論文 The Reduction in the Shear Strength of RC Linear Members after the Yielding of Longitudinal Reinforcement

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ABSTRACT: It was found that a RC member subjected to post-yield load reversals failed in shear even if it was designed to have the shear strength 2.16 times the flexural strength in the beginning[1]. The ductility which depends upon the shear capacity is affected in general by axial stress, shear reinforcement ratio, longitudinal reinforcement ratio and shear span-depth ratio. But, the shear reinforcement ratio being most dominating factor for the shear strength of linear members, in this study only the effect of shear reinforcement ratio is investigated without paying much attention to the other factors. Finally, a formula to calculate ductility factor is proposed.
KEYWORDS: reduction in shear strength, flexural yielding, shear reinforcement ratio (r_w)

1. INTRODUCTION

In early 1995, an earthquake of magnitude 7.2 on the Richter Scale shook one of the largest cities in Japan, down town Kobe. In this earthquake numerous concrete piers for highway viaducts and Sinkansen Lines for more than 1 km length were completely damaged. The failure was mainly due to shear. In this earthquake, even though before the yielding of longitudinal reinforcement the ratio of shear to flexural capacity was kept much higher than unity, the repetition of large deflection reversals weakened the member in shear and finally the clear shear failure was seen. The reason for this may be that being accompanied by the increase of deflection, the length and width of flexural cracks increase, shear cracks appear and the shear strength decreases. So, the deformation level and the ductility of a member is governed by the shear capacity itself [2].

Hence, it is essential to design a member with sufficient shear capacity so that even at large deflection repetition enough margin of shear capacity is available to avoid such dramatic shear failure. In this regard, it is also of extreme importance to know the pattern of post-yield shear degradation under load repetitions. So far, it is realised that it is very difficult to evaluate analytically the decrease of the shear strength after the yielding of the longitudinal reinforcement under load reversals. So, this research works presents the solution from the experimental observation on a the small scaled specimen which is the model of a reinforced concrete bridge pier.

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2. OUTLINE OF THE EXPERIMENT

2.1 SPECIMENS

Six specimens of cantilever type as shown in Fig. 1 were made keeping all constants except r_w. All specimens were designed so that flexural failure would take place before the shear failure would occur. The base of the specimen was heavily reinforced in order to prevent the cracks propagation over it and simulate it as a rigid base foundation .

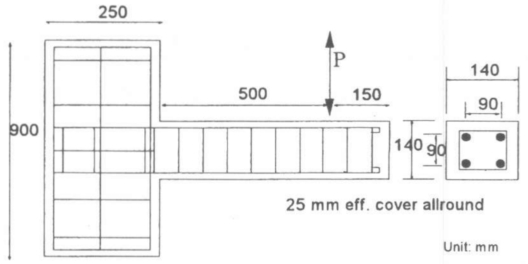

Fig.1 Example of Pier Model (PM-4)

2.2 LOADING SET-UP AND METHOD OF LOADING

Hydraulic jack was used to apply lateral load to the model pier, increasing subsequently in small increment and at very low frequency, i.e., static cyclic loading to simulate earthquake loading as shown in Fig. 2. The testing was started with the loading of specimen in one direction while the hydraulic jack on another direction was released. The loading was done till longitudinal bar showed yielding as indicated by the strain gauges. Then, the loading was stopped in the direction. At this moment, the load applied and the displacement at the loading point was measured which were taken as yield load (P_y) and yield displacement (δ_y), respectively. Then, the loading was started in another direction till δ_y deflection was reached. This was repeated upto three cycles. After this, same procedures follow but the displacement amplitude is increased incrementally in the following way: $\pm 2\delta_y$, $\pm 3\delta_y$, $\pm 4\delta_y$, $\pm 5\delta_y$,.........

