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Analysis of Reinforced Concrete Members Subjected to Cyclic Loads

Analyse d'éléments en béton armé soumis à des charges cycliques

Berechnung von Stahlbetongliedern unter zyklischer Belastung

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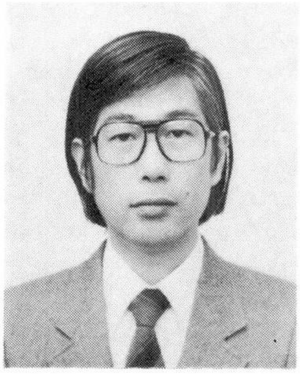


Norio Inoue, born 1947, got his Doctor of Engineering Degree from the University of Tokyo. For sixteen years he was involved in special dynamic and static problems of structures in Muto Institute of Structural Mechanics. Norio Inoue, now in Kajima Institute of Construction Technology, is specializing in nonlinear analysis of reinforced concrete structures by Finite Element Method.

SUMMARY

This study shows the analytical examples of reinforced concrete members subjected to cyclic loads like seismic forces. First,a shear wall and a panel were analyzed by macroscopic approaches based on the average stress-strain relationship. Next,columns were analyzed by a microscopic approach based on the characteristics of each element like concrete, reinforcement and bond. From these analytical studies, the applicability and future problems of cyclic analyses are described.

RÉSUMÉ

Cette étude montre des exemples analytiques d'éléments en béton armé soumis à des charges cycliques telles que les séismes. Une paroi de cisaillement et un panneau ont été analysés par approches macroscopiques basées sur la relation contrainte-déformation moyenne. Des colonnes ont été étudiées par une approche microscopique basée sur les caractéristiques de chaque élément, tel que béton, armature et adhérence. La valeur de ces analyses cycliques ainsi que les problèmes à étudier sont décrits.

ZUSAMMENFASSUNG

Diese Arbeit zeigt einige Berechnungsbeispiele für Stahlbetonbauteile unter wiederholter Belastung wie zum Beispiel Erdbeben. Zunächst werden eine Schubwand und eine Scheibe mit gemittelten Spannungs-Verformungsbeziehungen analysiert. Danach werden Stützen berechnet mit einem Mikromodell, das die charakteristischen Eigenschaften von Beton, Bewehrung und Verbund enthält. Nach diesen Beispielen werden die allgemeine Anwendbarkeit und zukünftige Probleme im Zusammenhang mit wiederholter Belastung beschrieben.

1. INTRODUCTION

Recently many researches have been performed concerning nonlinear analyses of reinforced concrete members and structures by using Finite Element Analysis [1] [2]. As to the loadings, several kinds of loads are considered in these studies. Among them the seismic forces are very important in the region like Japan where seismic intensity is very large.

Aseismic nonlinear behavior of structures is governed by the cyclic characteristics of members besides the monotonic ones. Such cyclic behaviors can be described generally by the envelope curve and the reversing loop of the load versus the displacement relationships. Therefore when the cyclic characteristics are investigated by static Finite Element Analysis, it is desirable to pursue the cyclic loops as they are. But the cyclic analyses are very complicated and need much computational time. So an alternative approach is often adopted in which reversing loops are assumed by referring to many experimental results and the envelope curve is obtained by analyses. In such a case it is necessary to make sure that the cyclic characteristics can be reflected to the assumed element properties and obtained results.

Concerning the analytical method itself there are two types of approaches, that is, a macroscopic one based on average stress vs. average strain relationship like Vecchio and Collins [3], and a microscopic one based on the characteristics of each element itself like concrete, reinforcement and bond.

In view of these circumstances, this study shows the applicability and future problems of cyclic analyses by presenting several analytical examples of both macroscopic and microscopic approaches.

First as an example to obtain envelope curves, a monotonic analysis is presented for a shear wall subjected to cyclic lateral forces by using a two-dimensional macroscopic approach. Next the cyclic analyses are shown for a reinforced concrete panel subjected to cyclic pure shear forces. In this case the same macroscopic method was applied with several additional cyclic rules. Finally the cyclic analyses are shown for a column subjected to one-dimensional cyclic lateral forces and a column subjected to two-dimensional lateral forces with circular displacements by using a three-dimensional microscopic approach.

2. MONOTONIC ANALYSIS OF SHEAR WALL

Monotonic analyses were performed for a shear wall subjected to cyclic lateral forces in order to investigate the envelope curve of hysteretic loops.

2.1 Analytical Method

The analytical method is based on the average stress vs. average strain relationship proposed by Vecchio and Collins [3] with some modification for being used with a Finite Element Method. This approach is very effectual for a shear wall in which many reinforcements are arranged uniformly. The detail was presented in the paper [4]. The main points and revisions are described below.

- The average strain of concrete is equal to that of reinforcement.

- The direction of principal stress for cracked concrete coincides with that of the principal strain.

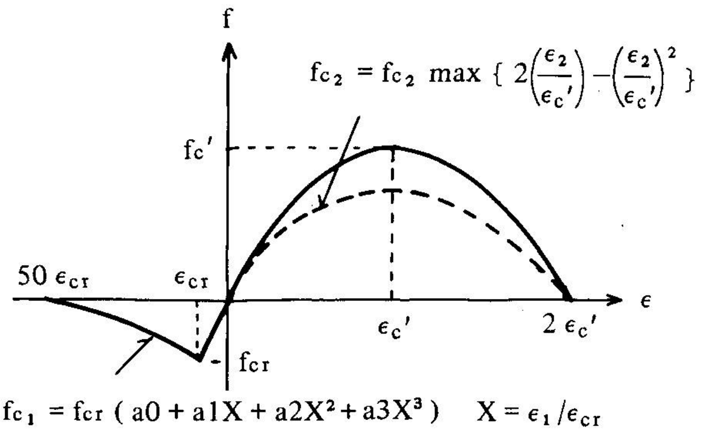
- The compressive principal stress vs. strain relationship of the cracked concrete is represented by Eq. (1), (2) and Fig. 1.

- The tensile principal stress vs. strain relationship is represented by a third order function like Fig. 1 in which the stress becomes zero at 50 times of cracking strain ϵ_{cr}.

- A stiffness matrix is made by the principal stress strain relationship with addition of an adequate shear rigidity like Eq. (3). The solution is performed by an iteration method for total stress and strain.

$$fc_2 = fc_2\max\left\{2\left(\frac{\epsilon_2}{\epsilon_c'}\right) - \left(\frac{\epsilon_2}{\epsilon_c'}\right)^2\right\} \quad \ldots\ldots\ldots\ldots (1)$$

$$\frac{fc_2\max}{fc'} = \frac{1}{0.8 - 0.34\dfrac{\epsilon_1}{\epsilon_c'}} \leq 1.0 \quad \ldots\ldots (2)$$



fc_1, fc_2 : Tensile and compressive principal stress
ϵ_1, ϵ_2 : Tensile and compressive principal strain
fcr, fc' : Uniaxial tensile and compressive strength
ϵ_{cr}, ϵ_c' : Strain at the tensile and compressive strength

Fig.1 Stress strain relationship of concrete

$$\begin{Bmatrix} fc_1 \\ fc_2 \\ \tau \end{Bmatrix} = \begin{bmatrix} E_1 & 0 & 0 \\ & E_2 & 0 \\ \text{sym} & & G \end{bmatrix} \begin{Bmatrix} \epsilon_1 \\ \epsilon_2 \\ \gamma \end{Bmatrix} \quad \ldots\ldots\ldots\ldots (3)$$

E_1 : Secant modulus in tension
E_2 : Secant modulus in compression
G : Shear modulus

2.2 Test Specimen and Analytical Model

The analyzed test specimen is a shear wall which was tested by Shiohara and Aoyama et al. shown in Fig. 2 [5]. The shear span-to-depth ratio is 0.5 and the lateral reinforcement ratio is 0.85%. Several cyclic lateral forces were loaded with a constant axial force. In this test relative displacements in three directions were measured at several portions by LVDTs. From these data the average strains in each zone could be calculated. The mesh layout used in this analysis is presented in Fig. 3.

