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Method for Checking Seismic Performance of Concrete Structures and its Effectiveness

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A method for evaluating seismic performance is proposed, and its versatility is verified by applying the method to space framed RC structures which were damaged at the Kobe earthquake. The check system consists of two steps. The necessity of seismic retrofitting is firstly judged by examining the ratio of shear to flexural capacity of seismic resistant members. The detailed secondary check with non-linear dynamic finite element analysis under estimated seismic actions is to be carried out if the first step judgment method is not applicable. Effectiveness of the proposed method is verified by comparing the analytical results and actual damage conditions of damaged RC space frames serving railway facilities.

Key words: seismic performance, seismic resistance, failure mechanism, space framed structure

1. Introduction

A number of civil engineering RC structures in Hanshin-Kobe area were seriously damaged by the "Hyogoken Nanbu Earthquake" in 1995. Taking this opportunity, the evaluation of seismic performance of existing RC structures has been raised as an urgent task for strengthening and/or seismic retrofitting and a tentative proposal has been published by Concrete Committee of JSCE (Japan Society of Civil Engineering). However, it can be hardly said whether the standard method for checking seismic performance of civil engineering structures has been established. Because it is desirable that the tentative proposal should be embodied as a manual or guideline applicable to each structural family having particular features through experience and piling up collation with past earthquake damage.

On this line, this paper presents a method for evaluating the seismic performance of continuous space framed RC structures and verifies its validity by comparing with actual damage condition of structures damaged under the former earthquake

2. Check of Seismic Performance

The check of the seismic performance is to judge whether the seismic retrofitting and/or strengthening of the structure concerned is required or not by comparing the actual seismic performance with specified one. For that purpose, the structure has to be modeled properly and analyzed under assumed seismic excitations that could be expected to occur at the site.

For the check of seismic performance of a lot of existing concrete structures, it might be more efficient to carry out the check step by step. First, necessity of seismic retrofitting is determined by a simple check followed up by the detailed secondary check, if the first judgement drops in gray zone.

In the first-stage check, a simpler method may be suitable with information that can be easily obtained such as construction year, adopted specification, structural type and dimension. In the second-stage check, it is to be verified whether the structure possesses the required seismic resistance against the assumed seismic actions by non-linear finite element analysis.

3. First-Stage Check

3.1 Check Method

For considering the past design code in which the shear strength of concrete was over-estimated, the failure mode could be classified into 3 categories; the shear mode with unstable propagation of diagonal crack, flexural failure accompanying yield of main reinforcement and the coupled post-yield shear.

In case of shear failure prior to the yield of reinforcement, the seismic retrofitting may be required since the entire collapse of structural system may occur with higher probability along the diagonal cracks caused by shear. Here, we suppose column members having side reinforcements. The maximum capacity in ultimate condition will be much greater than the load when main reinforcements yield. In order to check the ratio of loading when yielding to ultimate, 79 column members of space framed structures in JR line between Sumiyoshi and Rokkomichi have been investigated (Figure 1). Figure 1 shows that the yielding of member of structures concerned occurs when the 80% of ultimate load is carried in average.

