heavy confining reinforcement. In the first test, specimens were loaded to maximum response deformation angles of 1/30 and 1/16. Then, repair was performed by resin injection and patching. After the repair, the specimens were loaded to response of the plasticity region again.

MEANING FOR REPAIR IN SEISMIC DESIGN

In defining earthquake loads on a structure for large-scale earthquakes, peak ground velocity 40-50cm/sec and story drift angle 1/100 are almost standards (level 2 input action) Japanese seismic design. But, in the recent large earthquakes, such as the 1995 Hyogoken-Nambu Earthquake, the earthquake demand exceeded this standard. Therefore, this increased demand must be taken into consideration to ensure adequate response for huge earthquakes which occur in the future (level 3 input action). Typically, concrete structures respond inelastically when the deformation angle exceeds 1/100, as shown in Fig. 1 (Suzuki 1998). In this case, bearing capacity of seismic walls may be lost. Additionally, beams and columns with poor confining reinforcement will exhibit deterioration in bearing capacity deterioration of under cyclic loading. Collapse of the frame may begin as shown in curve B in Fig. 1. This collapse can be prevented by, for example, vibration control technology in restraining the magnitude of the building response of deformation angle within around 1/100 to the excessive input action. However the increase in construction cost for this cannot be ignored.

This research work is based on the use of members with confined concrete that exhibit the mechanical properties shown by curve A in Fig.1. Confined concrete results in very little increase in construction cost. Thus, repair technology is considered one

effective way to address seismic designs for future huge earthquakes. Therefore, demand representative of these huge earthquakes, which exceeds level 2 input a maximum deformation angle of 1/100, are considered in this study. Repair technology is evaluated for damage due to these demand levels associated with huge earthquake.

One way to repair concrete members is to confine the plastic zone of member. Increasing the volume of transverse reinforcement by reducing the pitch of hoops or stirrups in the plastic-hinge zone results in only a slight increase in construction cost.

OUTLINE OF THE SEISMIC TEST

Specimens

Specimens are cruciform-shaped sub-assemblages that consist of a reinforced concrete (RC) column and a partially pre-stressed and reinforced concrete (PRC) beam and are named №3 and №5 (Matsutani 1998). Fig. 2 shows the dimension of the specimens and the reinforcement arrangement. Specimen design details are provided in Table 1. Beam pre-stressing tendons are partial prestressed to 40% of equivalent yield strength. Members are designed to ensure beam flexure yielding controls response. The characteristics of the materials are shown in the Table 2.

Repair works

Repair of the beams and joint was done after the first test (Test 1) as shown in Figure 3. This repair consisted of two activities. One was resin injection. In this activity ten injection pipes were inserted into crack openings and glued in place, the surface of the cracks were covered with putty, and epoxy resin was injected into the cracks under low pressure using a rubber injector. The second activity was patching of concrete; this was done after the epoxy resin was allowed to cure for one day. The beam cover concrete, broken under compression loading, was removed and a mortar mixed with fiber was used to patch the specimen. The repair material characteristics are shown in the Table 2 (Suzuki et al. 1998).

Seismic loading

The same loading and measurement systems were used in both Test 1 and Test 2 (the test after the repair). The loading system consisted of applying equal but opposite shear load to the beam ends and a constant axial load to the column (column axial load of $0.15f_cA_g$ was used where f_c is the concrete compressive strength and A_g is the gross area of the column). Beam shear load was applied under displacement control to develop the story deformation angle, R , history shown in Figure 4.

RESULTS OF TESTS

Fracture mode

Fig. 5 shows the relationship between beam shear and deformation angle for Test 1 and Test 2 for both specimens. Points of interest on the load history and the calculated strength are provided also in Fig. 5. For both Test 1 and Test 2, concrete cracking progressed in the same manner. Cracking and crushing occurred in both

specimens in the same order, "beam flexural crack", "flexural shear crack", "panel shear crack", "beam tension bar yield", and "beam compression concrete crushed". Beam flexural cracking was concentrated at the column interface. Concrete crushing and beam bar slip in the panel zone occurred earlier in Test 2 than in the Test 1. Most cracks in Test 2 were new ones.

Stiffness and restoring force

Initial stiffness, 2nd stiffness (between the points of flexural cracking and yielding), the maximum restoring force and the deformation angle at maximum load are shown in the Table 3. Initial stiffness in Test 2 decreased to 0.78 or 0.68 of that observed in Test 1. This was likely due to the fact that resin was not injected into cracks with very small crack width openings .

Hysteresis characteristics

Envelope curves of the hysteresis loops for Test 1 and Test 2 are shown in Figure 6 along with the hysteresis loop for the final cycle of Test 1. The restoring force for Test 2 was about the same as for Test 1, though it is somewhat smaller at initial yield.

A single hysteresis loop for the maximum deformation angle demand of $3/100$ is shown in Figure 7. The data in Figure 7 suggest that the bar-slip in the joint panel at a deformation angle demand in excess of $3/100$ is more significant for Test 2 than Test 1. The peak load for Test 2 develops once deformation due to slip decreases. Repair of beam-end cover concrete using high-strength mortar is thought to contribute to the improved mechanical properties of the section.

Energy absorption

The equivalent damping constant calculated from hysteresis area is shown in Figure 8. The damping constant for Test 2, for all deformation states ,is smaller than that for Test 1. Damping characteristics for both Test 1 and Test 2 are stable with for multiple load cycles to the same displacement demand .

Consideration on repairing

A main difference between Test 1 and Test 2 is the slip of beam reinforcing bars in the joint. Not only minute crack openings but also gaps around reinforcing bars should be filled with resin during the injection work.

CONCLUSION

The recovery of mechanical properties in repaired specimens loaded up to deformation angles of $1/30$ and $1/16$ was investigated experimentally from the view point of seismic performance. Obtained results are as follows.