2.3 Analytical Results

The obtained load vs. displacement relationship is shown in Fig. 4. The analysis could represent well the envelope curve of the observed one.

The Mohr's circles of average strains for three zones (A, B and C) are shown in Fig. 5. When paying attention to the diameter of the circles, the analytical results represented well the observed ones up to the ultimate stress. This means that this monotonic analysis can predict well the shear strain and accordingly can represent well the load vs. displacement relationship. But as to the center of the circle the experimental results shifted to the tension side compared with the analytical ones. Considering that the center of a Mohr's circle means the dilatancy of a panel, it is thought that cyclic loading caused an additional dilatancy compared with monotonic loading. Therefore to investigate such a phenomenon the load vs. the sum of normal strains ($\varepsilon_x + \varepsilon_y$) in the zone B is presented in Fig. 6. It is noticed that the analytical strains have good agreement with the experimental ones up to the level of 980 kN before the first reversing step. But just after reversing the experimental strain is larger than analytical one. Such tendency was also observed for the second reversing step of 1560 kN. Then the residual strains were estimated by the test results and added to the analytical results. That is, between the load 980 kN and 1560 kN, the residual strain (0.70×10^{-3}) observed after the first cycle was added to the monotonic results at each step. Beyond the load 1560 kN the observed residual strain (2.35×10^{-3}) after the second cycle was added. This procedure caused a good agreement.

From these studies it is considered that the proposed monotonic analysis can predict the shear strain and the envelop curve of load vs. displacement relationship and that residual strains should be considered to estimate the normal strains and dilatancy.

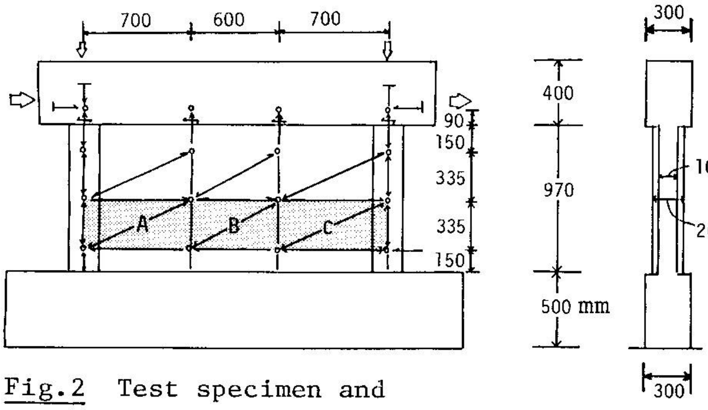


Fig.2 Test specimen and measuring points of displacement [5]