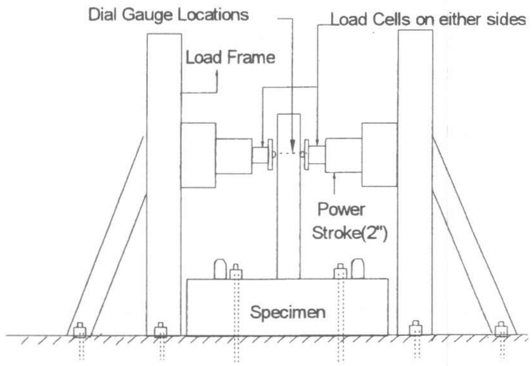

Fig. 2 Loading Set-up

3. LOAD-DISPLACEMENT RELATIONS

The load-deflection relations *(P-δ)* was obtained for each pier specimen and on the successive load repetitions the final displacement at which the load reduced to the yield load, smaller by the yield displacement was defined as the ultimate displacement (δ_u) and then loading was stopped. The ductility factor, the ratio of the ultimate deflection δ_u to the yield displacement δ_y is often used as an index of ductility of a member. The initial, cracking and yielding stiffnesses were almost the same in all specimens. The pier specimen showed very good energy absorption for increasing shear reinforcement ratio. Under the load repetition, the restoring force maintained well above the yield load until the shear cracks fully widened but after the shear cracks widened fully, the significant shear slip took place and the restoring force suddenly dropped to below the yield load. At final failure, the restoring force decreased well below the yield load and wider shear cracks were clearly visible. The P-δ relations of typical pier specimens are presented in Figs. 3, 4, 5 and 6, respectively and the summary of the test results is presented in Table 1.

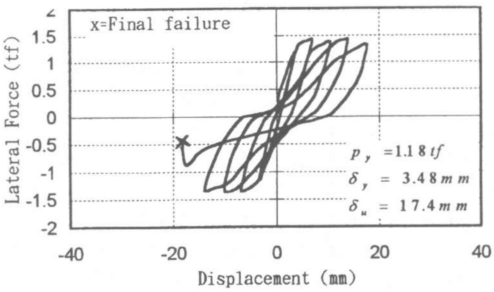

Fig. 3 Load-Displacement Relation of PM-1

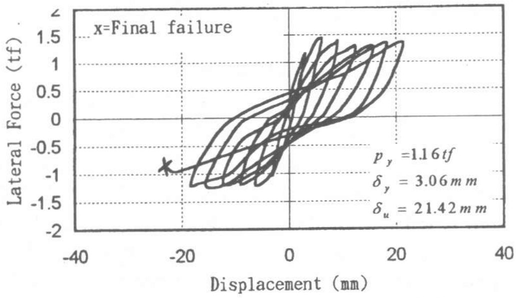

Fig. 4 Load-Displacement Relation of PM-3

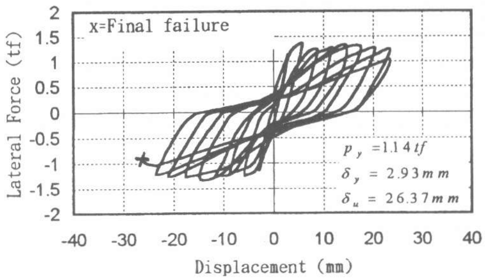

Fig. 5 Load-Displacement Relation of PM-5

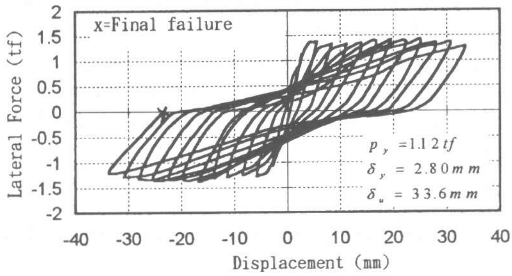

Fig. 6 Load-Displacement Relation of PM-6

Table 1 Summary of Test Results

Speci. No.	b x d	r_w	a/d	f_c'	f_y	P_y	δ_y	Yielding Cycle at d-section	Yielding Cycle at 2d-section	Mode of Failure
PM-1	14x11.5	0.05	4.35	239	3992	1.18	3.48	$2\delta_y$-1	$4\delta_y$-1	1
PM-2	14x11.5	0.10	4.35	261	4006	1.12	2.73	$2\delta_y$-1	$4\delta_y$-1	1
PM-3	14x11.5	0.15	4.35	250	3996	1.16	3.06	$2\delta_y$-1	$5\delta_y$-1	1
PM-4	14x11.5	0.20	4.35	250	3998	1.15	3.10	$2\delta_y$-1	X	2
PM-5	14x11.5	0.25	4.35	255	4005	1.14	2.93	$2\delta_y$-1	X	2
PM-6	14x11.5	0.40	4.35	256	4004	1.12	2.80	$2\delta_y$-1	X	3

Note;

f_c' = Compressive strength of concrete, kgf/cm^2

f_y = Yield strength of reinforcement bars, kgf/cm^2

b, d = Width and effective depth of the section, cm

r_w = Shear reinforcement ratio ($=100 A_w / bs$), A_w, and s represent the area of one shear reinforcement and the spacing of shear reinforcement, respectively, %.