- 1) The initial stiffness of repaired specimens decreases to about 70% of that observed for undamaged specimens. This may be caused by incomplete injection of resin and failure to inject resin into cracks with very small crack width openings.
- 2) Bearing capacity and ductility of repaired specimens is almost the same or higher than those of undamaged specimens.
- 3) Improvement by using of high strength mortars to patch cover concrete was fairly effective.

These results may suggest a new approach to seismic design. Concrete structures attacked by extremely strong earthquakes and deformed to large drift angles, such as 1/20, can be repaired under the condition that concrete in plastic hinge zones is well confined (in these tests confinement was D6@50mm).

ACKNOWLEDGEMENT

This research was carried out as one of the various researches for verification of the seismic performance of buildings girder using pre-grouted tendons with steel reinforcement in the study group of "After-Bond Cable" organized by companies: Konoike Construction, Shinko Wire Industries, Sumitomo Electric Industries, Netsuren, Fudo Construction and Ken Architect Office, around with Professor Y. Matsuzaki (Science University of Tokyo) and the author. And, the repair work took the cooperation of Sho-Bond. The author expresses his gratitude to the persons concerned.

REFERENCES

- (1) Kazuo SUZUKI (1998); Proceedings of JCI Symposium on "Deterioration of Concrete Members in the Plastic Yielding Stage under Cyclic Seismic Overload", P.9~(by Kazuo Suzuki), Aug. 1998, (in Japanese)
- (2) Kazuo SUZUKI, Yasuhiro MATSUZAKI et al (1998); "Experimental study on seismic performance of reinforced concrete building structure using 'After-Bond' prestressed cable", Annual Conference Abstracts of AIJ, B: 1053-1060
- (3) Teruo MATSUTANI, Kazuo SUZUKI, Suguru ARAKANE, Shigeru Mizoguchi (1998). "Experimental study on seismic performance of reinforced concrete building structure using 'After-Bond' tendons", Proceedings of JCI Symposium on "Deterioration of Concrete Members in the Plastic Yielding Stage under Cyclic Seismic Overload", Aug.1998, pp.337-342, (in Japanese)

Table 1--Specification of specimens

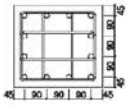
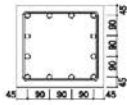
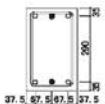
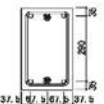
		No. 3 (AB-Y-40)	No. 5 (AB-B-40)
Frame size		H=1,440mm, L=3,240mm [H ₀ =1,080mm, L ₀ =2,880mm]	
Columns	Section		
	Size	b _c × D _c = 360mm × 360mm	
	Concrete	F _c = 380kgf/cm ² (design)	
	Main bar	12-D19 [ρ _g =2.66%]	
	Hoop	Col. 4-S8@50 [ρ _w =1.10%] Panel 2-S8 6sets [ρ _w =0.474%]	
Beams	Section		
	Size	b _b × D _b = 210mm × 380mm	
	Concrete	380kgf/cm ² (design)	
	Main bar & tendon	2-D16 + 1-15.2 φ 6 Strand [ρ _t =0.710%]	2-D16 + 1-19 φ 6 Bar [ρ _t =0.901%]
	Pretension	P _e /P _{py} =0.40 P _e =9.1tf/tendon	P _e /P _{py} =0.40 P _e =9.1tf/tendon
	Stirrup	End 2-S8@50 [ρ _w =0.943%]	Mid 2-S8@100 [ρ _w =0.472%]
Vertical Load		η = N / (b _c D _c σ _g) = 0.15	

Table 2--Material properties

1) Steel bar and tendon

	Yield strength kgf/cm ²	Yield strain × 10 ⁻³	Young's Modulus × 10 ⁶ kgf/cm ²	Tensile strength kgf/cm ²	Elongation %
D 16	3913	2057	1.98	5673	24.2
D 19	4003	2182	1.98	5762	24.1
S 8	9167	4118	2.22	10967	15
15.2 ϕ (strand)	17645	—	1.99	19703	6.4
19 ϕ (bar)	10177	—	2.05	11033	14

2) Concrete

	Compressive strength kgf/cm ²	Peak strain × 10 ⁻³	Young's Modulus × 10 ⁶ kgf/cm ²	Splitting strength kgf/cm ²
Test 1 (Age 9-20 weeks)	423 ± 3%	1979	3.22 × 105	26.5
Test 2 (Age 38 weeks)	410			

3) Repair materials

	Compressive strength kgf/cm ²	Young's Modulus × 10 ⁶ kgf/cm ²	Tensile/Bond strength kgf/cm ²
Mortars ^{*1}	767 (7 days) 879 (7 months)	2.78 (7 days) 3.07 (7 months)	45 (7 days)
Resin ^{*2}	624 (7 days)	0.17 (7 days)	[170 (7 days)]
Sealings ^{*2}	840 (7 days)	0.54 (7 days)	[146 (7 days)]

*1 Test size ϕ 50 × 100, JSA 1108, JSA 1113

*2 Test size 15 × 15 × 43, JSK 7208, JSK 5850

Table 3--Results of tests

		Initial Stiffness × 10 ³ tf/rad.	2nd Stiffness × 10 ³ tf/rad.	Peak Load tf	Deformation at Peak Load rad./1000
Specimen No.3	Test 1	2.29	1.06	8.63	29.9
	Test 2	1.79	0.57	9.42	29.9
	Test 2 / Test 1	0.78	0.54	1.09	1.00
Specimen No.5	Test 1	2.39	1.05	9.07	29.9
	Test 2	1.57	0.54	9.26	29.8
	Test 2 / Test 1	0.66	0.51	1.02	1.00

