SEISMIC RESPONSE ANALYSIS OF RC STRUCTURES CONSIDERING STRENGTH DEGRADATION CAUSED BY CYCLIC LOADING

Y. Nogami[1], Y. Murono[2] and T. Sato[2]

[1] *Railway Technical Research Institute, Tokyo. Japan*
[2] *Dr., Eng., Railway Technical Research Institute, Tokyo. Japan*
Email: nogami@rtri.or.jp, murono@rtri.or.jp, ben@rtri.or.jp

ABSTRACT :

It is well known that the strength of reinforced concrete columns is degraded by cyclic loading, particularly in the post-peak region. When we examine the seismic safety of RC structures, it is important to conduct the dynamic response analysis considering the strength degradation by cyclic loading. In this study, we first develop a nonlinear dynamic hysteresis model, which can take into account the strength degradation. Secondary, we attempt to evaluate the effects of duration time of earthquake in the post-peak region through seismic response analysis by using the proposed model. As a result, we clarified that the post-peak behavior of reinforced concrete structures varies depending on the duration time of ground motion.

KEYWORDS: cyclic loading, strength degradation, nonlinear hysteresis model

1. INTRODUCTION

It is a common knowledge that the strength of reinforced concrete columns is degraded by cyclic loading, particularly in the post-peak region. A lot of static cyclic loading tests for RC members have been conducted. According to these tests, it has been pointed out that the seismic performance of RC members is affected by cyclic patterns [1]. When we examine the seismic safety of RC structures, it is important to conduct the dynamic response analysis considering the strength degradation by cyclic loading.

Various nonlinear hysteresis models for dynamic response analysis have been ever proposed. Most of these models, for example the Clough model [2] and the Takeda model [3], however, do not consider the effects of strength degradation. Therefore, these models are generally applicable to before-peak region.

In order to evaluate the post-peak behavior of RC structures, we develop a nonlinear hysteresis model that can take into account the strength degradation for dynamic response analysis. Furthermore, we attempt to evaluate the effects of cyclic characteristics of earthquake in the post-peak region by using the proposed model.

2. DEVELOPMENT OF NONLINEAR HYSTERESIS MODEL

2.1 Concept of Strength Degradation

Figure 1 shows the fundamental concept of strength degradation. We represent the strength degradation by moving a previous oriented point d_p at the-(i-1)-step to a new oriented point at the-(i)-step as shown in Fig. 1. We assume that the moving distance Δd is closely related to positive and negative reversal points at (i-1) step. The distance Δd, therefore, is obtained by Eqn. (2.1). In Eqn. (2.1), the parameter c represents the strength degradation degree. If the value of c is large, a new oriented point moves largely and then the strength decreases more.

$$d_n = d_p + \Delta d = d_p + \left(d_{\max} - d_{\min}\right)\cdot c \tag{2.1}$$

where,

d_n : new oriented point
d_p : previous oriented point

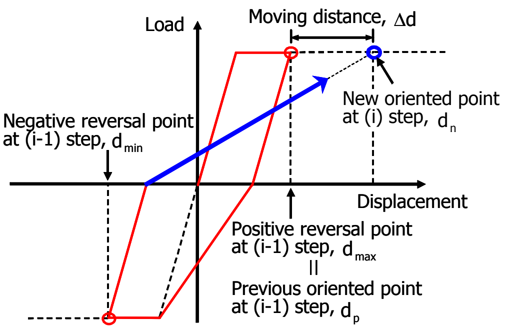


Fig. 1 Fundamental Concept of Strength Degradation

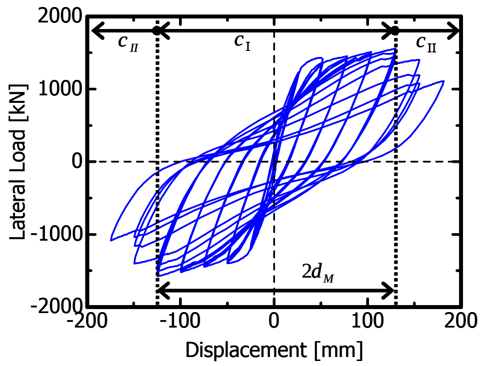


Fig. 2 Hysteresis Loop Under Static Loading

Δd : moving distance
$d_{\max}$: positive reversal point
$d_{\min}$: negative reversal point
c : parameter of the degree of strength degradation.