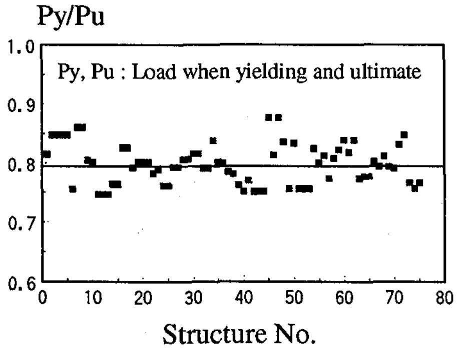

Figure 1 Ratio of Yielding to Ultimate Moment Capacity

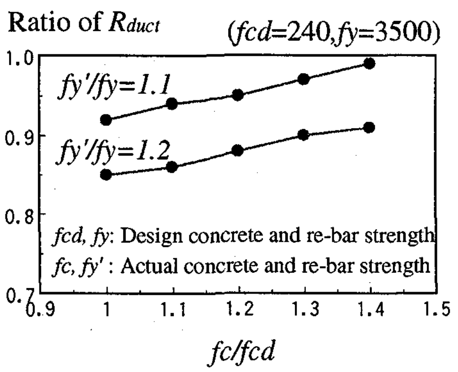

Figure 2 Variation of Shear to Moment Capacity with With Material Strengths

Moreover, considering that the actual strengths of concrete and reinforcement are larger than design strength in general, the probability of shear failure becomes much higher since the ratio shear-to-moment capacity ratio (R_{duct}) becomes smaller compared to that of original case with design strength values as shown in Figure 2. Therefore, it could be assumed that the shear failure prior to flexural yielding occurs if the ratio of shear-to-moment capacity satisfies Eq.(1). On the contrary, if the flexural failure would be certainly expected before the shear collapse of members the retrofitting may not be required since the load carrying system of entire structure may not collapse. It is known that the shear capacity of the concrete member subjected to reversed cyclic load tends to decrease after yielding of steel bars, but its decreasing rate is not yet clearly clarified. It is, therefore, assumed for safety that no collapse in shear occurs if the ratio satisfies Eq.(2). If the ratio satisfies neither Eq.(1) nor Eq.(2) the second-stage check with higher accuracy is to be conducted in order to check the retrofitting requirement.

$$R_{duct} = V a / M_u < 0.9 \tag{1}$$

$$R_{duct} = V a / M_u > 1.3 \tag{2}$$

where, M_u is flexural capacity of member derived from the beam theory, V is a shear capacity carried by concrete(V_c) and web reinforcement (V_s) and a is shear span length. For V_c, the following modified empirical equation is adopted [1].

$$V_c = 0.20 f_c'^{1/3}(0.75+1.4d/a) b_w d \beta_p \beta_n \beta_d \tag{3}$$

where, f_c' is compressive strength of concrete in MPa, d is effective depth, b_w is web width, a is shear span length, β_p, β_n and β_d are adjustment factors for reinforcement ratio in tensile zone, axial force and effective depth, respectively and given as follows.

$$\beta_p = (100P_c)^{1/3} \leq 1.5 \tag{4}$$

$$\beta_n = 1+M_o/M_d \leq 2 \text{ (in compressive force)} \tag{5}$$

$$= 1+2M_o/M_d \geq 0 \text{ (in tensile force)} \tag{6}$$

$$\beta_d = (100/d)^{1/4} \leq 1.5 \tag{7}$$

where, P_c is reinforcement ratio in tensile zone, M_o is counter flexural moment nullifying the stress induced by axial force at tension fiber of member and M_d is design flexural moment.

3.2 Results and Evaluation

In order to evaluate validity of defined criteria, the shear-to-moment capacity ratio of actually damaged RC space framed structures in JR Tokaido-Line between Sumiyosi and Rokkomichi (about 2 km) is computed and plotted in Figure 3. The characters S and M in the legend of Figure 3 stand for failure mode, i.e., shear and flexural mode, respectively and the following A, B and C stand for the damage levels. The detail damage level is described in Table 1.

Table 1. Description of damage level

Symbol	Description
SA	Drop and/or collapse of girder and slab
SB	Shear crack, yielding and/or cut-off web
SC	Diagonal shear crack
MA	Large buckling of longitudinal bars
MB	Spalling and buckling
MC	Spalling and small buckling

P_{max} in vertical axis of Figure 3 stands for maximum capacity, that is, maximum force that can be carried by the member and W_{slab} stands for average self-weight of the top slab supported by one column. As the dynamic force induced to the member in 1-DOF system can be expressed by product of mass and response acceleration, the maximum bearing response acceleration can be expressed as the ratio of the maximum capacity of member to its mass. Therefore, the ratio defined in vertical axis of Figure 3 represents maximum bearing response acceleration normalized by gravity acceleration. Although the weight supported by each column of the structure is not uniform in reality, its uniformity is assumed for simplicity.

As shown in Figure 3, seriously damaged structures are seen primarily in the shear failure zone in spite of their large maximum bearing response acceleration compared to structures in other zones. On the contrary, the structures with large shear-to-moment capacity ratio in flexural failure zone are free from serious damage in spite of their low maximum bearing response acceleration.

According to proposed first-stage check method, the number of failed structures in shear failure zone is 17 out of 20 including level "C". Therefore, the ratio of successful prediction is 85%. The number of structures damaged in flexural failure zone is 23 out of 31, but only one structure is classified into level "A". Unlike the case of shear failure, the entire collapse was avoidable in flexural failure if its damage level was not "A". In this point of view, the criterion seems to be effective and rational to check the seismic performance in the first-stage check.