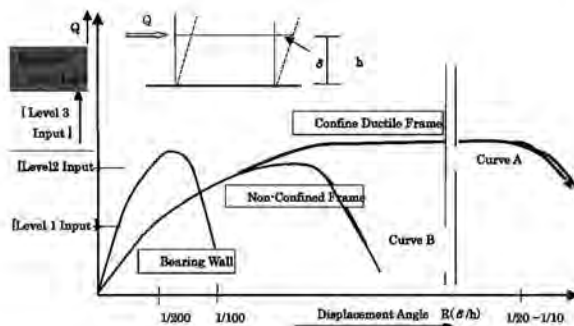
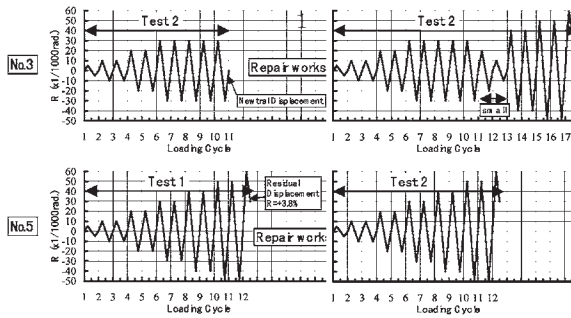
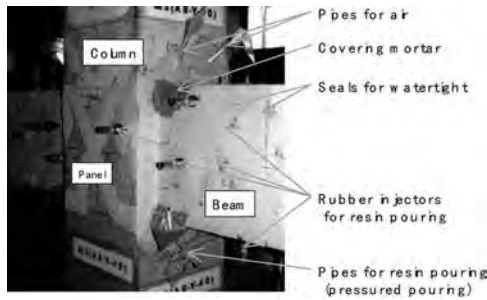
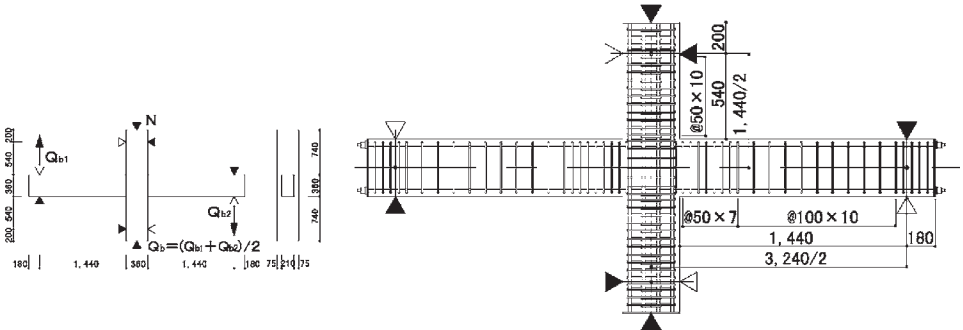


Fig. 1--Image illustration of relation between input and response



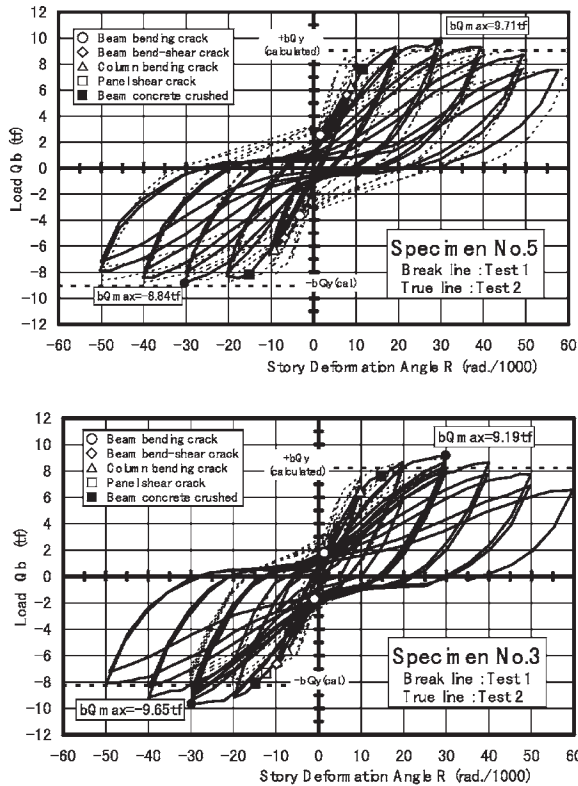


Fig. 5--Relation between load and deformation

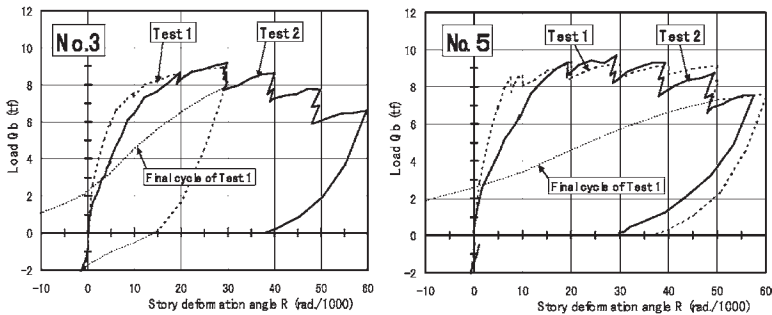


Fig. 6--Envelope curves

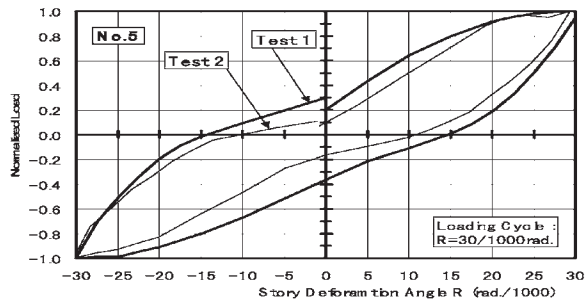


Fig. 7--Hysteresis loop at large deformation

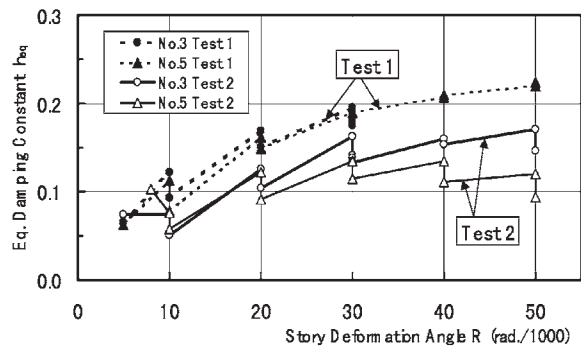


Fig. 8--Equivalent damping constant

Fatigue Failure Criterion for Deformed Bars Subjected to Large Deformation Reversals