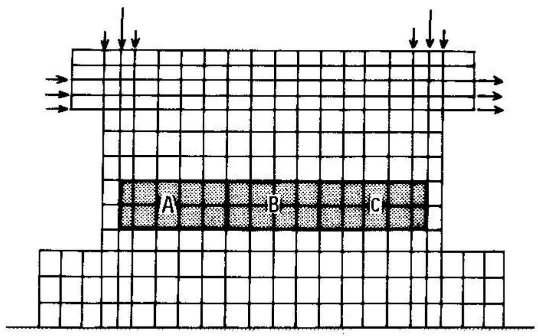


Fig.3 Mesh layout

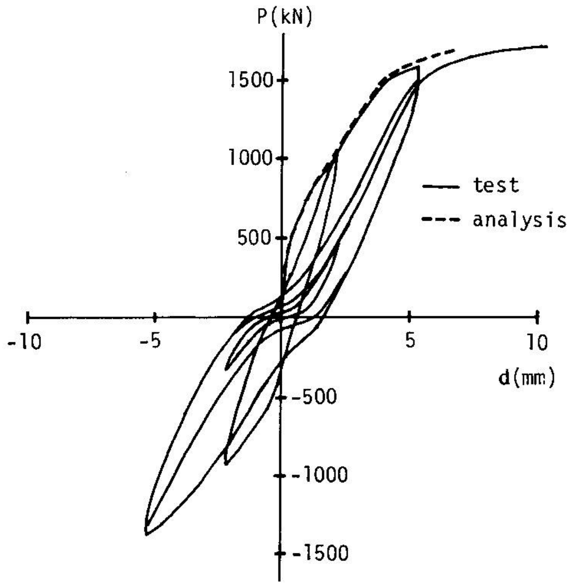


Fig.4 Load - displacement relationship

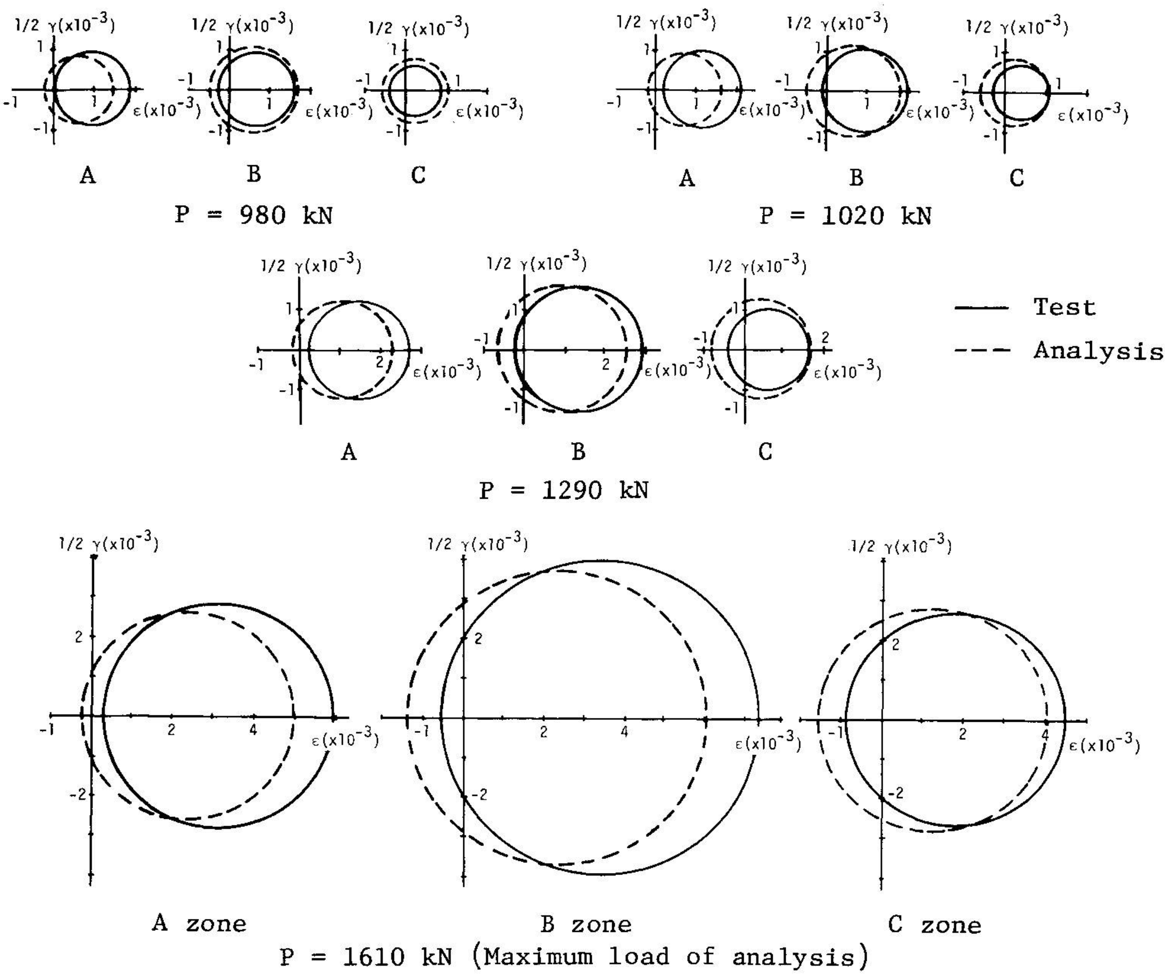


Fig.5 Mohr's circle of strain

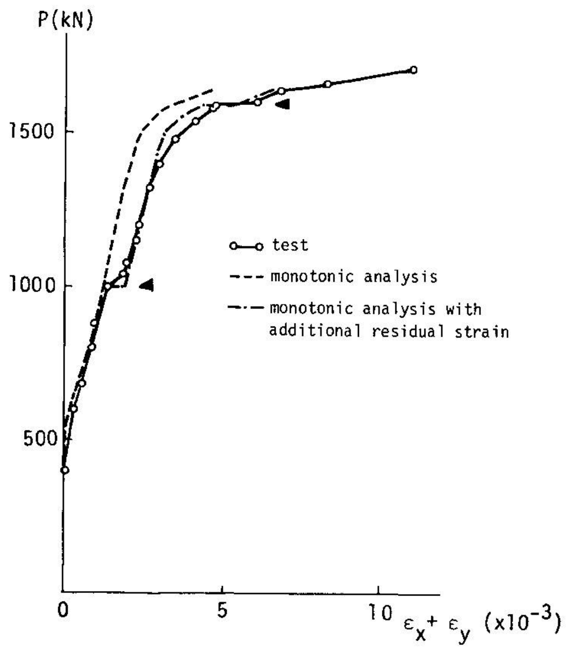


Fig.6 Load - sum of normal strains ($\varepsilon_x + \varepsilon_y$) relationship

3. CYCLIC ANALYSIS OF PANEL

Ohmori, Tsubota, Kurihara and the author et al. have been conducting a series of studies to clarify the characteristics of reinforced concrete panels subjected to in-plane forces at Kajima Institute of Construction Technology. An experimental and analytical study has already been presented concerning panels subjected to cyclic pure shear forces [6]. The study herein is aimed to propose a simple analytical method which can be applied in the design of actual complicated structures.

3.1 Panel Test

Several reinforced concrete panels have already been tested. The test specimens were 2,500 mm square and 140 mm thick and deformed rebars were arranged in two layers in two orthogonal directions as shown in Fig. 7. The percentage of reinforcing steel was varied from 0.8 to 2.0%, but always with the ratio of transverse reinforcement equal to that of longitudinal reinforcement.

The test panels were loaded by a newly developed testing facility as shown in Fig. 8. This facility can apply any combination of in-plane shear and normal forces to the test panels by automatically controlled 24 closed loop hydraulic actuators and a network of links. All tests presented in this paper were conducted in reversed cyclic in-plane pure shear.

The total average shear stress applied to the test panel was evaluated from measured data of 24 load cells attached to the hydraulic actuators. For evaluating the average strains of the test panel, 6 LVDTs were attached to the surface of the specimen.

The measured data from 6 LVDTs were reduced to average strains in the longitudinal, transverse and two diagonal directions. These strains were then used to define a Mohr's circle of average strains for each load stage and finally one optimum circle passing through all four measured points as close as possible was determined. Having defined the strain circle, the remaining strain parameters such as principal compressive strain, principal tensile strain and shear strain, could be determined.

Average stresses of reinforcement were determined from the measured strains in the longitudinal and transverse directions. Using these reinforcement stresses, the average concrete stresses in the longitudinal and transverse directions were calculated from equilibrium equations. By knowing the applied shear stress

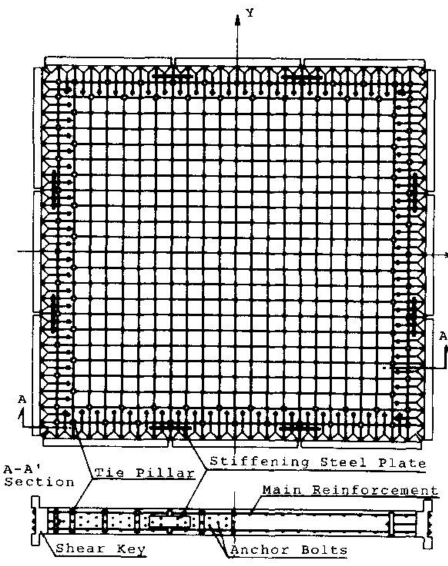


Fig.7 Test specimen

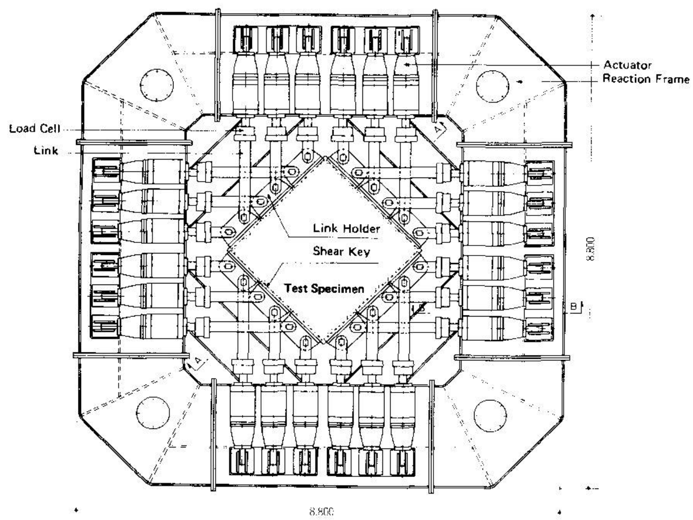


Fig.8 Testing facility

acting on the test panels the remaining concrete stress parameters could be determined. Thus, it was possible to determine concrete strain and stress circles at each load stage [3].

For example, the shear stress-strain relationship and the principal stress-strain relationship of concrete obtained from a typical panel (KP6) are shown in Fig. 13(a) and Fig. 14 (a) respectively. The reinforcement ratio of this specimen was 2.0% and its failure mode was a sliding shear failure of concrete prior to yielding of reinforcement. Among these test results, the principal stress-strain relationship of concrete is most important and from this relationship some significant characteristics of cracked concrete subjected to reversed cyclic in-plane shear stresses can be found. Namely, the decrease of stiffness during the unloading stage, the successive slip phenomena and the restoration of compressive stiffness at reloading stage are obvious.

In these figures, the predicted responses by Vecchio and Collins' model for monotonic loading are also indicated for reference. Here, the cracking strength of concrete was assumed to be the observed values.