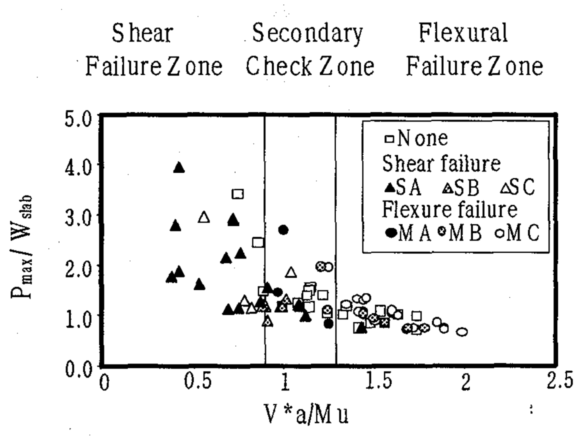

Figure 3 Failure Condition

4. Second-Stage Check

The structure in the secondary check zone that failed in shear and flexure were checked with respect to their seismic performance.

4.1 Seismic Actions

In order to check the seismic performance of structures, it is necessary to provide the earthquake that can be expected to occur at site. Here, the seismic actions as input wave were generated by inverse analysis of the wave observed at Kobe University near the site by using the universal program "SHAKE" in consideration of geological condition, dynamic properties of soil [2] and the path of earthquake wave. The predicted ground motions in both directions are shown in Figure 4

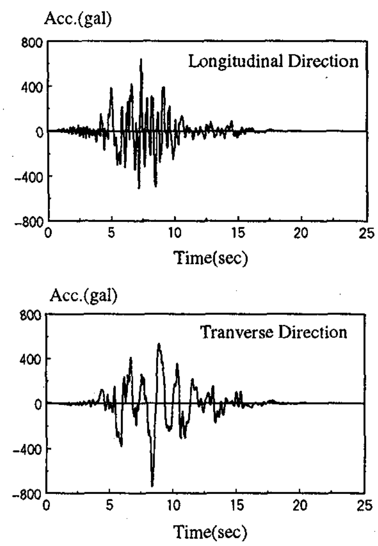

Figure 4 Estimated Seismic Action at Site

4.2 Structural Model

To evaluate seismic performance, a mechanical model for structure is indispensable. In general, the evaluation of seismic performance is conducted in each longitudinal and transverse direction individually under assumed seismic actions. However, the gravity center of top mass and the stiffness center are deviated since the stiffness of columns mutually differ due to their different column heights as shown in Figure 5. Hence, it may not be simple to determine the span length as an analytical unit in 2-dimensional extent in transverse direction especially. Therefore, the structure is modeled in 3 dimensional space.

In this analytical problem, the sectional force of column is our main concern. Then, the slab including girder is represented by elastic cubic solid elements, while the column is modeled to consist of several frame elements with RC (Reinforced Concrete) zone and PC (Plain Concrete) zone at each Gaussian integration point. Therefore, the different energy release rate can be taken into account under tensile load after cracking [3]. The foundation of structure is assumed to be fixed completely and input wave was given in terms of base acceleration.