by T. Higai, H. Nakamura, and S. Saito

Synopsis: After the Hanshin-Awaji earthquake, the volume of lateral reinforcement used in reinforced concrete structures in Japan was increased to improve seismic performance. Although a large volume of closely spaced lateral reinforcement will be effective in preventing elastic buckling of the longitudinal reinforcement, it will not prevent plastic buckling. Under severe earthquake loading, longitudinal reinforcement will be subjected to large reversed-cyclic deformation demands into the plastic regime in tension and the buckling regime in compression. The authors presented the results of low cycle fatigue testing of deformed bars and examination of failure criteria for these bars.

Keywords: buckling; deformed bars; failure criteria; low-cycle fatigue; plastic strain

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INTRODUCTION

To prevent the collapse of reinforced concrete structures under a severe earthquake, such as Hanshin-Awaji earthquake, it is necessary that structures be designed to achieve a large deformation capacity, e.g. deformation ductility factor of 10 for the single column piers. To attain a high level of ductility, not only should the structural components be designed to ensure that response is determined by flexural rather than shear, but also a large volume of closely spaced lateral reinforcements should be employed to prevent early buckling of longitudinal reinforcement and to confine concrete under compressive loading.

Although closely spaced lateral reinforcement is effective in preventing elastic buckling of longitudinal bars by reducing the unsupported length of the bars, it can not prevent plastic buckling of the bars unless the space of lateral reinforcement is extremely small. Moreover, as the curvature of the cross section becomes very large if the bars buckle over very short unsupported lengths, they may exhibit brittle fracture after a few load cycles. Fracture of buckled longitudinal reinforcement has been observed often during experimental investigation of the seismic performance of RC piers with large volumes of lateral reinforcement.

Although fracture of the longitudinal reinforcement should be avoided to ensure stability of the structure, only a few studies have been conducted to investigate the buckling behavior of longitudinal reinforcing bars and develop failure criterion. Accordingly, the authors carried out an experimental study on the low-cycle fatigue failure of deformed bars to clarify some aspects of this phenomenon. A fatigue testing machine with hydraulic grips was used. First, deformed bars were subjected to repeated constant-amplitude displacement cycles ranging into the plastic regime in tension and buckling regime in compression. Then fatigue tests were conducted using a cyclic displacement demand history of varying amplitude, this displacement history was intended to provide a simple representation of earthquake load conditions.

EXPERIMENTS

Outline of experimental program

Deformed D19 bar, which has a nominal diameter of 19mm, with right-angle lugs spaced at 10mm, were used in the experimental study. Physical properties of the bar are provided in Table 1. A servo-controlled hydraulic fatigue testing machine was used. The machine has hydraulic grips with maximum capacity of 200kN.

Repeated loading test with constant amplitude (Series A)-- In this series, a specimen was held at both ends using the hydraulic grips and subjected to reversed cyclic loading to a constant peak displacement demand until failure. The variables in the experiments were unsupported length l (18d, 12d and 6d, where d is the nominal diameter of the bar) and average strain under the peak displacement (where tension is defined positive).

According to the JSCE Standard Specification for Concrete (JSCE 2002), longitudinal reinforcement shall be supported by lateral reinforcement spaced at a distance that does not exceed 12 times the diameter of the longitudinal reinforcement. The unsupported length of test specimens was decided considering this provision. Peak values for displacement demand was determined by trial such that 1) the average strain at peak displacement demand was between 0 and $20,000 \mu$, in tension (ϵ_t , denotes peak tensile strain) and between $-13,000 \mu$ and $-54,000 \mu$ in compression (ϵ_c , denotes peak compressive strain). and 2) the number of load cycles at failure was less than 40 (Table 2). Testing was done under displacement control. The first cycle was applied pseudo-statically while the following cycles were applied dynamically with a sinusoidal wave of 0.1Hz.

Repeated loading test under variable amplitude (Series B)-- In this series, unsupported length was fixed at 18d, where d is the diameter of the longitudinal bar. Six displacement histories were used. Each history was a random combination of four basic histories with peak tensile strain and peak compressive strain of 10,000 and $-25,000 \mu$, 10,000 and $-50,000 \mu$, 20,000 and $-35,000 \mu$, 30,000 and $-25,000 \mu$. Similar to series A, the first displacement was applied pseudo-statically while the following cycles applied dynamically with a triangular wave of 0.1Hz.

Examination of measurement of deformation of the bars

Figure 1 shows the results of tension tests of the bars with unsupported length of 18d ($18 \times 19.1 = 343.8\text{mm}$). In this test, the measured displacement of the actuator head was approximately twice the displacement calculated from the strain in the reinforcing bar measured at mid length of the bar. Based on detailed measurement of the components of the test set-up, it is estimated that about 40% of the error in measured displacement was due to the movement of grips, and the remaining 60% was due to slip of the reinforcing bar and elasto-plastic deformation of the bar at the contact points with the teeth of the chuck. Given the results of this tests, it was concluded that it was not possible

to determine the displacement of the bar by measuring the displacement of the actuator head.