The shaking table tests have been ever conducted [5]. According to these experiments, after the amplitude of the response displacement exceeds the twice as much as displacement $2d_M$ of approximately maximum load sustained point, the damage of RC columns becomes sever and there is possibility of collapse [5]. These results mean that the strength is also degrading prominently beyond $2d_M$ in the static cyclic loading test as shown in Fig. 2. Therefore, we divide c into c_I and c_{II}. The parameter c_I is used until the amplitude of displacement exceeds $2d_M$. The parameter c_{II} is used beyond $2d_M$.

2.2 Hysteresis Rules of Newly Developed Model

In this section, we describe the detailed hysteresis rules about the proposed model. Figure 3 shows the example of hysteresis loop. Backbone curve is bilinear.

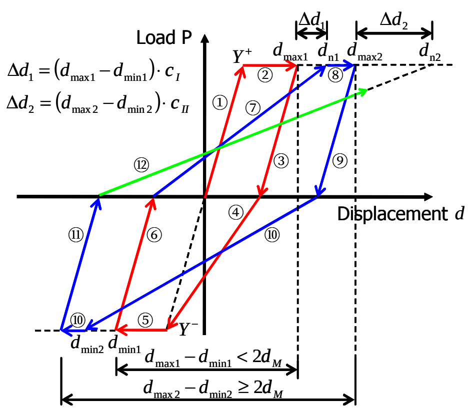


Fig. 3 Newly Developed Hysteresis Rules

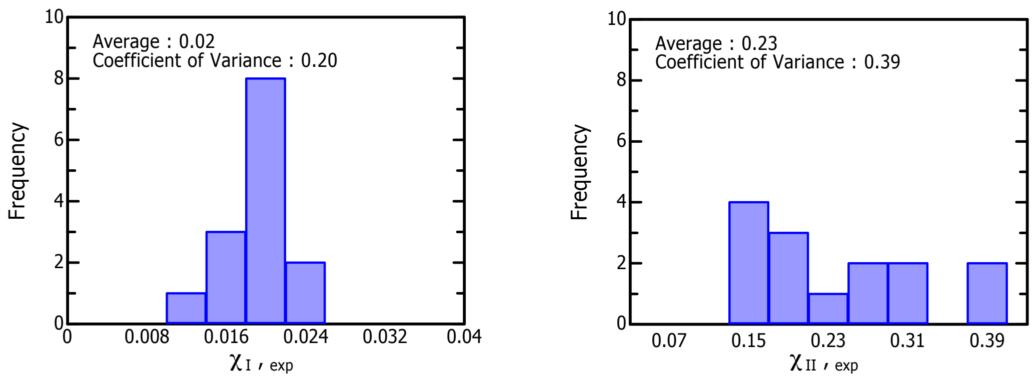
(i) If the response is below a positive yield point Y^+, the response traces the line ①.
(ii) The response shifts to the positive second slope beyond Y^+ (line ②).
(iii) If unloaded on the positive second slope, the load decreases to zero with unloading stiffness (line③).
(iv) After crossing $P = 0$, the response orients the negative yield point Y^- (line④).
(v) The response traces the negative second slope (line ⑤)
(vi) The same as (iii)
(vii) If the amplitude $(d_{\max 1} - d_{\min 1})$ of the previous loop (line ①–⑥) is less than $2d_M$, a new oriented point d_{n1} is calculated by Eqn. (2.1) with the parameter c_I (line ⑦).