P_y = Yield load, tf

δ_y = Yield displacement, mm

Mode 1 = Failure caused by the sudden development of new secondary shear crack in *2d*-zone or region (area between fixed end and *2d*-section). A section of the member at *2d* distance far from the fixed end is identified as *2d*-section.

Mode 2 = Failure caused by the widening of one of the diagonal shear crack comprising x-shaped crack in *d*-zone or region (area between fixed end and *d*-section). A section of the member at *d* distance far from the fixed end is identified as *d*-section.

Mode 3= Flexural failure of a member

X = No yielding of longitudinal reinforcements.

4. FAILURE MODES OF SPECIMENS

In the case of the specimens with r_w less than or equal to 0.15%, an existing crack located near *d*-section was gradually developed into the x-shaped diagonal cracks with the increased deflection. When the yielding of longitudinal reinforcements propagated upto *d*-section, the restoring force still did not decrease, but it decreased after the yielding region spread to *2d*-section by sudden development of a flexural crack there into the secondary diagonal crack. In the case of the specimen with r_w more than or equal to 0.15%, an existing crack located near the *d*-section was gradually developed into the x-shaped diagonal cracks with the increased deflection. When the yielding of longitudinal reinforcement at *d*-section took place, the restoring force tended to decrease. The final failure was caused by the rapid widening of one of the diagonal cracks comprising x-shaped shear cracks. The severe damage was concentrated in *d*-region. Cover concrete of longitudinal reinforcement in *d*-region was almost spalled out. On the contrary, the specimen with shear reinforcement ratio equal to 0.40%, did not show the significant loss of restoring force even upto $12\delta_y$ load reversals. The x-shaped cracks were present but the crack width was not so wide therefore the specimen was still capable of transferring the sufficient shear force. Instead, the flexural crack at the fixed end was wider showing the flexural failure as a failure mode.

5. SHEAR STRENGTH DEGRADATION PATTERN

All specimens except PM-6 which were initially designed to have higher shear capacity than the flexural capacity showed a clear shear failure when they were subjected to repeated load reversals. Of course, in the case of monotonic loading, the flexural failure can be expected unless the ratio of the shear capacity to flexural capacity is near to unity. This clarifies the shear degradation of reinforced concrete columns under reversed cyclic loading. The load-displacement envelopes are assumed to be horizontal (bilinear) at the yield load even after the yielding of longitudinal reinforcements under the reversed cyclic loading. So, the flexural capacity (P_u) is taken equal to the yield load (P_y).

Table 2 Final Shear Capacity at Failure

Spec. No.	Initial Shear Capacity, P_s (tf)	Shear Capacity Observed at Failure (tf)	$(P_s/P_u)_i$	Deformational Level at Shear Failure	μ_e*
PM-1	1.53	0.62	1.29	$5\delta_y$-1	4.0
PM-2	1.77	0.95	1.58	$6\delta_y$-1	5.0
PM-3	1.94	0.90	1.67	$7\delta_y$-1	6.0
PM-4	2.15	0.95	1.87	$8\delta_y$-1	7.0
PM-5	2.47	0.97	2.16	$9\delta_y$-1	8.0
PM-6	2.95		2.63	No Shear Failure	>12

*μ_e = Experimental Ductility Factor

The load measured at the final failure caused by the sudden widening of the shear crack is taken as the final shear capacity of reinforced concrete columns after severe degradation shown in Table 2 . So, we can get the two extreme ranges of shear capacity of a member, i.e., the initial calculated shear capacity and the final restoring force observed at the final failure from the experiment. The pattern of shear degradation during this process is still not formulated. However, in this research work the linear shear degradation with respect to r_w is assumed. Also, by incorporating the assumption that the shear capacity does not degrade upto first yielding of longitudinal reinforcement. The plot of the degrading shear capacity and the constant yield load normalised by the yield load versus the ductility can be shown in Fig.7. From the above nature of the shear degradation pattern, it was observed that the rate of shear degradation differed very slightly with each pier specimen. However, since this difference is very small , the linear degradation model with respect to r_w can be applied.