Thus, the authors developed the displacement measurement system shown in Fig. 2. First, a steel ring was screwed to the bar at each end of the bar adjacent to the chuck. Then a flexible wire was fixed to the upper ring and connected to LVDT-A through a plastic roller screwed on the lower ring. Axial deformation of the bar was measured as the change in distance between two steel rings.

To ensure accurate measurements, two additional features were added to the measurement system. First, to ensure that the rings did not loosen following yielding of the reinforcing bar, a spring was placed between the screw and the steel tip bearing on the bar, as shown in Fig. 3. The spring ensured that the steel tip was always pushed against the bar. The free length and stiffness of the spring were approximately 14 mm and 50 N/mm, respectively. Second, as LVDT-A was fixed on the steel bed of the fatigue testing machine (Fig. 2), any movement of the lower grip and chuck would cause error in the measurement. To avoid this effect, another LVDT (LVDT-B) was used as shown in Fig. 4. LVDT-B was pulled by flexible wire fixed to the lower ring. Axial displacement of the bar measured by LVDT-A was corrected by subtracting the displacement of lower ring measured by LVDT-B. Here, the length and slope of the pull wire of LVDT-B should be same as that of pull wire of LVDT-A. As flexible but low elongation pull wire was required, small diameter steel strand (outer diameter 0.5mm) was used and preloaded to approximately its tensile strength.

Figure 5 illustrates an example of measured tensile strain of the bar. The solid line in Figure 5 shows the intended applied displacement history while dotted line shows the displacement history as measured using the LVDT measurement system described above. Good correlation between the two histories is obtained up to very large strain demand (6%) and under cyclic loading.

FATIGUE FAILURE UNDER CONSTANT AMPLITUDE LOADING

Failure behavior

For the bar specimens with unsupported lengths of 18d, 12d and 6d, cyclic tests were carried out under different conditions by changing the value of peak tensile strain (ε_t) and peak compressive strain (ε_c). A summary of the test parameters and results is provided in Table 2. Deformation data were examined carefully and data were omitted if the displacements measured by the two LVDT-As were very different, or if the ratio of actuator to bar displacement was much different than the typical. Thus, while at least 3 specimens were tested for each set of test parameters, in some cases only one or two data points were considered to be reliable and include in the data in Table 2.

Figure 6 and Fig. 7 show the typical relationship between load and axial displacement of the specimens for the case of unsupported length of 18d under loading condition 4 and the case of unsupported length of 6d under loading condition 3, respectively. Table 2

shows applied loading conditions in this series.

Specimens with longer unsupported lengths (18d) exhibited elastic buckling (denoted by circle mark in figure) at a compressive force less than corresponding to the yield load, following the loading in tension into the plastic regime during the first cycle. After the start of buckling, compressive force decreased gradually as compressive displacement demand increased. After the second cycle, load at the peak displacement demand in tension and the maximum compressive load gradually decreased and the hysteresis loop gradually stabilized.

On the other hand, specimens with shorter unsupported length (6d) exhibited plastic buckling during the first cycle after compressive force reached that corresponding to yield load. As compressive deformation demand increased, compressive load increased further due to the strain hardening. After the second load cycle, both tensile and compressive load at peak displacement demand gradually decreased with increasing cycle counts. For these specimens, the hysteresis loop is spindle shaped and energy dissipation was greater than for the specimens with unsupported length of 18d.

Though the behavior of the specimen with an unsupported length of 12d was intermediate between those with unsupported length of 18d and 6d, it exhibited elastic buckling and thus its behavior was nearer to that of the 18d specimen. In all cases, as shown in Fig. 8 specimens exhibited buckling in the direction normal to the plane intersecting the two longitudinal ribs and cracking initiated on the concave side of the bar at mid-span of the unsupported length. The reason for crack initiation on the compression side of the bar is that the local strain amplitude in the compression extreme fiber is larger than in the tension extreme fiber.

Failure criterion

Figure 9 shows the relationship between nominal value of average strain at the peak displacement demand, ε_c and ε_t , and number of loading cycles, N , at failure. Numbers in the figure denote the values of peak tensile strain. As expected, the larger the absolute value of peak strain, the smaller the loading cycle number, N , at failure. Additionally, for the same peak strain demand, the number of loading cycles at failure, N , is decreased when the unsupported length of the specimen is decreased.

Assuming the number of load cycles at failure is defined by Eq. 1, constants A , B and C may be determined using the experimental results by means of the least-square method. As shown in Fig. 10, the effect of average strain varies as a function of the specimen's unsupported length, though the effect of ε_c is different than that of ε_t . Parameter C is approximately constant for all cases.

$$N = EXP(-A \cdot \varepsilon_t + B \cdot \varepsilon_c + C) \quad (1)$$

Considering these results, Eq. 2 was proposed for use in defining failure. Constants in

Eq. 2 were determined using the least-square method and the experimental data. For this analysis, the 67 data points listed in Table 2 were augmented with another 14 data points that are not presented in Table 2. These additional data points were not included in the table because of the large differences between the measured and target strains, however, since there was no other problem with these experiments, they were used in this analysis. Here the values of peak strain are not the nominal values but the average of the peak values measured in all cycles.

$$N = EXP \left\{ \left(A \cdot \frac{l}{d} + B \right) \varepsilon_t - \left(C \cdot \frac{l}{d} + D \right) \varepsilon_c + E \right\} \quad (2)$$

This analysis resulted in the following values for the parameters in Eq. 2: $A=2.019$, $B=-67.61$, $C=1.254$, $D=-49.55$ and $E=4.502$. Fig. 11 and Table 3 show the relationship between calculated failure cycle N_c and experimentally observed failure cycle N_e . For all 81 data points, as seen in Table 3, the average of the ratio N_c/N_e is 1.02 and the coefficient of variation is 24.7%. The average values of the ratio are approximately the same for specimens with different l/d ratios. However, the coefficient of variation of the ratio is larger for the 6d case, reflecting the difficulties in experimental testing of the plastic buckling.