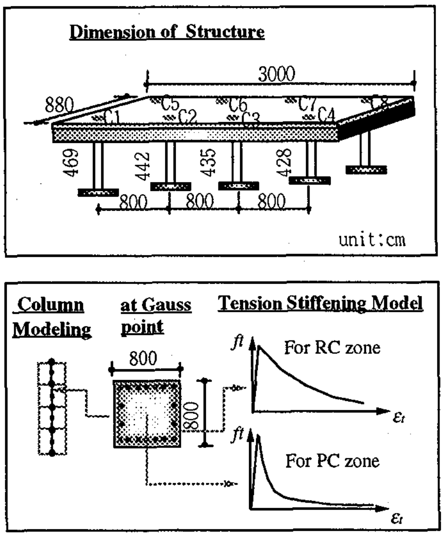

Figure 5 Structural Modeling

4.3 Analytical Model

The constitutive laws for RC consist of modeling of cracked concrete and reinforcing bars, which were derived from cyclic path-dependent relationship between averaged stress and averaged strain in control volume. Smeared crack constitutive laws adopted consist of tension stiffening model[4][5], elasto-plastic and continuum damage model for compression[6] and crack width dependent shear stiffness model. Once cracks are generated in the concrete, the stress-strain relations are modeled in the directions parallel as well as normal to cracks. Hence, the degradation of compressive stiffness after cracking as well as the stress transfer originating from bond effect of concrete and the reinforcing bar is taken into account. For the constitutive law of reinforcing bars, Kato model was adopted so that Bauschinger effect and strain hardening can be taken into account in fair accuracy [7]. All these constitutive laws stated above have been installed in finite element analysis program named COM3, Version 8.1, which has been developed in the concrete laboratory. The computation is to be terminated when normal strain reaches a pre-defined failure strain. Therefore, the flexural failure can be simulated with fair accuracy. On the other hand, the shear failure will be checked by empirical equation since the shear force is simply computed so as to satisfy equilibrium with flexure.

4.4 Shear Capacity

Prediction of shear capacity of RC members is still an open problem under dispute since there is no versatile empirical equation to predict it perfectly. Here, the equation given by Eq. (3) is used to predict it. It is known that the shear capacity of RC member subjected to orthogonal static loads decreases with increase of load intensity level in the other direction.

Analytical result predicting shear capacity of member subjected to multi-directional forces by using full three dimensional analysis is plotted in Figure 6, which shows that the shear capacity under bi-shear load can be approximated by the formula of ellipse. The shear capacity predicted by Eq (3) is plotted together, which is in fair agreement with analytical result. As seen in Eq (3), shear capacity varies according to axial force induced to the member denoted by β_n, that is, the failure envelope curve expands or contracts. If induced shear forces and axial force are given it can be plotted with its corresponding failure envelop curve as shown in Figure 6. Hence, the shear failure can be judged whether the induced force marked by square lies on the failure envelop curve or not at each time step, which can be readily accomplished by comparing the norm of induced shear force vector(|S|) with that of corresponding shear capacity vector(|V|).

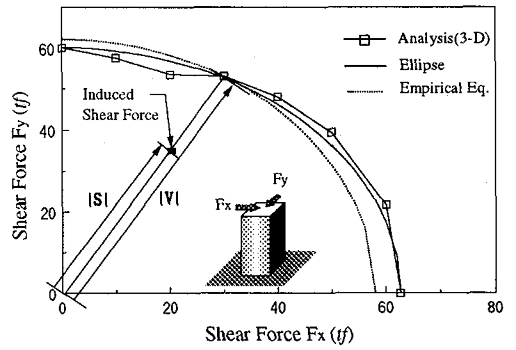

Figure 6 Shear Capacity of Member subjected to Multi-Directional Forces

4.5 Analytical Results : Failure in shear

In order to examine the seismic performance of structure in terms of shear failure, one structure in secondary check zone was analyzed. The minimum shear-to-moment capacity ratio was computed in C3 in longitudinal direction. The damage condition was classified into "SA". The detailed external dimension of structure is shown in Figure 5. The computation was normally terminated reaching the final step, which means implicitly that no flexural failure occurs in the structure. Therefore, the shear failure is to be checked by proposed method.

As can be expected from minimum shear-to-moment capacity ratio, shear failure occurs in column C3 (Figure 7).

The shear forces induced to column C3 in time domain are shown in Figure 8, which shows that the shear force in longitudinal direction is readily amplified compared to that of transverse direction so that it may lead to entire failure of structure concerned in longitudinal direction. This fact can be confirmed by interaction diagram between the induced shear force up to failure and its corresponding failure envelop curve shown in Figure 9. It describes that the longitudinal seismic action is dominant to the failure since the contribution of transverse one is negligible at the failure moment. Figure 10 is a hysteric curve of response acceleration and displacement of C3 in longitudinal direction. It can be confirmed that the structure fails in shear with its low ductility just after overall yielding of structure. Therefore, the retrofitting improving the ductility should be done in order to attain seismic resistance against assumed seismic actions.