Table 1 Properties of Existing Test Pieces

Test piece No.	Section size [mm]	Effective depth d [mm]	Shear span a [mm]	a/d	Longitudinal reinforcement*1)	Hoop reinforcement*1)	Axial stress s_0 [N/mm²]	Tensile reinforcement ratio p_t [%]	Hoop reinforcement ratio p_w [%]	Concrete strength f_c' [N/mm²]	Cyclic pattern*2)
H95-1	900*900	821	3300	4.0	SD345-D32	SD345-D13	3.87	1.07	0.28	30.0	3times / $1d_y$
H95-2									0.42	28.0	
H95-3						SD345-D16			0.66	31.4	
H96-11					SD390-D32	SD345-D13		0.95	0.42	29.2	
H96-12					SD390-D29				0.42	30.3	
H97-1					SD345-D32	SD345-D16		1.07	0.66	26.9	
H97-2									0.88	28.2	
H97-3						SD345-D19			1.27	29.2	
H97-4						SD345-D16			0.88	30.9	
H97-5							0.97		0.66	30.7	
H97-6							3.87		0.66	31.8	1times / $1d_y$
T97-1	800*800	728	3000	4.1	SD345-D25	SD345-D13	3.68	0.78	0.78	30.0	3times / $1d_y$
T97-2							0		0.78	32.6	
T97-3						SD345-D10	3.68		0.45	24.3	3times / $2d_y$

*1) SD(①) 345(②) - D32(③) ① deformed bar , ② nominal strength , ③ nominal diameter

*2) n times / m d_Y : number of cyclic times per m d_Y



(a) Parameter $c_{I,\exp}$ (b) Parameter $c_{II,\exp}$

Fig. 4 Frequency Distribution of Parameter c Obtained by Static Loading Tests

(viii) If the response reaches the second slope, traces line ⑧.
(ix) The same as (iii)
(x) The same as (vii)
(xi) The same as (iii)
(xii) If the amplitude $(d_{\max 2} - d_{\min 2})$ of the previous loop (line ⑦–⑪) is greater than $2d_M$, a new oriented point d_{n2} is calculated by Eqn. (2.1) with the parameter c_{II} (line ⑫).

2.3 Parameter Identification

It has been ever pointed out that the strength degradation degree depends on properties of RC members, such as tensile reinforcement ratio p_t and hoop reinforcement ratio p_w [6]. The proposed model controls the degradation degree by using the parameter c. Therefore, we attempt to formulate the parameter c by using the results of many static cyclic loading tests.