Applicability of the failure criterion to the bars with different diameter

Equation 2 was derived statistically using experimental data for D19 bars. The applicability of the equation for bars of different diameter was studied analytically, because limitation in loading facilities precluded testing of bars of larger diameter.

Buckling behavior was studied analytically using the finite element method where the bar was modeled using beam elements and finite deformation theory was applied. The employed beam element was a 3D-beam element with 2 nodes and 12 DOF. Material non-linearity was considered and the stress-strain relationship used for the steel fibers was that proposed by Fukuura and Maekawa (1997). The boundary condition of the analytical model was set similar to that of experiment, with both ends of the bar fixed against displacement and rotation and with load applied to the bar end via displacement control. To simulate the initial irregularity of the bar that induces buckling, a small lateral load was applied at mid-span of the bar. A more detailed discussion of the analytical method can be found in Tanoue et.al (2001). The results of this Tanoue et.al. study indicate that the behavior of a steel bar subjected to large axial displacement reversals, which lie between tensile plastic regime and compressive buckling regime, can be estimated well using this analytical method.

Figure 12 shows the calculated relationship between average stress and average strain for D25 bar (nominal diameter : 25mm) with l/d of 18, under two cycles of loading with peak compressive strain of $-30,000 \mu$ and peak tensile strain of $20,000 \mu$. Figure 13 shows calculated local strain at the extreme fiber in the direction of buckling at the

mid-span section of the bar under the same condition. Both results are confirmed identical with the calculated results for D19 bar. These results show that the relationship between average stress (load/cross-sectional area) and average strain (deformation/unsupported length) and that between average stress and local strain are the same regardless of the bar diameter, provided the ratio of l/d is constant. Thus, Eq. 2 can adequately predict the behavior of steel bars of varying diameter, since failure is controlled by the local strain history at the critical section.

Further experiments are required to improve the accuracy of failure prediction by taking into account other factors that influence response, such as the lug pattern of the bar that may affect the stress concentrations that develop around the lugs, and the chemical composition of the steel that may affect the performance of bars under bending and stretching action.

FATIGUE FAILURE UNDER VARIABLE AMPLITUDE LOADING

Loading conditions

Under seismic loading, longitudinal reinforcement will be subjected to reversed-cyclic axial displacement demand of random magnitude. To investigate behavior under these conditions, in this test series, the random displacement histories resulting from earthquake loading are simulated in a very simple manner. Four types of “basic loading conditions” are defined in Table 4. Peak tensile and compressive strain of the basic loading conditions are $10,000 \sim 30,000 \mu$ and $-25,000 \sim -50,000 \mu$ respectively, and the failure cycle N_e under the repetition of each basic loading condition is about 13 to 32. Failure cycle N_c under the basic loading, calculated by Eq. 2 based on the average values of measured peak strains, is also shown in Table 4. In this series, the unsupported length of the specimen was 18d.

The four basic loading conditions were then combined into the six “loading patterns” A to F shown in Table 5. As an example, the strain history of pattern A is shown schematically in Fig. 14. In the subsequent sections, application of one basic loading condition is denoted “Cycle”, and application of one loading pattern, i.e. combination of four basic loading conditions, is denoted “Round”.

Failure criterion

Three specimens were tested under each loading pattern. Observed fatigue life of the specimens is shown in Table 5. A description of 6-2, in the column of fatigue life in Table 5, indicates that the specimen failed during 2nd cycle of 6th round.

The applicability of Miner’s law for predicting damage under variable amplitude loading was examined using the data. Miner’s law assumes that fatigue failure occurs when accumulated damage D , calculated by Eq. 3, reaches to 1.0.

$$D = \sum_i n_i / N_i \quad (3)$$

where,

n_i = number of acting strain cycles at a given strain level

N_i = number of cycles causing failure at the same strain level

Accumulated damage, D , at failure for each specimen is shown in Table 5, where N_i is the failure cycle calculated using Eq.2, with the peak strains measured in each loading cycle of first round. Since accumulated damage, D , is between 0.82 and 1.14 for all patterns and the average value for all patterns is approximately 1.0, Miner's law can be considered to provide good accuracy in predicting failure.

Figure 15 and Fig. 16 illustrate the relationship between load and axial displacement of the specimen in the first round of pattern A and F, respectively. Numbers in the figures denote the number of basic loading condition. As shown in these figures, the load-displacement history is very much affected, particularly in tensile displacement region, by the order in which basic loading conditions are combined. On the other hand, the data in Fig. 17 and Fig. 18 shows that the envelope of the load-displacement relationship after the second round of loading is the same regardless of the loading pattern used. This explains why fatigue life is approximately the same for loading patterns A to F.

CONCLUSIONS

Low cycle failure criterion for deformed bars subjected to large tensile displacements into the yield regime and large compressive displacements into the buckling regime was studied, primarily using experimental testing. Although there still remain some aspects to be studied further, such as the effect of the deformation pattern on the bar surface and the effect of the chemical composition of the steel, primary conclusions of this study are summarized as follows.

(1) A new method of measuring the axial displacement of deformed bar in plastic and buckling region is developed.

(2) Fatigue failure criterion is shown to be a function of the ratio of unsupported length to the diameter of the bar (l/d) and the average strains at peak displacement demand (ϵ_t, ϵ_c). This criterion is considered to be applicable to bars with different diameters, because the results of FEM analysis show that the relationships between average stress and average strain, and between average stress and local strain at the critical cross section are identical provided the ratio l/d is constant.

(3) Miner's law is examined and found to be applicable and provide acceptable accuracy in estimating accumulated damage for random cyclic loading.