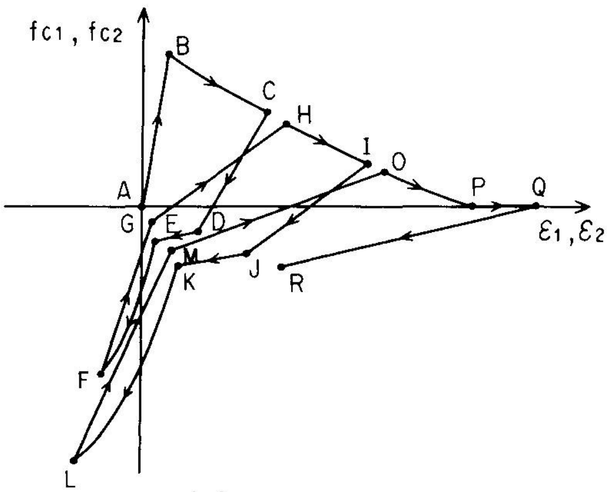
3.2 Analytical Method

3.2.1 Principal Stress-Strain Relationship for Cracked Concrete

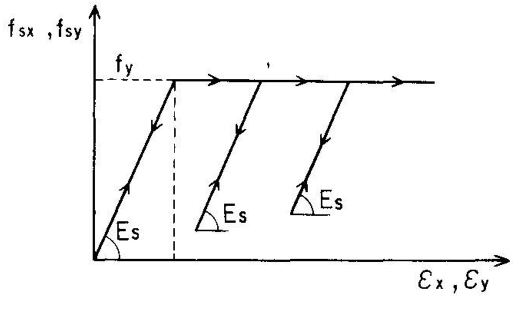
Fig. 9(a) shows a schematic hysteretic loop model of the principal stress-strain relationship of cracked concrete. The points C, F, I, L and Q correspond to the maximum applied shear stress stages in each loading cycle and the points A, D, G, J, M and R correspond to the loading step when applied shear stress is equal to zero.

The outlines of the proposed hysteretic loop model for cracked concrete are described below.

- At first, the residual stresses and strains of cracked concrete are defined as the stresses and strains when the applied shear stress is equal to zero (D, G, J, M, and R in Fig. 9(a)). This consideration is the main point of the analysis as the importance of residual strains has been presented in the aforementioned monotonic analysis of a shear wall. The magnitude of the residual strain is considered to depend on the maximum strain at the start of unloading. In Fig. 10 relationships between the measured residual strains and measured maximum strains are shown. From these data, an approximate function to define the magnitude of the residual strain used in the proposed analytical model was assumed as the curve indicated in Fig. 10. Since the residual stress and strain are known, the unloading path (CD, FG, IJ, LM and QR in Fig. 9(a))



(a) Concrete



(b) Reinforcement

Fig.9 Schematic hysteretic loop of principal stress - strain relationship can be defined.

- Next, the stiffness of cracked concrete in the slip region must be defined. Fig. 11 shows the relationship between the decreasing ratio (β) of the slip stiffness to the elastic stiffness of concrete and the residual principal strain with an approximate function. Since the behaviour in this slip region is unstable, the measured data displays considerable dispersion.

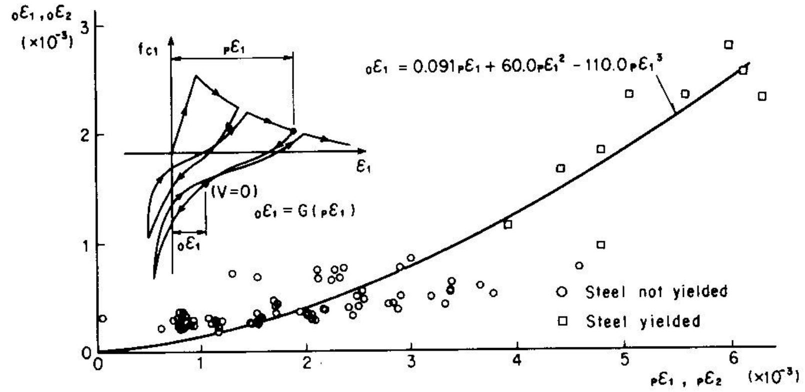


Fig.10 Residual principal strain - maximum principal strain relationship

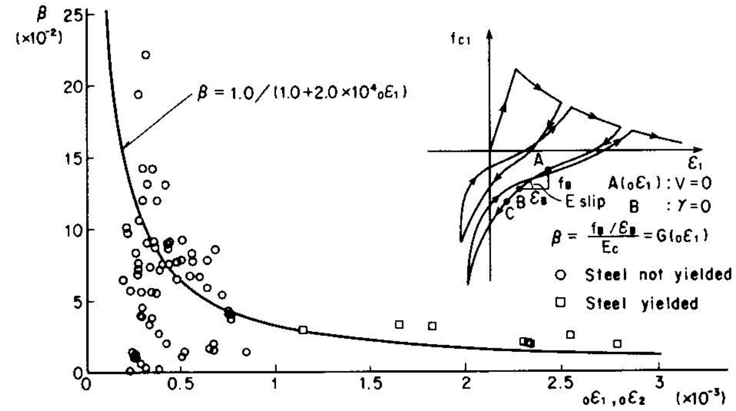


Fig.11 Slip stiffness ratio (β=Eslip/Ec) - residual principal strain relationship

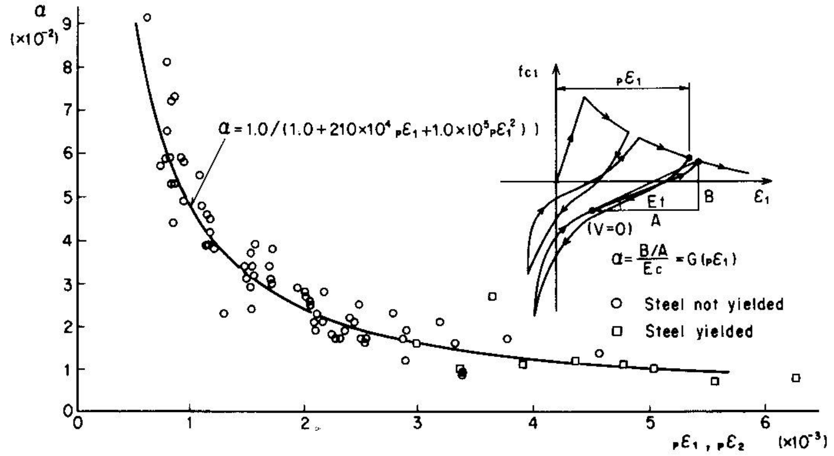


Fig.12 Restoring stiffness ratio (α=Et/Ec) - maximum principal strain relationship

- The restoring tensile stiffness (Et) of cracked concrete from the residual stress state to reloading tensile stress state is defined as a function of the maximum tensile strain experienced in the former loading cycle. Fig. 12 shows the relationship between the ratio (α) of restoring tensile stiffness to elastic stiffness and maximum tensile strain with an approximate function.

- The original principal compressive and tensile stress-strain relationships of cracked concrete proposed by Vecchio and Collins were adopted for evaluating the envelope stiffness of cracked concrete during the loading stage (EF and KL in compressive region and BC, HI and OPQ in tensile region shown in Fig. 9(a)). In the proposed analytical model, however, appropriate coordinate transformations of the original stress-strain relationships are performed in order to satisfy the continuous conditions of the hysteretic loop at the junction points (E and K in Fig. 9(a)).

3.2.2 Stress-Strain Relationship for Steel

As for the stress-strain relationship of reinforcing steel, the usual bilinear uni-axial stress-strain relationship was adopted as shown in Fig. 9(b) .

3.2.3 Numerical Solution Technique

By using the aforementioned stress-strain relationships of concrete and steel, equilibrium equations and compatibility requirements, nonlinear simultaneous equations of unknown average stress and strain parameters were derived. The Newton-Raphson iteration technique was adopted to solve these nonlinear equations. The solutions were continuously obtained step by step at each loading stage.

3.3 Analytical Results

The proposed analytical model was adopted to predict the response of the specimens tested in this investigation. Among the obtained analytical results, the shear stress-strain relationship for specimen KP6, is presented in Fig. 13 (b). As shown in this figure, the predicted envelope curve represents the observed behavior well. However, the predicted ultimate shear strength is larger than the observed one. Such a discrepancy is also found in the predicted result of monotonic loading as shown in Fig. 13 (a). This means that more precise studies are necessary to evaluate more accurate envelope curves under not only reversed cyclic loading but also monotonic loading.