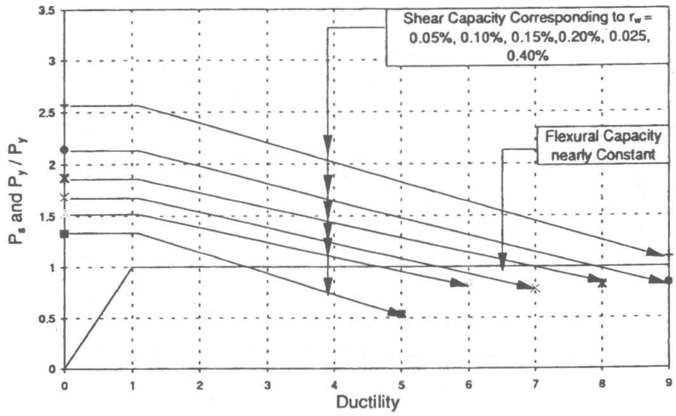

Fig. 7 Shear Strength Degradation Pattern

For the typical shear degradation pattern (Fig. 7), from the ratio of the initial shear capacity to the yield load, $(P_s/P_y)_i$, and the shear stiffness, *k (average of all shear degradation rates)*, the ductility factor of a specimen can be calculated as follows. From the shaded triangle in Fig. 8,

$$\left(\frac{P_s}{P_y}\right)_i - 1 = (\mu_{cal} - 1) * k \tag{1}$$

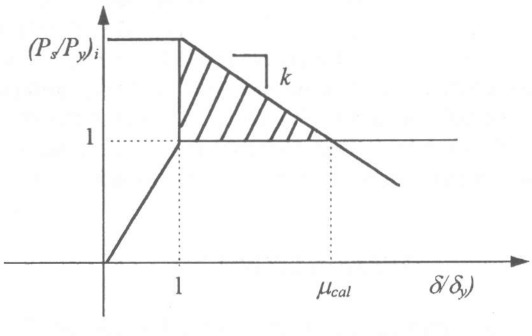

Fig. 8 P_s/P_y versus δ/δ_y Relationship

P_s is the shear capacity of a concrete member with the shear reinforcement which consists of the shear capacity carried by concrete (V_c) and the shear capacity carried by shear reinforcement (V_s), and can be computed by using any codal provisions. By assuming all material and sectional properties as constant P_s can be expressed as $P_s = A + B\, r_w$. P_y can be assumed to be equal to the flexural capacity of a member. So, the final form of the Eq. (1) is,

$$\mu_{cal} = 1 + \frac{1}{k}\left\{\left(\frac{A + Br_w}{P_y}\right)_i - 1\right\} \tag{2}$$

This gives the ductility factor of a member. This equation can be used to predict the shear failure of a member successfully. The accuracy of this evaluation is totally dependent upon the precision of k . A and B serve simply as constants for assumed constant material and sectional properties. The ductility of the above specimens calculated from Eq. (2) and that obtained from the experiment is compared in Fig. 9. The small discrepancy between these values is due to the existence of small variation or scatter on k with respect to r_w which is assumed to be constant in this research.

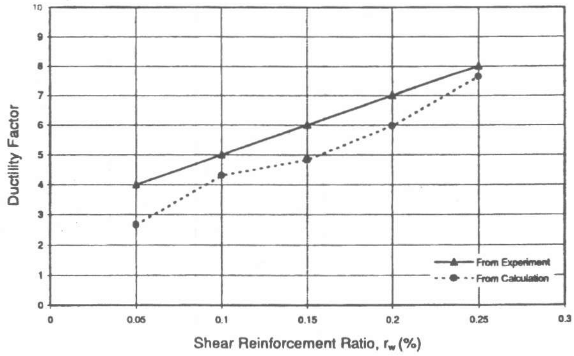

Fig. 9 Comparison of Experimental and Calculated Ductility Factor

6. CONCLUSIONS

1. It was seen that a RC pier which was designed to have much larger shear capacity than flexural capacity showed brittle post-yielding shear failure under load reversals.
2. The failure mode and cracking region was also affected by shear reinforcement ratio. It was seen that the region where the diagonal crack occurred was wider for smaller r_w. On the contrary, the cracks were limited to region near the fixed end if the shear strength was large, i.e. r_w is large.
3. Two types of shear failure mode were clearly seen. The failure mode of pier models with r_w equal to 0.15% (PM-1, PM-2, PM-3), and that with r_w more than 0.15% (PM-4, PM-5), were different.
4. By using the appropriate amount of r_w, the failure mode can be changed from the shear mode to the flexural mode even for the large load reversals at the moment of a strong earthquake.
5.The shear failure can be predicted from Eq. (2) but the accuracy is totally dependent on the accurate estimation of the shear degradation rate, k. Repeated load in post-yield region causes drastic loss of shear transfer contributed by concrete which is the main reason behind the shear strength degradation.

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