REFERENCES

Fukuura, N. and Maekawa, K. (1997). Computational Model of Reinforcing bar under

Reversed Cyclic Loading for RC Nonlinear Analysis. Jour. of materials, concrete structures and pavements. JSCE. No.565/V-35: 291-295
 JSCE.(2002). Standard Specifications For Concrete Structures, Seismic Performance Verification.

Tanoue, K. et al. (2001). Modeling of Nonlinear Cyclic Behavior of Buckled Reinforcing Bars. Jour. of structural eng. JSCE. Vol.47A: 725-734

Table 1-- Physical properties of the bars

Series	Yield stress N/mm ²	Tensile strength N/mm ²	Young's modulus 10 ⁵ N/mm ²
A	359	—	1.78
B	372	575	1.79

Table 2-- Summary of the experimental results in Series A

Fixed length	Loading condition	Target peak strain 10 ⁻⁶		Number of specimens	Loading cycles at failure	
		Tension	Comp.		Each specimen	Average
18d	1	0	-49000	4	20,28,24,25	24.3
	2	9000	-25000	3	34,25,32	31.0
	3		-35000	4	25,25,19,33	25.5
	4		-50000	6	14,15,17,15,14,12	14.5
	5	20000	-29000	5	26,24,23,25,28	25.2
	6		-40000	3	18,15,17	16.7
	7		-54000	6	13,14,9,12,13,18	13.2
12d	1	0	-44000	2	17,19	18.0
	2	8000	-20000	2	16,31	23.5
	3		-29000	1	23	23.0
	4		-45000	4	10,10,9,9	9.5
	5	18000	-20000	1	25	25.0
	6		-29000	3	15,13,13	13.7
	7		-45000	1	10	10.0
	8		-52000	3	6,6,5	5.7
6d	1	0	-35000	3	25,31,32	29.3
	2	6000	-20000	2	37,24	30.5
	3		-24000	4	18,20,17,14	17.3
	4		-35000	1	10	10.0
	5	13000	-13000	4	19,29,26,19	23.3
	6		-20000	2	24,17	20.5
	7		-26000	1	8	8.0
	8		-35000	2	7,10	8.5

Table 3 -- Applicability of proposed equation

Fixed length	18d	12d	6d	All
Number of specimens	41	19	21	81
Average ratio of N_u/N_e	1.02	1.14	1.08	1.06
Standard deviation	0.216	0.253	0.324	0.262
Coefficient. of variation %	21.3	22.1	30.0	24.7

Table 4 -- Basic loading conditions

Basic loading condition	Target peak strain 10 ⁻⁶		Measured peak strain 10 ⁻⁶		Number of cycles at failure		
	Ten.	Comp.	Ten.	Comp.	N _e (Exp.)		N _e (Cal.)
					Each specimen	Average	
1	10000	-25000	6335	-29235	30,34,37,28,29	31.6	33.6
2	10000	-50000	8302	-54549	11,16,16,16,13,16,17	15.0	16.0
3	20000	-35000	20742	-39942	14,17,17,14,13	15.0	16.0
4	30000	-25000	28798	-28160	16,11,13,13,14	13.4	17.1

Table 5 -- Summary of the results by variable amplitude loading test

Loading pattern	Combination of basic loading conditions	Cycles at failure Round - Cycle	Accumulated damage D	
			Each specimen	Average
A	3 → 1 → 4 → 2	6-2, 6-1, 5-3	1.09, 1.01, 0.94	1.01
B	4 → 1 → 3 → 2	5-1, 5-4, 5-4	0.90, 1.06, 1.06	1.00
C	4 → 3 → 1 → 2	6-1, 5-2, 6-2	1.08, 0.94, 1.14	1.05
D	3 → 1 → 2 → 4	4-4, 5-3, 5-3	0.82, 0.97, 0.97	0.92
E	4 → 3 → 2 → 1	5-4, 5-1, 6-1	1.03, 0.88, 1.09	1.00
F	1 → 4 → 3 → 2	5-1, 5-1, 4-4	0.85, 0.85, 0.82	0.84