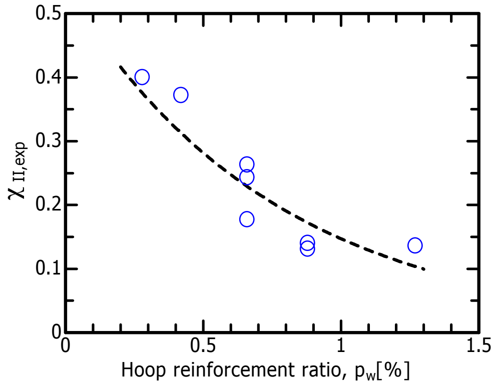


Fig. 5 Relationship between $c_{II,\exp}$ and p_w

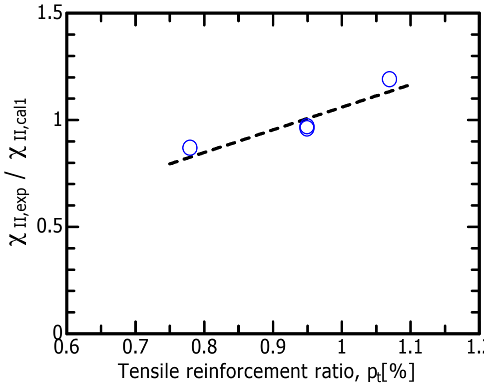


Fig. 6 Relationship between $c_{II,\exp} / c_{II,cal1}$ and p_w

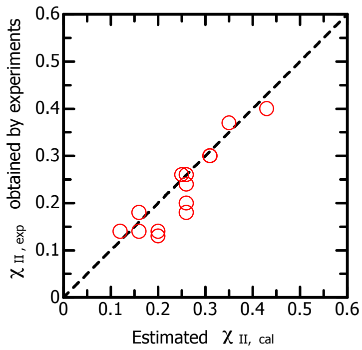


Fig. 7 Comparison of $c_{II,cal1}$ and $c_{II,\exp}$

The properties of existing test pieces are shown in Table 1 [7]. The parameter $c_{I,\exp}$ and $c_{II,\exp}$ obtained from the static loading tests are shown in Fig. 4. As shown in Fig. 4(a), the average and the variation coefficient of $c_{I,\exp}$ are very small. Therefore, we regard $c_{I,\exp}$ as zero. On the other hand, $c_{II,\exp}$ varies considerably depending on properties of RC members. Therefore, we formulate the regression equation among $c_{II,\exp}$, p_t, and p_w. Figure 5 shows the relationship between $c_{II,\exp}$ and p_w. The parameter $c_{II,\exp}$ decreases exponentially with increasing p_w. Its regression equation, therefore, can be expressed as follows:

$$c_{II,cal1} = 0.54 \cdot e^{-1.3 \cdot p_w} \tag{2.2}$$

where e is exponential function and, p_w, hoop reinforcement ratio [%]

Secondary, the relationship between $c_{II,\exp} / c_{II,cal1}$ and p_t is shown in Fig. 6. $c_{II,\exp} / c_{II,cal1}$ increases linearly with increasing p_t. Therefore, the regression equation of the relationship is expressed as follows:

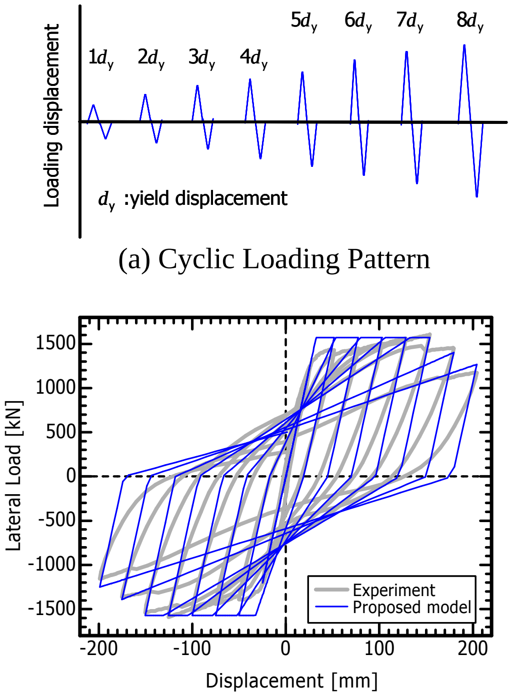
$$c_{II,\exp} / c_{II,cal1} = 1.06 \cdot p_t \tag{2.3}$$

where p_t is tensile reinforcement ratio [%]

Consequently, the parameter c_{II} is expressed by substituting Eqn. (2.2) into Eqn. (2.3) as follows:

$$c_{II} = 0.57 \cdot p_t \cdot e^{-1.3 \cdot p_w} \tag{2.4}$$

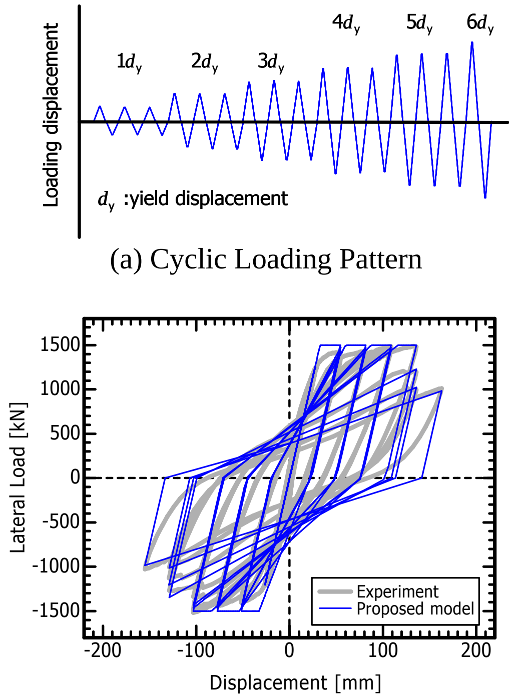
Figure 7 compares c_{II} with $c_{II,\exp}$. The parameter c_{II} is estimated by Eqn. (2.4) and The parameter $c_{II,\exp}$ is obtained from the static loading experiments. We can see that the parameter c_{II} agrees well with $c_{II,\exp}$. Therefore, the parameter c_{II} can be formulated by Eqn. (2.4) using p_t and p_w. In addition, this equation is applicable within the range of properties of RC members shown in Table 1.



(a) Cyclic Loading Pattern

(b) Load-Displacement Relationship

Fig. 8 Simulated Result of H97-6



(a) Cyclic Loading Pattern

(b) Load-Displacement Relationship

Fig. 9 Simulated Result of H97-1

3. VERIFICATION OF NEWLY DEVELOPED MODEL

In order to verify our newly model, we simulate the result of the static cyclic loading tests in Table 1. As the examples, the results of H97-6 and H97-1 are shown in Fig. 8 and Fig. 9. (a) indicates the cyclic loading pattern. While (b) expresses the load-displacement relationship.

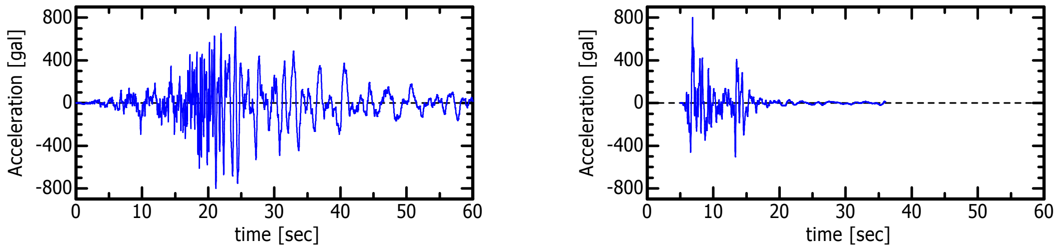
Paying attention to post-peak region, we see that this model can simulate the strength degradation by cyclic loading at both loading pattern. In addition, we have confirmed that this model can also represent the strength degradation at the other cases in Table 1.

4. SEISMIC RESPONSE ANALYSIS

In this chapter, we conduct the seismic response analysis of the pier, which can be expressed by a single-degree-of-freedom system, in order to evaluate the effects of cyclic characteristics of earthquake. Input ground motions are shown in Fig. 10 [8]. The wave of spectrum I targets near-land interplate earthquakes. The wave of Spectrum II expresses earthquakes produced by inland active faults. The parameters in this analysis are as follows:

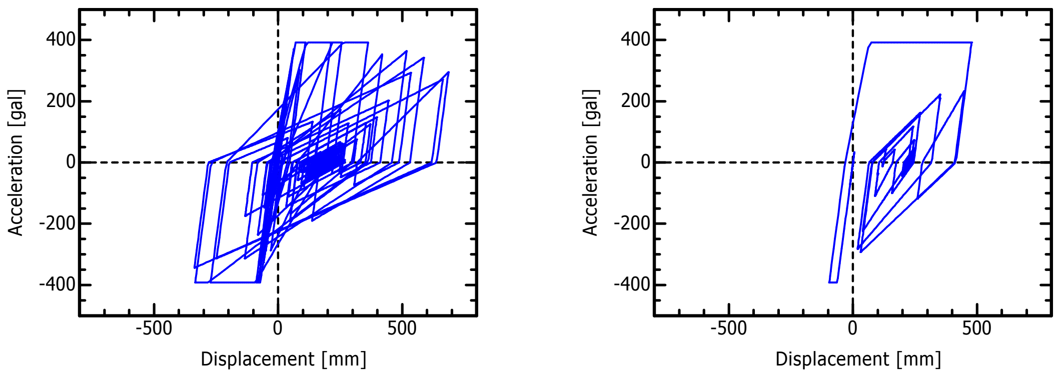
1) The yield seismic coefficient of pier is 0.4.
2) The equivalent natural period is 0.8.
3) The parameter c_I is zero and, c_{II}, 0.15.