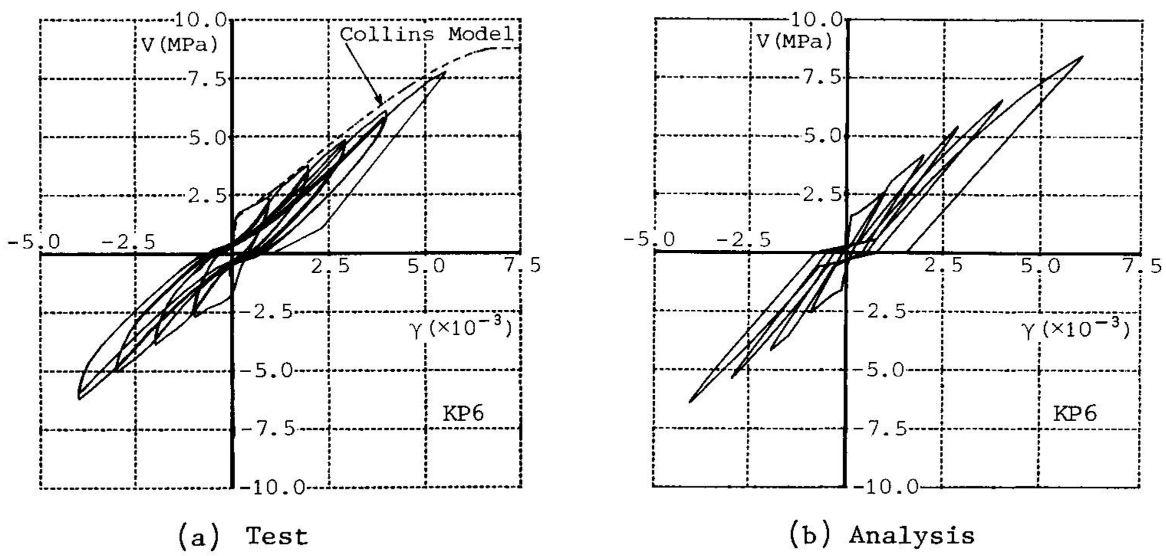
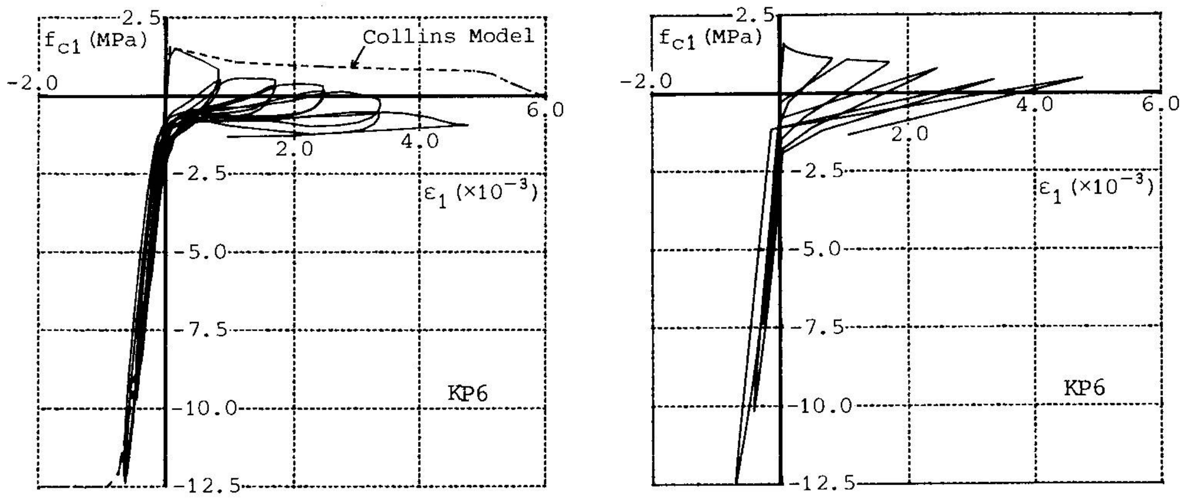


Fig.13 Shear stress - shear strain relationship



(a) Test (b) Analysis

Fig.14 Principal stress - strain relationship of concrete

As to the hysteretic loop of the shear stress-strain relationship, the following characteristic behaviors under reversed cyclic shear loading are well predicted by the proposed method such as, a) decrease of concrete stiffness at unloading stage, b) slip stiffness depending on residual strain and c) keeping a constant value of shear stress at shear strain equal to zero.

Next the predicted principal stress-strain relationship of concrete is presented in Fig. 14 (b). The analytical result well represents the dominant decrease of stiffness in the unloading stage from the tensile stress state, the successive slip phenomena and the restoration of compressive stiffness at the reloading stage.

4. CYCLIC ANALYSIS OF COLUMN

A column contains comparatively thick reinforcing bars which are arranged rather sparsely. From this specific feature a microscopic approach is effectual which can model the behaviors of reinforcements and bonds. As to the loading condition bi-directional lateral forces are very important. For such a case the three-dimensional approach is necessary.

4.1 Analytical Method

A reinforced concrete column is considered to be composed of core concrete, cover concrete, reinforcing bars and bonds. Nonlinear analyses are performed incrementally based on the nonlinear properties of each element. The approach is almost the same as indicated in the previous paper [7]. Hereunder only the main points are described below.

- Core concrete is represented by a hexahedral isoparametric element. The constitutive equation is defined by the plasticity theory based on the Drucker Prager's yield criterion like Eq. (4),(5). Here the parameter α is assumed by the simulation analysis of hooped columns subjected to axial forces. The hysteretic loop is defined as the tri-linear curve which is expressed under uniaxial force like Fig. 15.

$$f = \frac{3}{2}\alpha(\sigma_x + \sigma_y + \sigma_z) + f_0 = k \qquad \text{..........(4)}$$

$$f_0{}^2 = \frac{1}{2}\{(\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 + (\sigma_x - \sigma_y)^2 + 6(\tau_{yz}^2 + \tau_{zx}^2 + \tau_{xy}^2)\} \qquad \text{.... (5)}$$

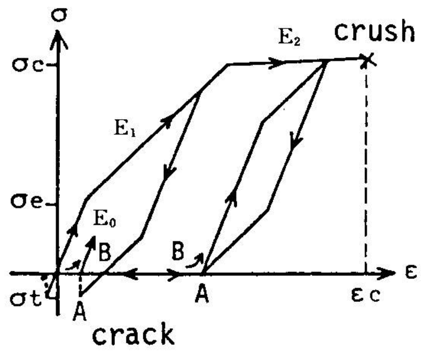


Fig.15 Hysteretic loop of core concrete

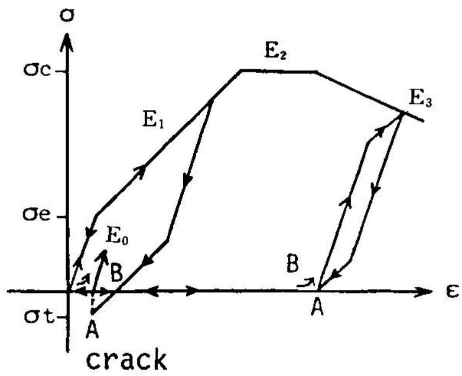


Fig.16 Hysteretic loop of cover concrete

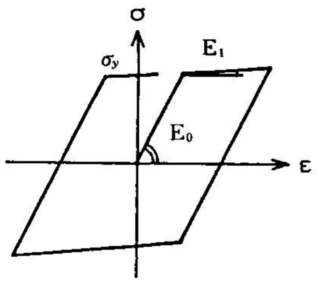


Fig.17 Hysteretic loop of reinforcement

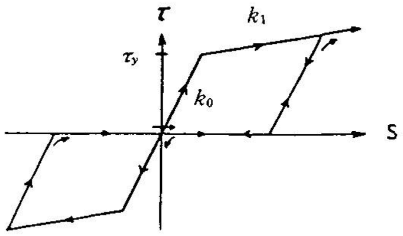


Fig.18 Hysteretic loop of bond

- Cover concrete is represented by a rod element or a *quasi*-three-dimensional element in which the displacements of each node are extrapolated from its adjoining core concrete. But the nonlinear characteristic is estimated from the axial stress only. This approach can make it simple to treat the negative property of cover concrete after peak stress like in Fig. 16.