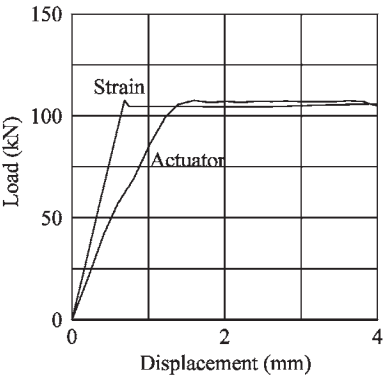


Figure 1 -- Actuator displacement

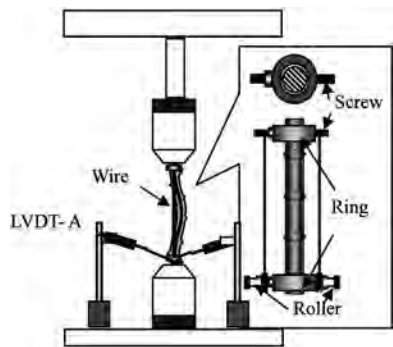


Figure 2 -- Measurement of axial displacement

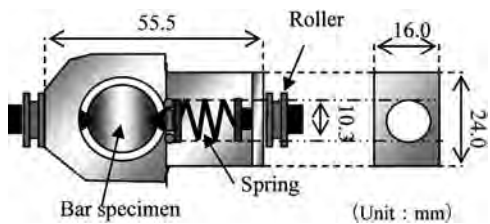


Figure 3 -- Detail of steel ring

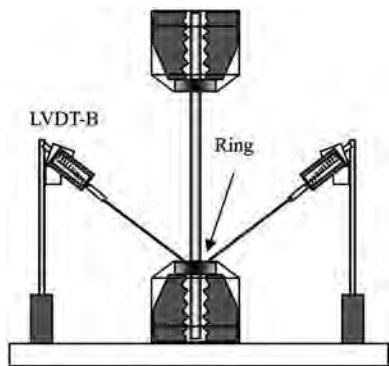


Figure 4 -- Supplementary measurement

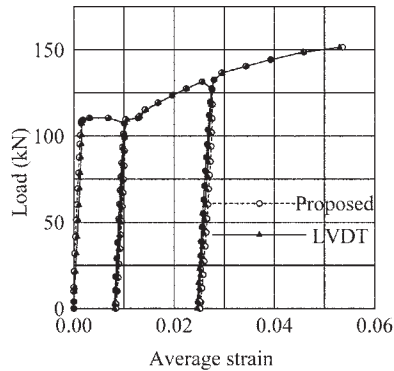


Figure 5 -- Measurement by proposed method

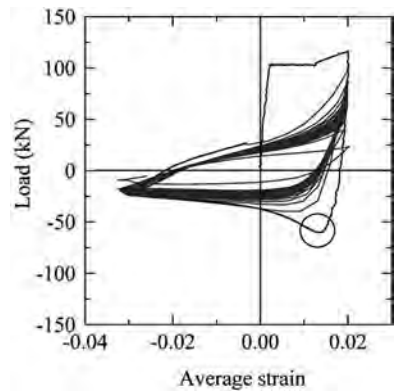


Figure 6 -- Load-displacement curve (l/d=18)

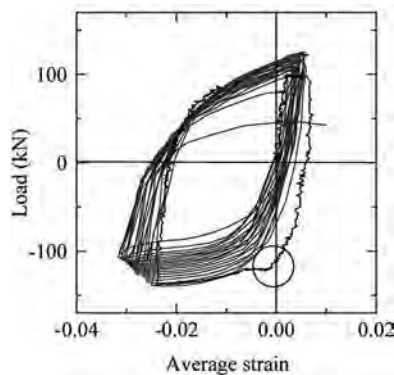


Figure 7 -- Load-displacement curve (l/d=6)

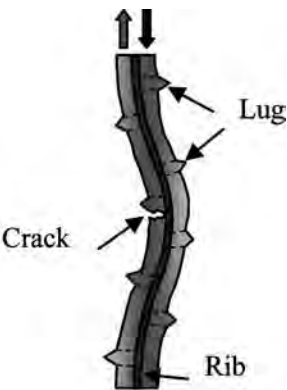


Figure 8 -- Development of crack

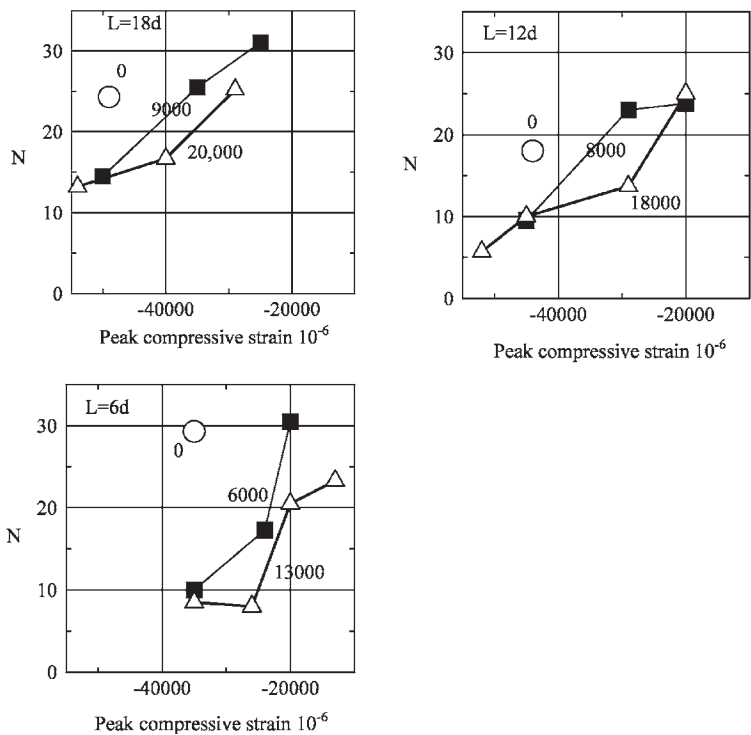


Figure 9 -- Relation between peak strain and failure cycle

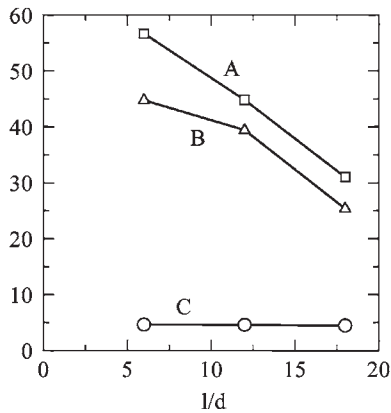


Figure 10 -- Coefficients in Eq. 1

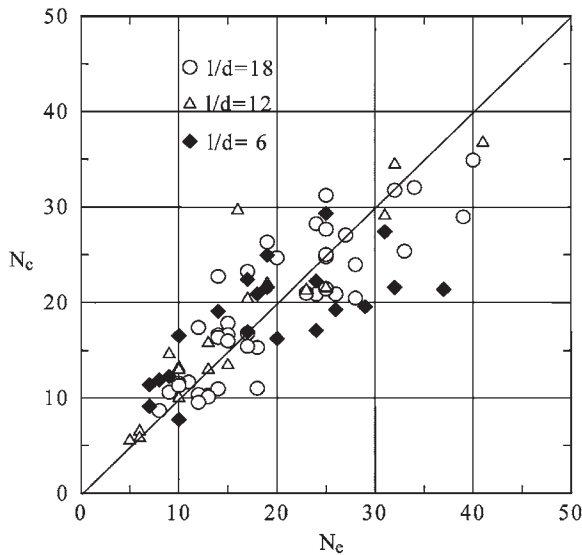


Figure 11 -- Relation between calculated and experimental fatigue lives