The results of response analysis are shown in Fig. 11. In Fig. 11(a), the strength is decreased by a lot of cyclic pulses, and then the response displacement becomes large. On the other hand, in Fig. 11(b), the maximum response displacement is determined by a few pluses. The response displacement is, therefore, relatively small. As described above, the dynamic behavior of RC structures differs depending on the duration time of earthquake motion. In order to evaluate the seismic performance, it is necessary to consider the effects of cyclic characteristics of earthquake motions.



(a) Spectrum I (b) Spectrum II

Fig. 10 Time-history Waveforms of Input Ground Motion



(a) Spectrum I (b) Spectrum II

Fig. 11 Results of Response Analysis by Using The Proposed Model

5. CONCLUSIONS

In this study, we have developed a nonlinear dynamic hysteresis model for RC members, which can take into account the strength degradation caused by cyclic loading. Second, we have attempted to evaluate the effects of cyclic characteristics of earthquake through seismic response analysis by using this model. As a result, the following two major characteristics are clarified.

1) When the duration time of the earthquake motion is long, the strength degradation becomes more predominant and the deformation of the structure becomes very large.
2) When the duration time is short, only a few pulse controls the earthquake response characteristics and the deformation of the structure does not progress.

As mentioned above, the post-peak behavior of RC structures depends on cyclic characteristics of the ground motion. Consequently, in order to verify the seismic performance of RC structures, it is important to conduct the dynamic response analysis by using our developed model considering the strength degradation.

REFERENCES

[1] Hoshikuma. J., Unjoh. S., Kawashima. K. and Nagaya. K., "A Ductility Evaluation of Flexural Reinforced Concrete Columns Based on Loading History and Plastic Curvature," *Journal of Structural Engineering A*, Vol. 44-2, pp. 877-888, 1998

[2] Clough. R. W. and Johnston. S. B., "Effect of Stiffness Degradation on Earthquake Ductility Requirements," *Proceedings of The Japan Earthquake Engineering Symposium*, 1966

[3] Takeda. T., Sozen. M. A. and Nilsen. N. N. "Reinforced Concrete Response to Simulated Earthquakes," *Proceedings of The ASCE*, Structural Division, pp.2557-2573, 1970

[4] Umemura. K., Ichinose. T., Ohashi. K. and Maekawa. J. "Development of Restoring Characteristics Model Considering Strength Degradation for RC Members," *Proceedings of The Japan Concrete Institute*, Vol. 24, No2. pp.1147-1152, 2002

[5] Tadokoro. T., Tanimura. Y., Murono. Y., "Evaluation Method of Limit State of Collapse for Viaduct Columns Subjected to Repetitive Seismicity," *RTRI Report*, Vol. 22, No. 3, pp. 11-16, 2008

[6] Takiguchi. M., Watanabe. T., Tanimura. Y., "Experimental study on the decrease of the bearing capacity caused by cyclic loading tests," *Proceedings of Annual Conference of The Japan Society of Civil Engineers*, Vol. 56, pp. 742-743, 2001

[7] Takiguchi. M., Watanabe. Y., Sato. T., "Evaluation of Ductility for Reinforced Concrete Members," *RTRI Report*, Vol. 13, No. 4, pp. 9-14, 1999

[8] Railway Technical Research Institute, "Design Standard for Railway Structures (Seismic Design)," 1999