- A reinforcing bar is represented by a rod element possessing only axial stiffness. The stress vs. strain relationship is assumed to be a bilinear loop as shown in Fig. 17.

- Bond is modeled as a set of link elements connecting reinforcing bars and concrete elements. For bond vs. relative diplacement in-between , a slip-type bilinear loop is assumed as shown in Fig. 18.

4.2 Column Subjected to One-dimensional Lateral Forces

4.2.1 Test Specimen and Analytical Model

The analyzed test specimen is a hooped column with the total longitudinal reinforcement ratio of 0.91% and the lateral reinforcement ratio of 0.36% [8]. The shear span-to-depth ratio is 2 and the compressive strength of concrete is 31.6 MPa. Many cyclic lateral forces were applied under rigid conditions at both ends with a constant axial force.

Analysis was performed for a quarter portion of the column like Fig. 19. The assumed mesh layout of core concrete is shown in Fig. 20. Here three cycles of loading were applied, that is, the first cycle up to 70% of the yielding load, the second cycle at yielding deflection δ_y and the third cycle at $2\delta_y$.

4.2.2 Analytical Results

An obtained deflection is shown in Fig. 21. As to the envelope curve the experimental results were well predicted. But the analyzed area surrounded by the hysteretic loop was smaller comparatively. To solve this problem it is necessary to estimate well the restoration of stiffness of cracked concrete considering the residual strain after cracking and to represent a more detailed hysteretic rule of bond slip behavior [9]. Such problems can be investigated adequately by two-dimensional approaches.

The strains of longitudinal reinforcement at the critical section are shown in Fig. 22. Analytical results represented well the observed behavior in tension and compression bars. Reversing loops were also well predicted.

The strains of lateral reinforcement parallel to the loading direction are shown in Fig. 23 (a) and (b). The remarkable phenomena were well predicted that under positive loading the strain suddenly increased after shear cracking of core concrete and shear forces were carried directly by the lateral reinforcement, and that under the reversing and sequent negative loading the strains were reduced rapidly at the first stage and then some residual strains remained.

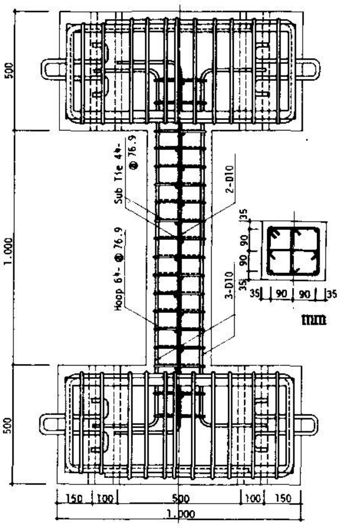


Fig.19 Test specimen

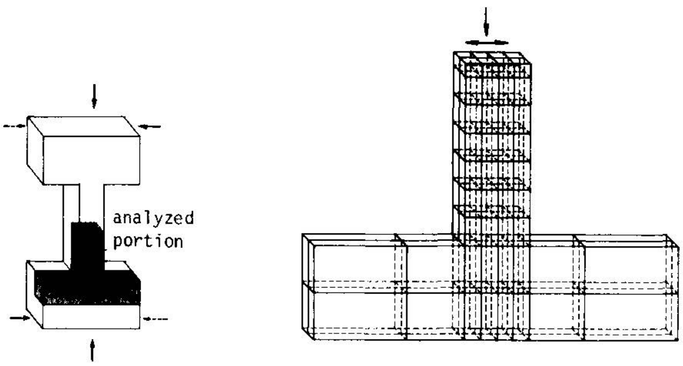
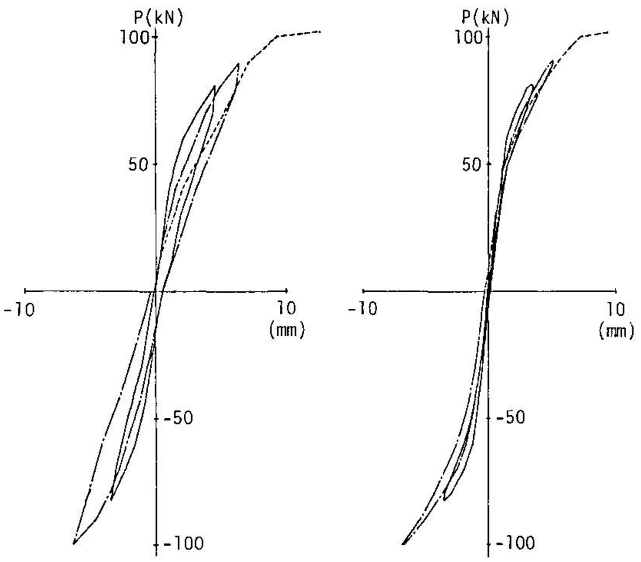
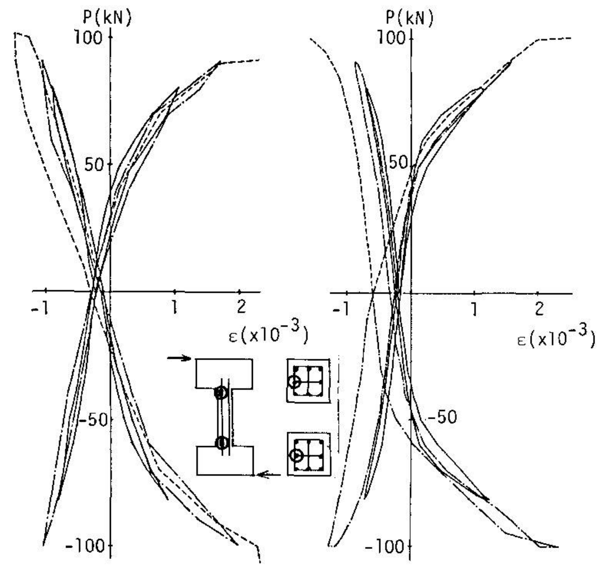