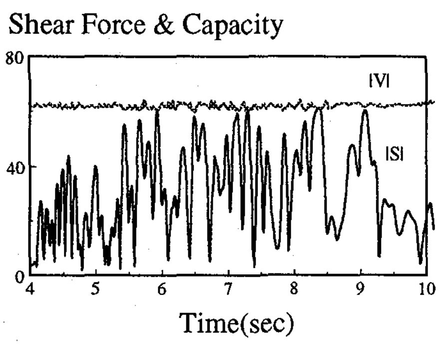

Figure 7 Norm of Shear Force(|S|) & Capacity(|V|) in C3

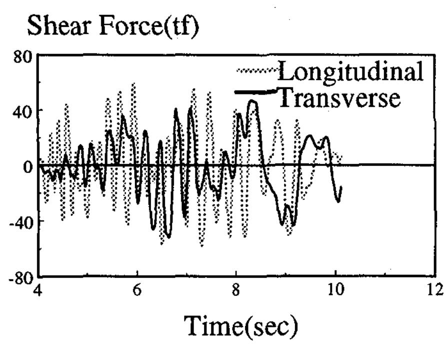

Figure 8 Shear Forces Induced to Column C3

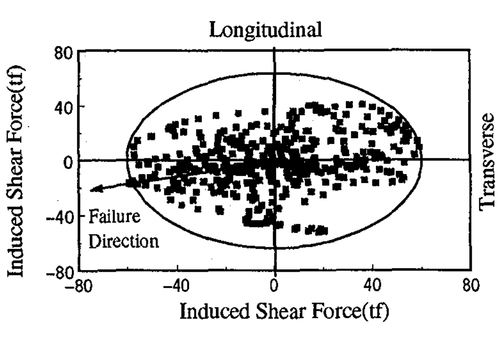

Figure 9 Interaction Diagram of Shear Force and Failure Envelop Curve

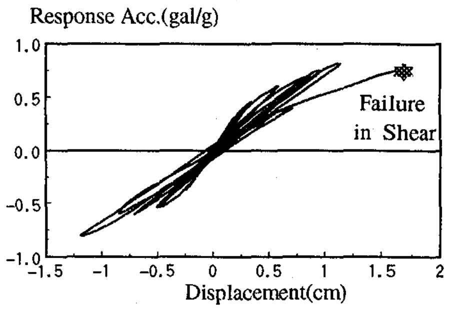

Figure 10 Hysteric Curve of Response Acceleration and Displacement

4.6 Analytical Results : Failure in flexure

The structure with damage "MA" in secondary check zone is analyzed to check the seismic performance. The shear-to-moment capacity ratio is 1.09.

Detailed dimension of structure concerned is as follows. The length and width of top slab are 36.62m and 8.8m respectively. The heights of columns represented by C1, C2, C3 and C4 in Figure 5 are 775, 786, 789 and 794cm, respectively and sectional dimension of exterior column, C1 and C4, is 80cm. 100cm is, however, utilized for interior columns C2 and C3 since the interior span length is about twice of exterior span one. The computation was terminated as flexural failure when the normal strain of column reached the defined failure criteria. Figure 11 shows the response displacements of top of columns in both directions. Relative large displacements are induced in transversal direction. As shown in this figure, the longitudinal displacement is identical due to alignment of columns. On the contrary, the transversal displacements of columns are linearly distributed with maximum displacement in column C4 satisfying torsional self-equilibrium, which will be governed by stiffness of exterior columns, C1 and C4. It can be, therefore, easily expected that the most severe damage will be introduced to the column C4 if the stiffness of columns is identical. The flexural failure, however, occurs in column C3 in transverse direction since larger moment is induced due to its relatively larger stiffness and response displacement. It can be confirmed by Figure 12 showing the induced moments to columns in time domain that large moment is introduced to the column C3 in transverse direction and leads to the failure of structure. Figure 13 is interaction diagram between introduced moment and capacity, which describes that the failure direction of structure is close to transverse direction, but the contribution of longitudinal moment to failure is not negligible. Figure 14 is hysteric curve of response acceleration and response displacement of C3 in transverse direction, which shows the flexural failure with insufficient ductility in C3.

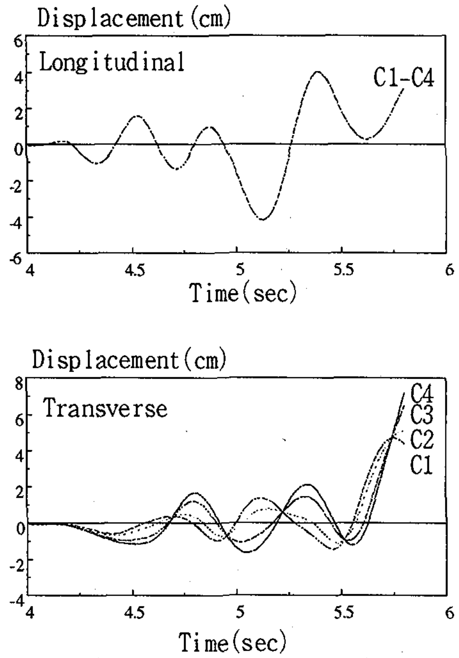

Figure 11 Response Displacement

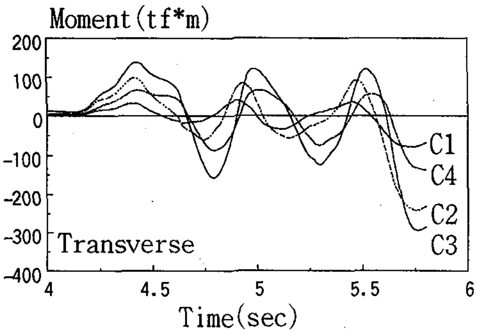

Figure 12 Induced Moment in Transverse Direction

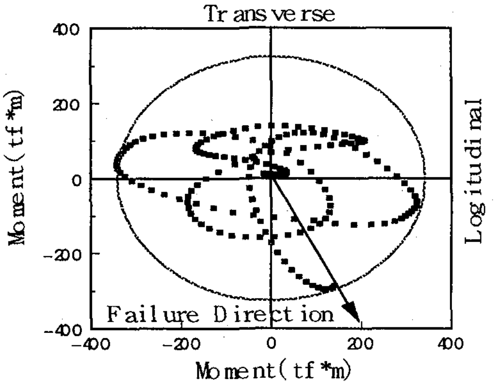

Figure 13 Interaction Diagram of Moment & Failure Envelop Curve

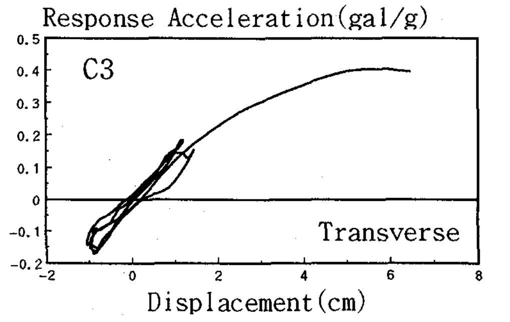

Figure 14 Hysteric Curve of Response Acceleration and Response Displacement in C3

5. Conclusion

For the sake of the seismic performance evaluation, the method consisting of 2 stages, i.e., the first-stage check for simplicity and the second-stage check in detail was proposed. In the first-stage check, the shear-to-moment capacity ratio of the member was adopted as the index to judge whether the seismic retrofitting or the second-stage check is required or not. The detailed structural modeling and the check method for shear failure of member subjected to multi-directional shear forces have been introduced in the second-stage check method. The effectiveness of proposed method has been verified by comparing the analytical results with the actual damage condition of continuous RC space framed structures in JR line.

References

1) Okamura,H, and Higai,T. (1980). "Proposed Design Equation for Shear Strength of Reinforced Concrete Beams without Web Reinforcement", Proc. of JSCE, **300**, 131-141
2) Kokusho, T. (1982). *A Study on the Dynamic Properties and Nonlinear Seismic Response of Ground*, Doctoral dissertation, The University of Tokyo, Tokyo, Japan.
3) An,X. (1996). *Failure Analysis and Evaluation of Seismic Performance for Reinforced Concrete Shear*, Doctoral dissertation, The University of Tokyo, Tokyo, Japan
4) Okamura, H., Maekawa, K., and Sivasubramanyyam, S. (1985) "Verification of Modeling for Reinforced Concrete Finite Element, Finite Element Analysis of Reinforced Concrete Structures, *ASCE*, pp. 528-543
5) Shima, H., Chou, L. and Okamura, H. "Micro and Macro Models for Bond Behavior in Reinforced Concrete" *Journal of the faculty of engineering, The University of Tokyo(B)*, Vol. 39, No2, pp. 133-194, Tokyo, Japan
6) Kato,B. (1979). "Mechanical Properties of Steel under Load Cycles Idealizing Seismic Actions", *CEB Bulletin D'Information*,**131**,7-27.