Fig.20 Mesh layout of core concrete



Test Analysis

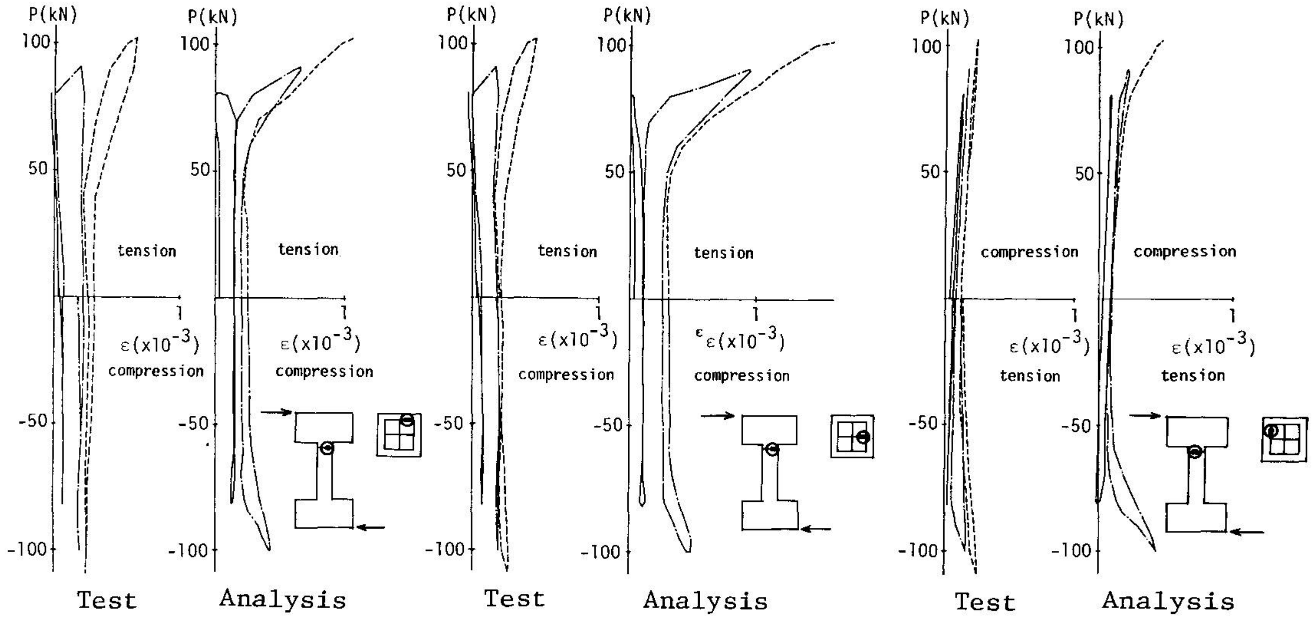
Fig.21 Load - story deflection relationship



Test Analysis

Fig.22 Load - strain of longitudinal reinforcement relationship

The strains of lateral reinforcement perpendicular to the loading direction are shown in Fig. 23 (c). Under positive loading the strains increased gradually and under reversing they recovered the same curve. Under negative loading the strains decreased at first and then increased after shear cracking. Such behavior can be investigated only by a three-dimensional analysis.



(a) Hoop parallel to loading direction

(b) Tie parallel to loading direction

(c) Hoop perpendicular to loading direction

Fig.23 Load - strain of lateral reinforcement relationship

4.3 Column Subjected to Two-dimensional Lateral Forces with Circular Displacement

4.3.1 Test Specimen and Analytical Model

The analyzed test specimen is a cantilever column with the total longitudinal reinforcement ratio of 1.42% and the lateral reinforcement ratio of 0.64% as

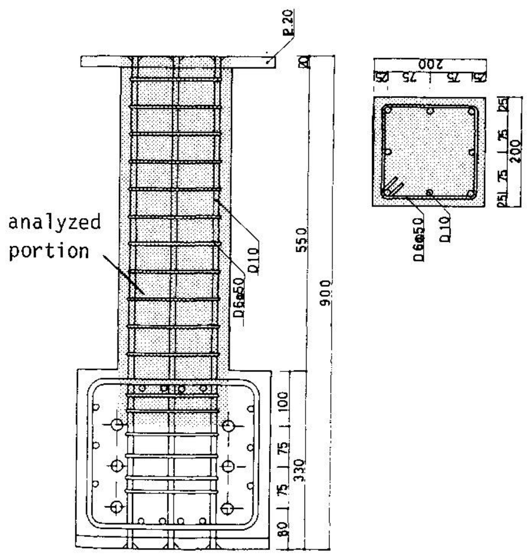


Fig.24 Test specimen and analyzed portion

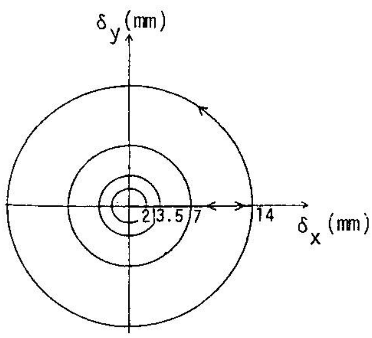


Fig.25 Hysteresis of displacement at top of column

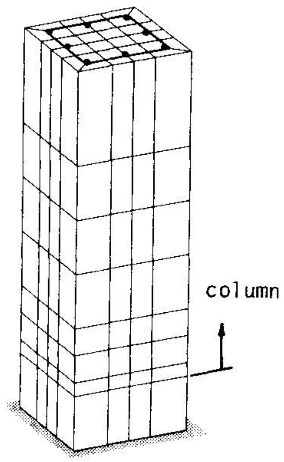


Fig.26 Mesh layout of concrete

shown in Fig. 24[10]. The shear span-to-depth ratio is 3 and the compressive strength of concrete is 15.7 MPa. Two-dimensional lateral forces were applied at the top of the column with circular displacements. The hysteretic displacement is shown in Fig. 25. That is, at first it became deformed in the X-direction from the center and then several circles were traced at a constant displacement.

The analyzed portion and mesh layout are shown in Fig. 24 and Fig. 26 respectively. The basement was modeled as elastic elements in which the anchorage of longitudinal bars could be considered. Axial loads were introduced at the first step and after the second steps the lateral displacements were given. Here one-dimensional diplacement was introduced up to 2.5 mm and after that two cycles were analyzed at a constant displacement of 2.5 mm.

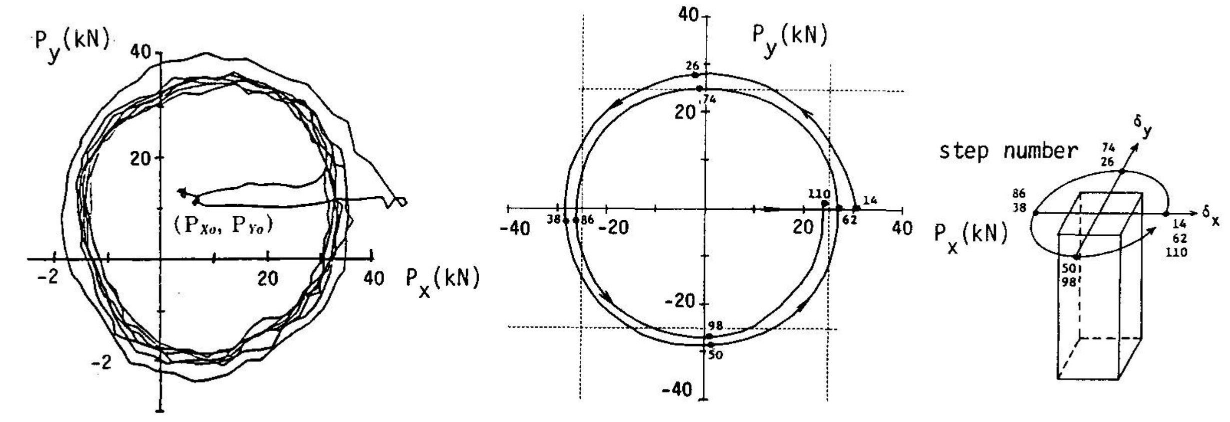
This application is considerably complicated in comparison with the one-dimensional cyclic loading because the stress direction changes every moment in accordance with the looped hysteresis.

4.3.2 Analytical Results

The hysteresis of shear forces in two directions is shown in Fig. 27. The test results showed that they became stable with almost circular shapes, excluding the first cycle of increasing displacements and that the direction of two-dimensional forces preceded the direction of two-dimensional displacements at about 15 degrees. Here the shear forces at the initial stage were not zero because the eccentric loads were added by the actuator weights.

In the analytical results the locus of shear forces showed almost circles from the zero point because of neglecting the initial eccentric loads. This locus predicted well the experimental results. As to the phase lag the angle of forces preceded at 7.5 degree to that of displacements. This tendency was observed in tests. But this value was nearly one half of the observed one. The discrepancy means that the area surrounded by the hysteretic loop of load vs. displacement curve was smaller than observations because the shear forces were smaller at the displacements zero. This problem has already been discussed in the previous one-dimensional study.

The analyzed strains of longitudinal bars and lateral bars are presented in Fig. 28 and Fig. 29 respectively. Here the loads in the X-direction were adopted. The results of longitudinal bars show that the hysteresis converged to certain constant loops excluding the first step with each different shape, inclination and rotating direction. From the results of lateral bars it was shown that the shapes of loops were different in accordance with their locations and that the



(a) Test (b) Analysis

Fig.27 Cyclic shear forces in two horizontal directions strains were accumulated while cyclic loading. The latter means that the damage of the column progressed while loading cyclically. This accumulation was verified in the previous study of one-dimensional loaded column.

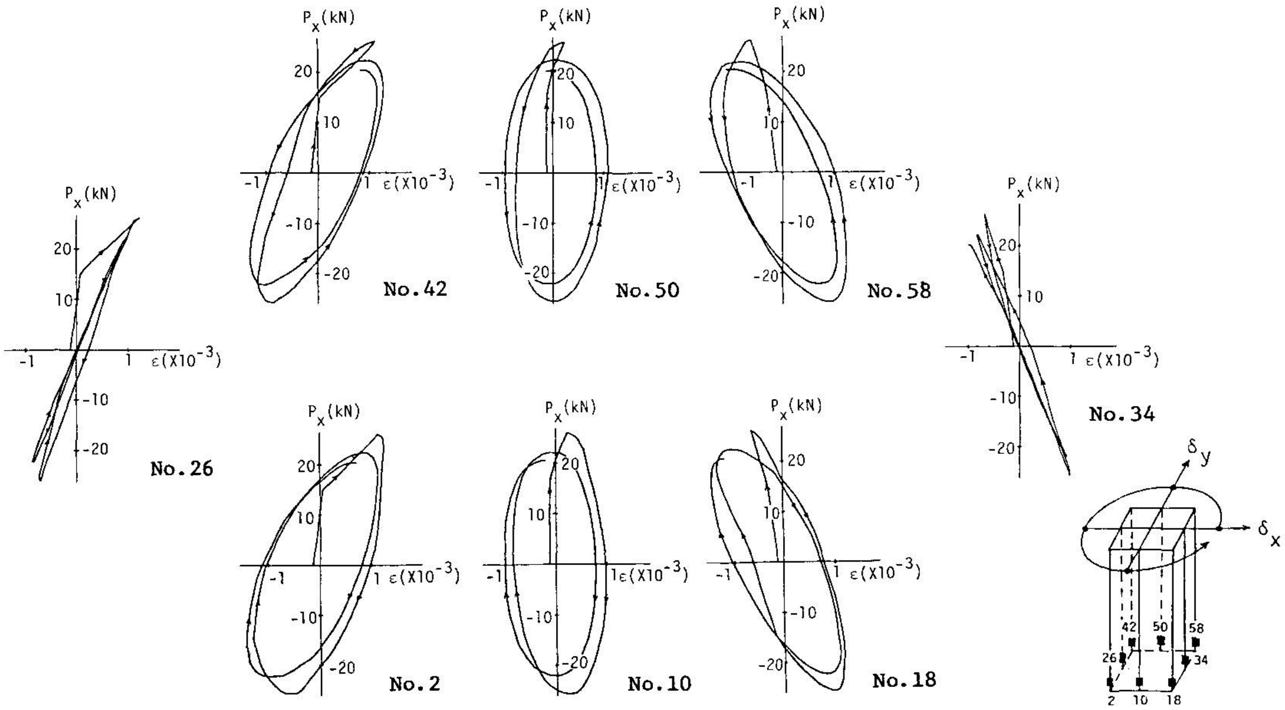


Fig.28 Load in X-direction - strain of longitudinal reinforcement relationship

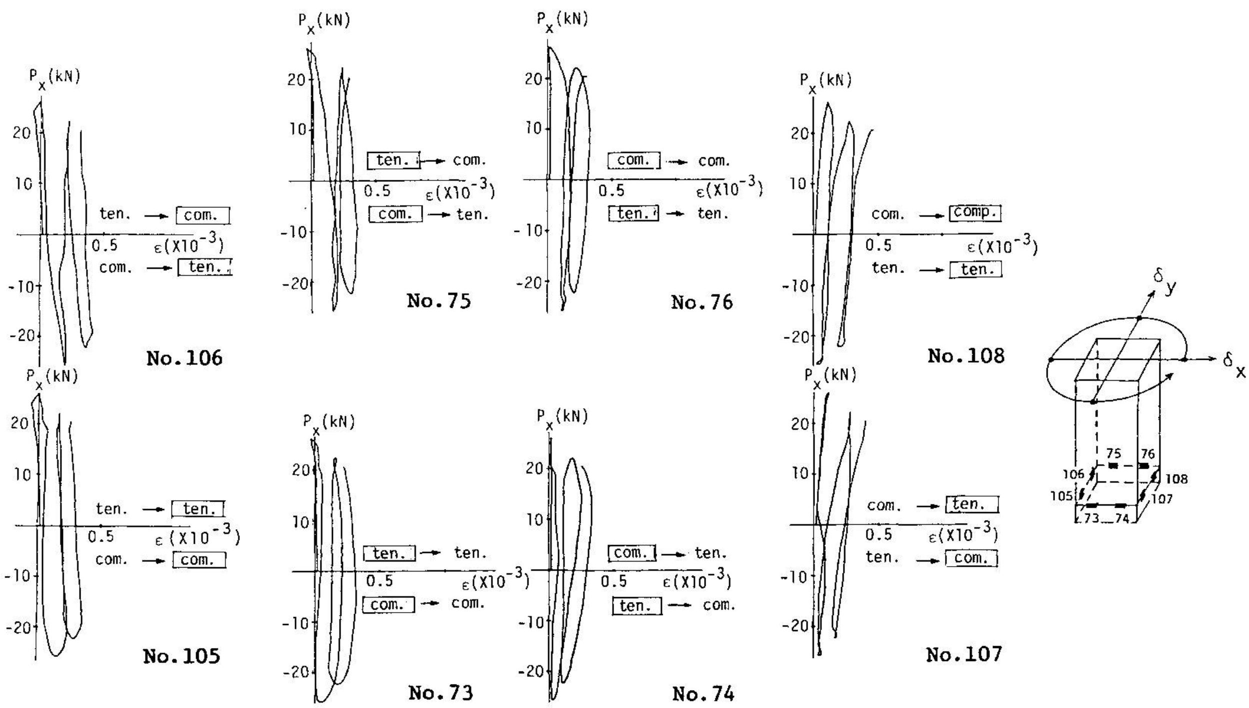


Fig.29 Load in X-direction - strain of lateral reinforcement relationship

5. CONCLUSION

A macroscopic approach based on the average stress strain relationship was effectual for a shear wall in which many reinforcements were arranged uniformly. A microscopic approach considering reinforcement and bond was effectural for a column with thick rebars which were arranged rather sparsely.

In the monotonic analysis of a shear wall subjected to cyclic lateral forces, it was found that the proposed method could predict the shear strain and the envelope curve of load vs. displacement relationship and that the residual strains should be considered to estimate the normal strains and dilatancy.

In the analysis of a panel subjected to cyclic pure shear forces, the cyclic behavior including the reversing loops could be well represented by modeling of residual strains, slip stiffness and restoring stiffness of concrete derived from experimental data.

In the analysis of columns subjected to cyclic one-dimensional or two-dimensional lateral forces, the proposed three-dimensional approach could represent well the envelope of load vs. displacement curve and the cyclic behaviors of reinforcements, especially the strain accumulation of the lateral reinforcements by modeling the three-dimensional characteristics of concrete and the slip behavior of bonds. But it is necessary to model more precisely the cyclic rules of each element in order to predict well the loop area surrounded by the load vs. displacement curve.

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